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**LONG-TERM EFFECTS OF SUPER
HEAVY-WEIGHT VEHICLES ON BRIDGES**

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Long-Term Effects of Super Heavy-Weight Vehicles on Bridges

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16. Abstract <p>A permit truck which exceeds the predefined limit of 108 kips is defined as a superload in Indiana. This study was conducted to examine the long-term effects of superload trucks on the performance of typical slab-on-girder bridges and to assess the likelihood of causing immediate damage. Typical steel and prestressed concrete slab-on-girder type bridges were analyzed using both beam line analysis and detailed finite element models. Furthermore, one prestressed concrete bridge and one steel bridge were instrumented using more than 50 sensors each. Strains and deflections were measured during a live load test, and each bridge was monitored for more than six months. Capacities of the investigated bridges were calculated and compared with the demands generated by various groupings of typical superload trucks. Analysis of the steel and prestressed concrete bridges demonstrated that typical superload trucks up to a gross vehicle weight of 500 kips are not expected to cause any damage or impair the long term performance of the investigated bridges. Serviceability limit states of the prestressed concrete bridges controlled the rating, and the bridges had adequate strength to accommodate all superloads included in the database. However, strength limit states controlled the rating of steel bridges. Long term monitoring of a continuous and a simple span bridge indicated that strains comparable to those of a 366-kip superload truck can be generated by regular truck traffic. The field measurements also demonstrated that the in-service behavior was different than the design assumptions. Fixity due to integral abutments, effectiveness of the continuity joint in continuous prestressed concrete bridges and contribution of the secondary members lead to notable differences between the expected and the anticipated behavior. Furthermore, the AASHTO girder distribution factor equation was found to be conservative for the investigated bridges. Use of a more accurate method such as FEA or the spring analogy method is recommended for the evaluation of bridges traversed by very heavy superload trucks.</p>			
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LIST OF SYMBOLS

A	area of solid section
A_1	load factor for dead load
A_2	load factor for live load
A_I	area of parapet
C_I	capacity of member
C	material crack growth constant
D	dead load
D	deflection
E	elastic modulus
$E_{meas.}$	experimental elastic modulus
$E_{calc.}$	calculated elastic modulus
F_G	geometry correction factor
G	shear moduli
GDF	Girder distribution factor
H	weld leg height
I	moment of inertia
I_r	polar moment of inertia
I_{NA}	moment of inertia of the parapet with respect to the neutral axis of the entire bridge cross-section
I_{NAP}	moment of inertia of the parapet with respect to its own neutral axis
J	torsional constant

IM	impact factor
K	stress intensity factor
K_T	torsional constant
K_g	longitudinal stiffness parameter
K_{TD}	torsion bar stiffness used in the spring analogy model for calculation of girder deflections
K_{TM}	torsion bar stiffness used in the spring analogy model for calculation of bending moments
L	extreme distance between any group of two or more consecutive axles (ft)
L	live load
L	spacing of the beams
L	sum of stiffener plate widths (at both sides of the web) and web thickness
L	span length of beam (ft)
L	span length
L	spacing of beams in spring analogy
M	flexural bending moment
M	bending moment or couple
M_{DL}	dead load moment
M_{LL}	live load moment
M_{NA}	couple resisted by the parapet about the neutral axis of the entire bridge cross-section
M_{NAP}	bending moment resisted by the parapet about its neutral axis
M_n	nominal moment capacity
M_t	total flexural bending moment
N	number of axles being considered
N	number of cycles
N_T	total fatigue life
N_1	number of cycles for the first phase of crack propagation at a welded stiffener
N_2	number of cycles for the second phase of crack propagation at a welded stiffener

P	axial force
RF	rating factor for live load capacity
S	girder spacing (in)
S	girder spacing (ft)
S_R	nominal stress range
S_{RE}	maximum stress generated at the edge of the stiffener plate due to in plane bending
S_{RT}	stress normal to the critical plane
T	fundamental period of vibration
$T_{calc.}$	calculated period
$T_{meas.}$	measured period
V_{DL}	dead load shear force
V_{LL}	live load shear force
W	maximum allowable gross weight of two or more consecutive axles (lb)
a	crack size
a_i	initial crack length
a_f	final crack length
d	distance between the neutral axis of the entire bridge cross-section and the neutral axis of the parapet
e_g	distance between center of gravity of beam and center of gravity of deck (in)
f'_c	compressive strength of concrete
f_{pu}	ultimate strength of prestressing steel
f_s	strand stresses
f_y	yield stress
g	girder distribution factor (single lane)
g	web gap length
k	stiffness of the system
k_{calc}	calculated stiffness
$k_{meas.}$	measured stiffness
m	mass

n	modular ratio between beam and deck material
t_p	thickness of stiffener plate
t_p	thickness of deck (in)
t_w	thickness of web
y	distance from neutral axis
w_{flange}	top flange width
Δ	relative deflection of the beams
Δd	maximum relative deflection value
ΔK	range of stress intensity factor
γ_{DL}	load factor for dead load
γ_{LL}	load factor for live load
ε	strain
κ	curvature
σ_{DL}	dead load stress
σ_i	stress at the bottom flange of the i^{th} girder
σ_{LL}	live load stress
σ_{max}	maximum stress
σ_{wg}	maximum stress in the bottom of top web gap
σ_x	longitudinal stress
ϕ	load reduction factor

CHAPTER 1. INTRODUCTION

Growth of industry in last decade has lead to a higher demand for energy. Furthermore, massive construction projects for power plants and factories have started taking place across the country. Accordingly, large non-divisible loads such as transformers, pressure vessels or heavy machinery must be transported through the highway network. Transportation of these heavy loads on the major highways have raised some concerns about the response of aging infrastructure.

Highway transport of heavy materials and equipment is sometimes limited due to legal weight restrictions on the national highway systems. To accommodate the transport of these materials and equipment, state departments of transportation (DOTs) often conditionally allow passage on their highways and bridges. For passage over state bridges, state DOTs, such as the Indiana Department of Transportation (INDOT), require special permits that detail the proposed route and a list of all state owned bridges on that route. In addition, these special permits specify the axle weights and spacings of the trucks used to transport these heavy materials and/or equipment. Transport trucks used to carry very heavy loads are often referred to as Superloads. Superload trucks typically carry heavy, non-divisible components for industrial facilities. The trailers of these trucks usually have special configurations in order to spread out the heavy load to multiple axles and to provide a load distribution comparable to that for a regular truck.

The Indiana Department of Transportation requires special permits for all Superloads, which they define as any truck with a gross vehicle weight (GVW) in excess of 108,000 lb (Indiana Department of Revenue, 2001). Once provided the request for

these special permits, state DOTs perform a structural analysis for each bridge on the proposed route, given the detailed truck configuration. Based on the results of these structural analyses and any additional information regarding specific bridges, a decision is made to allow the *unrestricted passage*, allow a *restricted passage*, or *disallow passage* of the load over each bridge. Restricted passage usually consists of traveling in a specified lane and/or slowing down to a speed of 5 mph.

The motivation of this study is to investigate the influence of increased superload traffic on bridge structures. Approximately 1,500,000 overload trucks traveled on the highway network of the United States in the federal fiscal year 1989 according to permit applications; statistics indicate an increase in both the number and weight of overload vehicles (Fu and Hag-Elsafi, 2000). Analysis of the recent (between 1989 and 2000) bridge failures in the United States reveals that 8.8% of 503 reported failure cases were due to overloads and 8.6% were due to deterioration (Wardhana and Hadipriono, 2003). Throughout the nation, a significant number of bridges may be damaged, possibly to failure, due to the short term effects of superloads (i.e., overloading) and/or due to long term effects of superloads (i.e., accelerated deterioration). Therefore, the impact of superload vehicles on bridge structures requires further research. It should also be noted that about 130,000 of the approximately 600,000 bridges forming the U.S. bridge network are rated as structurally deficient (Ghosn and Moses, 2000). The increasing number of superloads may endanger the safety of highway network and increase the number of deficient bridges; their short and long term effects must be evaluated and mitigated.

The main objectives of this study are to investigate the effects of superloads on bridge structures and to develop a strategy for simplifying the evaluation of these effects. The scope of this study is limited to slab-on-girder bridges, typical on the interstate highway network. Girders of slab-on-girder bridges are made mostly of steel or prestressed concrete. This study focuses on the girders and the secondary members of

representative highway bridges. Evaluation of the substructure was beyond the scope of this study, although it might be critical for some bridges.

1.1. PREVIOUS STUDIES ON SUPERLOADS

Effects of superloads have been investigated by researchers, but most of these studies had a limited scope. Observations of the researchers only during and after the superload passages were reported. Long term effects of superloads were not evaluated. Notable studies on superloads are summarized below.

Duncan (1977) analyzed bridges in South Africa for superload effects and emphasized the importance of accurate techniques to assess the effects of superloads on bridges in order to utilize lower margins of strength for controlled superload passages. Kostem (1978) investigated the effects of overloads with respect to deck damage and emphasized the importance of including the material nonlinearities in the analysis in order to represent the redistribution of forces throughout the structure. Kostem (1978) also reported that the damage initiates through flexural cracking of the deck slab in the overloading of the bridges.

Turer and Aktan (1999) instrumented and analyzed three steel-girder-bridges in Ohio for the passage of an 817-kip superload truck carrying a cooling unit. Cross-frames, which connect the girders to each other and provide stability, were found to be the most heavily stressed members with a stress range of 10 ksi. Also, the transverse deck stresses due to bending in the transverse direction were found to be critical. However, the stress level of the girders remained at about 5 ksi. No damage was detected in any of the three bridges. One common damage mechanism was stated as the loss of composite action, but this type of damage was not observed. Also, the deflections were well below the traditional limit of the span length divided by 800 (AASHTO, 2004).

Ziehl and Lamana (2003) measured strains and deflections of a prestressed concrete (PC) girder bridge in Louisiana before, during and after a superload passage. The acoustic emission technique was also used for the evaluation of damage. The researchers did not observe any damage or change in the response of the bridge after the superload passage.

1.2. RESEARCH OBJECTIVES

The main purpose of this study is to evaluate the effects of superloads on steel and prestressed concrete bridges. Both long term and short term effects of superloads are investigated for slab-on-girder bridges. Furthermore, development of a simple method to predict the bridge response for minimizing the effects of superloads without imposing overly conservative load limits on bridges was considered.

A simple and accurate structural analysis method for the analysis of slab-on-girder bridges and possible damage mechanisms for bridges traversed by superload trucks is believed to be necessary for adequately capturing the bridge response, but without requiring a detailed analysis. Current methods used and proposed for bridge design and rating do not include any modification for the superload trucks. A bridge traversed by a superload truck is typically analyzed as if a regular truck was crossing the bridge. However, this approach may not be accurate enough for the analysis of bridges subjected to superloads, since it has a built-in conservatism and it does not provide the three-dimensional response.

1.2.1. DAMAGE PREDICTION AND DAMAGE MODEL

The current approach to possible fatigue problems related to superload trucks is not different than that for regular trucks. The effect of an overload on the long-term

performance of a bridge is not considered. Several overloads can alter the fatigue response of a bridge. For example, a fatigue-prone detail may become more critical after the passage of several superload trucks, or overload cycles may increase the fatigue life due to load interaction effects. It can also be shown that periodic overloads can also have a beneficial effect on fatigue critical details. Possible damage mechanisms are investigated in detail.

Short term effects of superload vehicles or immediate damage caused by a superload truck have been evaluated using the Miner's cumulative damage equation by several researchers (Dicleli and Bruneau, 1995; Mohammadi and Polepeddi, 2000; Wang et al., 2005). It was assumed that each superload passage consumes a fraction of the fatigue life. Furthermore, limiting the number of superload passages with respect to the reduction in service life of the bridge structure due to the superload trucks was recommended (Mohammadi and Polepeddi, 2000). The applicability of this approach to evaluation of superload effects is investigated.

For prestressed concrete bridges, superloads may cause cracking of the girders. This may not impair the short term safety, but can modify the response of a bridge to regular traffic. Cracking of the girders may lead to corrosion of the strands. Therefore, long term performance of the bridge can possibly be affected by the passage of superload trucks. Prestressed concrete girder and tendon stresses are evaluated to examine cracking and long-term damage.

1.2.2. A SIMPLE STRUCTURAL ANALYSIS METHOD FOR PREDICTION OF BRIDGE RESPONSE

A simple and accurate method of analysis can provide more realistic results for the permitting process than a beam line analysis. Development of a structural analysis model which can predict the overall bridge response better than a simple approach such as the use of girder distribution factors (GDFs), but which requires less computational

effort than a detailed three-dimensional finite element analysis, is one of the key steps to providing a practical approach for the evaluation of superload effects on bridges. A simple and accurate structural model will also be useful for ordinary trucks; however, it is believed to be essential for the evaluation of bridges and permitting of superload trucks.

Analysis of a bridge for superload trucks requires an accurate modeling technique which can predict both global response and local behavior for critical locations. The girder distribution factors (GDFs) computed with respect to the AASHTO (2004) were found to be higher compared to those obtained from detailed analyses and field tests (Sotelino et al., 2004; Mabsout et al., 1997; Eom and Nowak, 2001). Therefore, inaccuracies in the structural analysis of bridges may result in lower bridge ratings, and a permit application for a superload truck may be refused conservatively. A detailed finite element analysis (FEA) may be preferred for higher accuracy, but it would be time consuming and require considerably more computational effort than a beam line analysis. In addition to advanced structural analysis software, a detailed three dimensional analysis also requires significant mesh generation and post-processing effort. As a result, it may not be as practical as a beam line analysis.

The traditional beam line analysis enables computation of maximum bending moments or shear forces resisted by the girders. However, it has been observed that out-of-plane bending of the web gap region causes distortion of the web and results in high stresses in the vertical direction (Castiglioni et al. 1988; Fisher and Keating, 1989; Roddis and Zhao, 2003; Jajich and Schultz, 2003). Interaction of the bracing elements with the girders also stresses the bracing elements and their connections. Although this interaction may result in short term and long term damage, it is not considered in the design and rating of bridges. Measures such as avoiding fatigue prone details or configurations in this region and providing a thicker web can improve the performance of a bridge subjected to regular truck traffic. Meanwhile, some superload trucks can cause much higher stresses than regular traffic and can jeopardize the structural integrity of a bridge or lead to long term damage such as fatigue cracks in the web. This possible

damage mechanism may not be predicted analytically unless a detailed analysis is performed. Therefore, a simple and accurate tool for predicting the out of plane bending response and the live load demand on the bracing components is needed.

Also, the contribution of secondary members such as cross-frames and parapets can be beneficial during a superload passage. Field monitoring demonstrates that bridges typically have a higher capacity than that predicted by traditional analysis methods (Ghosn et al. 1986; Mabsout et al., 1997; BDI, 1999; Eamon and Nowak, 2002). The unexpectedly higher capacity is attributed to the contribution of the deck and the secondary members. However, a practical approach for including the effects of secondary members such as parapets has not been proposed to date. The parapets, which have similar geometries and reinforcement properties across the nation, contribute considerably to the flexural stiffness of a bridge and have adequate capacity to resist the loads generated by superload trucks (Akinci et al., 2005).

1.3. RESEARCH METHODOLOGY

Possible problem locations of bridge superstructures are determined by comprehensive numerical analyses and instrumentation. The instrumentation program and three dimensional analyses are used to reveal the strain ranges in these locations. Live-load tests in addition to long-term monitoring are used to evaluate the response of a bridge to loading and to predict the effects of superloads. The damage mechanisms due to superloads and life of fatigue prone details are estimated accordingly.

Detailed finite element models (FEMs) for these four bridges are analyzed using SAP2000 and ANSYS programs. Secondary members are included in the FEMs, and a refined mesh is used at the secondary member and girder connections for the steel bridges. For steel sections, material and geometric nonlinearities are included in the models prepared using ANSYS.

Three dimensional finite element analysis results and bridge response data gathered through instrumentation are used for the development and verification of the tools for assessing the response of bridges to superloads. Typical bridges selected from INDOT's bridge database are analyzed and instrumented for refinement of the proposed methods and improvement of recommendations with respect to the response of actual structures. Two steel and two prestressed concrete (PC) bridges representative of many bridges found in Indiana have been selected from an extensive bridge inventory for detailed study.

A simple structural analysis method for slab-on-girder bridges is developed and refined with respect to the finite element analysis results. Bridges representing a range for important parameters such as girder spacing, deck width and span properties are used for the calibration. Existing databases of steel and PC bridges analyzed by other researchers are analyzed with the proposed method and the results are compared for further validation of the simple analysis method. More than 80 bridges were included in the existing databases.

One steel and one prestressed concrete is instrumented with sensors to measure strains and deflections. A special strain transducer is developed for measuring strains on concrete surfaces with an acceptable accuracy.

1.4. SUPERLOAD TRUCKS

The Indiana Department of Transportation defines a superload as any vehicle with a gross vehicle weight (GVW) in excess of 108,000 lb that does not meet one of six approved axle configurations (Figure 1.1) (Indiana Department of Revenue, 2001). superloads with a GVW greater than 200,000 lb require structural analyses of all bridges on the proposed route. Since the main objective of this study is to compare the results

from the structural analysis software used by INDOT with the actual responses of bridges, only superloads with a GVW greater than 200,000 lb are included in this study.

The structural analysis by INDOT provides one of four results that regulate the passage of superloads over bridges. The permit could be: 1) approved for unrestricted passage across a given bridge, 2) approved for passage with a restricted maximum driving speed of 5 mph to eliminate impact, 3) approved for passage with restrictions on speed and the transverse location on the bridge, or 4) denied passage across a given bridge. Generally, a permit is only denied when a bridge has a very low load rating for given superload axle configurations.

1.4.1. AXLE CONFIGURATIONS

Superloads, like all other Interstate traffic, are subject to axle configuration regulations set forth by both AASHTO and State DOTs. Federal agencies such as AASHTO have developed a federal bridge formula, which is detailed in the “Oversize/Overweight Vehicle Permitting Handbook” (Indiana Department of Revenue, 2001) is used to compute the weight of axle groups:

$$W = 500 \left[\frac{LN}{N-1} + 12N + 36 \right] \quad \text{Eqn. 1.1}$$

where

W =Maximum allowable gross weight of two or more consecutive axles (lb)

L =Extreme distance between any group of two or more consecutive axles (ft)

N =Number of axles the group being considered

Using the maximum axle weight tables corresponding to the federal bridge formula, upper limits to axle weights are set. This weight is, however, allowed to be exceeded with sufficient lateral load distribution measures or through a separate permit.

Increasing lateral load distribution is commonly done by increasing the number of wheels per axle (more than 4 wheels per axle), and/or widening the axle wheel base or gauge width (greater than 6 ft). Additionally, axle limits are set in order to control damage to pavements. Typically, heavier axle loads with longer spacing are more detrimental to pavements since loads are more concentrated. For this reason, it is generally preferred to maximize the number of axles present in a group and minimize the spacing between axles (U.S. DOT, TS&W Study, 2000).

Although numerous axles with small spacing are more beneficial for pavements, this type of configuration can cause other problems. One such problem is the truck's ability to turn. Generally, more than four axles spaced at the minimum limit is avoided since tires tend to slide or skid across the pavement rather than smoothly rolling over the pavement when turning. The minimum axle spacing allowed is predominately a function of the tire size. For superloads, this minimum spacing is typically around 54 in for standard tire sizes, but a shorter spacing is allowed when smaller tire sizes are used.

Another concern associated with numerous axles at the minimum spacing is that they create large loads at relatively concentrated locations with respect to bridge spans. This concentration of large loads has its greatest effect on short span bridges since it tends to cause high positive moment stresses when situated at the center of short spans. This is commonly the case when the total length of a truck is longer than the span it is traversing. Longer bridge spans typically are not affected as severely by multiple-closely spaced axles. On longer spans, the entire truck weight may be on the bridge, but due to the longitudinal distribution of the weight over the total length of trucks, the positive moment stresses are generally not critical. The results of having the entire truck on a bridge could however have the reverse effects when considering negative moment stresses. Concentrated axle groups separated by a long spacing could cause large negative moments when the truck is straddling the pier of a continuous span bridge (U.S. DOT, TS&W Study, 2000).

1.4.2. SUPERLOAD CLASSIFICATION

To gain an understanding of typical superload axle configurations which are analyzed by INDOT, an inventory of approximately 80 superload permits submitted to the Indiana and Wisconsin DOT's (INDOT and WisDOT) were analyzed. The trucks described varied in total length from 75 ft to 255 ft. Gross vehicle weights varied from 200,000 lb to 1,174,000 lb. The loads being transported included generators, turbines, bridge girders, excavators, cranes, transformers, and other specialty equipment. In some cases, the axle configurations of the superloads were designed specifically to transport a particular type of load. The destinations of some superloads may be in areas with restricted access or very sharp turns, which require certain axle configurations to allow improved maneuverability. Depending upon the geometry of the loads and how they are placed on the truck bed, the length and width of the superload varied. The variety of axle weights and spacings greatly complicates efforts to categorize trucks. Figures 1.2 to 1.5 show some actual superload trucks.

With approximately 80 superload axle configurations, it was necessary to devise a system for grouping the trucks so that they could be analyzed with a finite element model (FEM). Sketches of all superloads were drawn to scale to identify trends in axle spacings. Trucks with similar axle configurations were then grouped together. Initially, eight different groups were developed. Each group was then represented by a single "average truck" which approximated the average axle spacings and axle weights of all the trucks in their respective group.

The "average truck" from each group was then analyzed in a beam line analysis influence-line model to generate maximum moments for various bridge span lengths. Based on the maximum moment results and GVWs, the superload groupings were condensed into four groups. A plot showing the maximum HS20 normalized midspan

moments for various span lengths is shown in Figure 1.6, and schematics of each of the truck groups can be seen in Figure 1.7. A Maximum Truck which represents the most severe superload permit that INDOT has processed in recent years was also included in the classification system to provide an upper-bound for analysis (Figure 1.8). In addition, an HS20 truck and a Toll Road Loading which are commonly used in bridge design are also included for comparison (Figure 1.8). A tabular description of each of the trucks used in the final classification system can be found in Table 1.1.

Group A Superloads represent the vast majority of superload permits submitted to INDOT. They range in GVW between 200 kips and 210 kips. Group B Superloads represent the next largest group of superload permits. These superloads are longer than the Group A Superloads, have more axles, and range in GVW between 210 kips and 300 kips. Group C and D Superloads represent the extreme minority of permits and exceed a GVW of 300 kips. Both of these groups are most likely specially designed to carry specific loads, and hence were more difficult to generalize. Group C Superloads have widely varying axle configurations, and range in GVW from 300 kips to over 1,000 kips. Group D Superloads represent double-lane trailers which have a gauge width of 10 ft. This group's wheel base is comprised of eight tires per axle, which allows for heavier axle weights. Group D Superloads can weigh anywhere between 200 kips to over 1,000 kips. The majority of Group D Superload Permits obtained from INDOT and WisDOT have GVWs between 400 kips and 850 kips.

1.4. OUTLINE OF THE STUDY

The study has six chapters and two appendices. In Chapter 1, the problem statement, motivation of the study, research objectives and description of the superload trucks are presented. Properties of the investigated steel and PC bridges are presented in Chapters 2 and 3, respectively. In Chapter 2, FEM analysis results and capacity evaluation of two steel bridges are presented. Furthermore, load test and long term

monitoring results of a plate girder bridge are presented. Also, possible damage mechanisms in steel bridges due to superload trucks are discussed in Chapter 2. Chapter 3 is similar to Chapter 2 in terms of organization and content, but it covers the effects of superload trucks on PC bridges. Analysis results, capacity evaluation and damage mechanisms for the investigated PC bridges as well as the load-test and long term monitoring results of a PC bridge are presented in Chapter 3. Chapter 4 shows the effects of parapets on the live load response of bridges subjected to superload trucks. Demand versus capacity comparison for the parapets of the investigated bridges is also presented in Chapter 4. In Chapter 5, a new method to predict the three-dimensional live load response of slab-on-bridges is introduced. The accuracy of the proposed method is evaluated with respect to detailed three-dimensional finite element analysis results. Finally, Chapter 6 summarizes the findings and conclusions of the study and includes recommendations. In Appendices A and B, as-built drawings of the investigated steel and PC bridges are presented.

Table 1.1 Superload truck information

Loading	GVW (kip)	Total Number of Axles	Total Truck Length
HS-20	72	3	28'-0"
Toll Road Loading	90	5	28'-0"
Group A	201	9	80'-2"
Group B	247	13	125'-8"
Group C	366	17	152'-8"
Group D*	500	16	95'-0"
Max Truck*	824	21	127'-7"

* Trucks have 8 tires per axle

Figure 1.1 Preapproved INDOT axle configurations (Indiana Department of Revenue, 2001)

Figure 1.2 13-axle Superload Truck (Source: www.diamondheavyhaul.com)

Figure 1.3 9-axle Superload Truck (Source: www.guymtturner.com)

Figure 1.4 13-axle Superload Truck (Source: www.guymtturner.com)

Figure 1.5 Dual Lane Superload Truck (Source: www.diamondheavyhaul.com)

Figure 1.6 HS-20 normalized maximum midspan bending moments for various Superload trucks at varying span lengths

Figure 1.7 Axle configurations of Superload Groups

HS20

GVW: 72 Kip
Total Length: 28'-0"
2-tires per axle.

Toll Road Loading No. 1

GVW: 90 Kip
Total Length: 28'-0"
4-tires per axle.

Maximum Truck

GVW: 824. Kip
Total Length: 127'-7"
8 tires per axle

Figure 1.8 Axle configurations of Design Trucks and Maximum Truck

CHAPTER 2. EFFECTS OF SUPERLOADS ON STEEL BRIDGES

Effects of superloads on slab-on-steel girder bridges are discussed in this chapter. Two steel bridges, representing a range of steel bridges found in Indiana, were analyzed; one of them was instrumented to identify behavior and possible damage mechanisms due to superloads. Since the bridge selected reflects the current highway bridge design practices in Indiana, some of the results from the constructed FEM may be applied to other Indiana State Highway bridges with similar details subjected to similar loadings. Both of the investigated bridges, as well as the analysis and experimental techniques used in the evaluation of these bridges, are described in detail. Furthermore, short term and possible long term damage mechanisms due to superload trucks are assessed.

2.1. BACKGROUND

Many bridges across the world have been analyzed using both finite element models and various forms of instrumentation. To learn from previous studies, a detailed literature review was conducted. This literature review consisted of first gathering information that would assist in developing an accurate finite element model of a selected bridge. Secondly, this literature review incorporated experimental results and instrumentation plans from previous studies in which bridges were load tested with unusually heavy trucks. Although there have been numerous studies on the behavior or fatigue resistance of steel bridges, there are only a few studies on the effects of superloads on steel bridges. Some of these studies are summarized in this section.

2.1.1. FINITE ELEMENT MODELING

The development of an accurate model of the selected bridge makes estimation of the effects of many different loading scenarios possible. For this reason, an investigation into FEA modeling assumptions was conducted. It was determined that a detailed model of the full bridge would be most desirable to simulate loadings in various travel lanes. In addition, all potential load carrying members including parapets, sidewalks, cross-frames, and diaphragms should be analyzed so as to realize the full capacity of the overloaded bridge, and to identify any potentially vulnerable locations.

Numerous modeling schemes have been used to evaluate bridges. A study by Mabsout, et al. (1999) compared four different modeling techniques. One modeling technique idealized the concrete slab and steel girder web as quadrilateral shell elements, while the girder flanges were idealized as space frame elements. The concrete deck was eccentrically connected to the top girder flange by means of rigid links. A second modeling technique idealized the concrete slab as isotropic brick or solid elements, and the steel girder flanges and web as quadrilateral shell elements. All four of the modeling techniques analyzed in this study were limited to linear elastic analysis of the concrete slab. This study concluded that all four modeling techniques give similar results and lateral distribution factors. Furthermore, the results of a more detailed analysis of the second model described above correlated well with both published test data and the NCHRP 12-26 distribution factors (Zokaie, et al. 1991).

It is widely accepted that secondary structural elements such as parapets and cross-frames contribute to the distribution of load to the main structural elements. Eamon and Nowak (2002) attempted to quantify the effects of these secondary elements by means of finite element analyses. Cross-frame members were modeled as beam elements connected to steel girders at upper and lower junctions of the web and flange. Barriers (or parapets) were modeled as solid elements, and were assumed to make a “complete connection (perfect strain compatibility)” with the deck. Using these modeling

assumptions and an elastic analysis, they concluded that barriers and diaphragms (or cross-frames) together could reduce maximum girder moments by 11 to 25%. Furthermore, this study suggests that girder distribution factors could be decreased by 10 to 40% when secondary elements are considered in the analysis of a bridge.

One of the selected steel bridges for this study has fully integral abutments. Although the State of Indiana requires that integral abutment bridges be designed as having pin and roller supports, it is reasonable to expect some rotational and translational restraint from girders embedded 15 in into a concrete abutment. As will be demonstrated in this study, the end support conditions assumed in FEMs greatly affect the predicted live-load response of the modeled bridge. Lawver, French, and Shield (2000) instrumented a three-span integral abutment bridge and monitored seasonal responses as well as live-load responses. Their live-load test results suggest that the integral abutments do provide some rotational restraint. Based on midspan moments, end moments, and deflections, their measured data suggests approximately 55-65% fixity from the integral abutments.

2.1.2. FIELD TESTING FOR EFFECTS OF SUPERLOADS ON BRIDGES

Due to the increased frequency of superloads across the country, several studies that include field test data have been conducted. Turer and Aktan (1999) studied the influence of an 817-kip Superload on three steel stringer bridges. This project included both the instrumentation and FEA of all three structures, one of which was an integral abutment bridge. Between 66 and 74 sensors, including strain gages, displacement transducers, and tiltmeters, were installed on each bridge. Strain gages were placed in pairs of two on both the girder top and bottom flanges. Cross-frames were also instrumented with strain gages. It was found that the maximum live load girder stresses caused by the 817-kip Superload did not exceed 5 ksi for any of the bridges studied. Larger stresses (up to 10 ksi), which were predominantly due to differential deflections

between adjacent girders, were observed in cross-frame members. FEA also predicted dead-load stresses to be three to five times greater than live-load stresses caused by the 817-kip Superload. No immediate damage was detected in any of the three bridges.

Minor and Woodward (2001) also conducted a study to evaluate the performance of various bridges subjected to superloads. The superloads used in this study were part of the Military's Heavy Equipment Transporter System (HETS) and weighed approximately 220 kips each. Analytical and field test data were collected to generate a bridge rating program for bridges on the HETS. No damage was reported as a result of these superloads. In fact, it was found that "the HETS vehicle distributes its high weight very effectively throughout the bridges analyzed." It was also found that at maximum speed the HETS truck produced less than 5% impact. This low impact was attributed to the HETS gradual introduction of load onto the bridge due to its wide wheel base (8 tires per axle).

Another similar study was conducted for the Ohio DOT by Helmicki and Hunt (2004). A six-span steel stringer bridge was modeled and field tested for superloads weighing up to 883 kips. Analysis and field test data were then compared with analytical predictions obtained with BARS, a common bridge rating program used by DOTs. Based on preliminary FEA, it was decided that critical regions that should be instrumented were at locations of maximum positive and negative bending moments (at midspans and over piers). The top and bottom flanges of the three interior girders were instrumented with transducers at the critical regions. A calibrated FEM was generated based on a controlled load testing. This study predicted maximum girder stresses of 10 ksi, some temporary loss of "unintended composite action between the steel girders and the concrete decking," and some minor transverse cracking of the concrete deck.

Although work has been done on analyzing the effects of superloads on highway bridges, several issues remain unclear. The long-term effects of superloads on the fatigue lives of various load carrying members remain unknown; they are highly dependent upon

a variety of bridge details. Furthermore, the bridges included in many of the studies described previously do not necessarily represent current INDOT design practices.

2.2. DESCRIPTION OF THE INVESTIGATED BRIDGES

Due to the time constraints for this project, a complete analysis of all bridges in the INDOT's inventory was not feasible. For this reason, two typical and very different slab-on-steel girder bridges were selected from an extensive database of Indiana bridges. The main reason for selecting these two bridges was to investigate the behavior of representative bridges and to obtain results that could be projected to a range of steel bridges. The first bridge is located on US-52, and the second one is located on I-65. Of the significant differences between the I-65 Bridge and the US-52 Bridge, the major differences are the type of the girders and the width of the bridge. Furthermore, two different types of bracing systems were used in these two bridges. The diaphragms of the US-52 Bridge and the cross-frames of the I-65 Bridge had different levels of live load demand, and their interaction mechanisms with the main members were different. Cross-sections of both bridges are presented in Figure 2.1. Note that the same scale is used for each bridge.

Furthermore, to evaluate the effects of superloads on typical Indiana State Highway Bridges, one representative steel girder bridge (I-65 Bridge) was selected for instrumentation. The selected bridge for this project had to meet several criteria set forth by INDOT. First, the bridge needed to be located on a corridor which was known to be a common route for superloads. Second, it needed to be representative of contemporary bridge construction in Indiana. Third, to maximize the effects of superloads, the bridge needed to have a substantial span length of at least 80 ft long.

2.2.1. US-52 BRIDGE

The US-52 Bridge was built in 1970. Eastbound and Westbound traffic travel on two separate, identical structures. Each structure carries two 12 ft wide traffic lanes and a 10 ft wide shoulder. The US-52 Bridge crosses over North 9th Street Road and a set of CSX railroad tracks in Lafayette, Indiana. The first span of this bridge is reinforced concrete, and it is not connected to the next steel span (Figure 2.2). Only the steel spans were considered in this study. After the concrete span, the bridge has five-continuous spans, each with eight steel beam sections supporting a reinforced concrete deck.

The cross-section of the US-52 Bridge is presented in Figure 2.1. The span lengths are 92 ft, 108 ft, 94 ft, 101 ft and 92 ft. The width of the 7.5-in thick bridge deck is 43 ft (edge to edge). The US-52 Bridge does not have any skew. The bridge had concrete curbs and railings on both sides originally, but these were replaced by 33-in concrete parapets in 1989. The first steel span of this bridge was instrumented by Canna and Bowman (2002).

The steel beams of the U.S. 52 Bridge are 36-in deep rolled shapes. In the positive moment regions, the steel beams are made composite with the concrete deck. In the negative moment regions (over the interior supports), there are no shear studs; hence, the slab is not composite with the beams. Although the friction between the beams and slab may provide some unintended composite action, this was not considered in the analysis. In the positive moment regions, WF36X150 sections are used throughout the bridge, whereas WF36X230 and WF36X194 sections are used in the negative moment regions. Spacing of the interior girders is 6 ft (center to center) and the exterior girders are located at 5 ft from the interior girders. Girders are connected to each other with diaphragms whose spacing varies between 19 ft and 23 ft. WF18X45 diaphragms are directly welded to the beam webs as shown in Figure 2.3. The diaphragm geometric center is located near midheight of the beam. Intermittent fillet welds connect the webs of the diaphragms to

the beams, while continuous fillet welds were used to attach the top of the flanges to the beams. Some of the as-built plans of the US-52 Bridge are presented in Appendix A.

The first interior support, which is at the intersection of the first and the second steel spans, was specified as a fixed support. A photograph of this bearing is shown in Figure 2.4. The fixed support condition refers to a bearing type of support which permits rotation but does not allow translation in the horizontal plane due to a dowel pin in the middle of the bearing. All other supports were specified as expansion bearings. For the expansion supports, rocker bearings provide vertical support and allow longitudinal movement. A typical rocker bearing of the investigated bridge is shown in Figure 2.5.

The steel beams of the first and second spans were made from ASTM A572 Grade 50 steel with a minimum yield strength of 50 ksi and a tensile strength of 65 ksi. The diaphragms were made of ASTM A36 steel with a minimum yield strength of 36 ksi. The compressive strength of the concrete used in the reinforced concrete deck and the parapets was specified as 4 ksi. The deck had not been replaced when this study was conducted. Some deteriorated regions of the deck had been repaired with concrete patches when the curbs were removed and parapets were added to the bridge.

2.2.2. I-65 BRIDGE OVER RIDGE ROAD

The bridge that was selected is located in the Northwest corner of Indiana in Lake County between Gary and Hobart. The bridge carries south-bound traffic on Interstate 65 over Ridge Road (37th Avenue), and is located at RP 258+36 and log mile 257.67. The bridge number is I65-265-8203 SBL, and has an NBI number of 38750.

This bridge location was selected because it is on a known route for superloads. Heavy materials and equipment often enter Indiana via marine ports on along Lake Michigan. Due to clearance issues, the materials and equipment are then transported via

truck (superloads) South on Interstate 65, to US-30. Superloads then typically travel east on US-30 across Indiana, or to the desired location.

The selected bridge was designed for HS20-44 Loading in accordance with 1996 AASHTO Standard Specifications as well as for a Toll Road Loading in accordance with the Indiana Department of Transportation Bridge Manual. Construction of the bridge was completed in 2001. The Annual Average Daily Traffic (A.A.D.T) (2000) was 43,270 Vehicles per Day (V.P.D), of which 6,900 were trucks. The bridge carries three travel lanes, as well as an exit lane.

The I-65 Bridge over Ridge Road was built in 2001 and crosses over Ridge Road between the cities of Gary and Hobart in Northwest Indiana. It is a one span integral abutment bridge with a span length of 118.1 ft (Figure 2.6) and does not have any skew. The I-65 Bridge has ten 57 in deep plate girders which are composite with a 9-in thick reinforced concrete deck. Spacing of the girders is 11.2 ft, and the total bridge width is 106 ft-8 in which includes overhangs (Figure 2.7). Northbound and Southbound lanes are carried by two separate, identical bridges. Each structure carries three traffic lanes, one ramp lane and three shoulder lanes, one of which is between the exit lane and the traffic lanes. The other two shoulder lanes are at the edges of the bridge. The two structures are separated from each other by a construction joint. The width of the deck is 106.5 ft. The cross-section of the I-65 Bridge is presented in Figure 2.1.

The I-65 Bridge has cross-frames spaced at 21 ft for lateral bracing. The framing plan and a close-up the cross-frame are presented in Figures 2.8 and 2.9, respectively. Cross-frames, which brace the girders, are 4 in x 4 in x 3/8 in angles which are welded to gusset plates which are in turn bolted to vertical web stiffener plates. The bottoms of the vertical web stiffener plates are welded to small plates which are in turn bolted to the bottom flange. The top of the vertical web stiffener plates are welded directly to the top flange of the girders. A 1-1/2 in chamfer is cut out of the vertical web stiffener plates at

both the junction of the web and top flange and the web and bottom flange (Figure 2.9), creating a web gap.

All structural steel members were made of ASTM A572 Grade 50 Weathering steel. However, ASTM A36 steel was used for the cross-frame members. The specified compressive strength of the concrete used in the deck and the parapets was noted as 4 ksi in the as-built drawings. Continuous 46 in high parapets were used between the two abutments. A photograph of the I-65 Bridge over Ridge Road showing the plate girders, cross-frames and the integral end abutment is presented in Figure 2.10. Some relevant as-built drawings of the I-65 Bridge over Ridge Road are presented in Appendix A.

2.3. FINITE ELEMENT ANALYSIS

The structural analysis programs used by INDOT to process Superload Permits utilize a beam line analysis with lateral load distribution factors specified by AASHTO to determine girder stresses. This form of analysis is generally conservative for estimating girder stresses, but does not have the capability to analyze stresses in secondary load carrying elements including cross-frames, stiffener plates, and the concrete deck. For this reason, a more detailed analysis procedure is needed to identify any regions of high stress that may not be detected by a beam line analysis.

The US-52 and I-65 Bridges were analyzed using the finite element analysis programs SAP2000 and ANSYS. Three dimensional (3D) finite element models (FEM) were developed with respect to as-built drawings using SAP2000 and then converted to ANSYS input files without modifying the mesh density or the element connectivity. The objective of using ANSYS in addition to SAP2000 was to conduct nonlinear analysis. Nonlinear material models and geometric nonlinearity of shell elements were not available in version 7.44 of SAP2000. Nonlinear analysis was needed to evaluate the extent of possible damage due to superloads. Detailed finite element models included all structural and non-structural members which could contribute to the overall stiffness and

affect the structural behavior. Therefore, parapets, cross-frames or diaphragms were modeled in addition to the deck and girders. Four node shell elements with six degrees of freedom at each node and two node frame elements with six degrees of freedom at each end were used in the FEMs. The support conditions were modeled without any rotational restraint initially, but then refined according to field measurements for the I-65 Bridge over Ridge Road.

The reinforced concrete deck and the parapets were modeled with shell elements. The steel girders were made up of shell and frame elements. Shell elements were used for the web, whereas frame and shell elements were used for the flanges of the US-52 and I-65 Bridges, respectively. Shell elements were used to model the stiffener plates of the I-65 Bridge. Therefore, the flanges of the I-65 Bridge could not be modeled using beam elements. Shell elements were used to model the flanges of the I-65 Bridge to provide connection with the stiffener plates.

Since the connectivity of shell and frame elements was at nodes at their centroids, rigid link elements provided the proper distance between the centroid of the deck and the tops of the girders and the bottom of each parapet as shown in Figure 2.11. For the composite sections, the rigid link element enforced the constraint between the deck and girder, allowing proper simulation of the moment of inertia. In the noncomposite sections of the US-52 Bridge, the rigid link elements connected the nodes only in the vertical direction to provide continuous bearing between the deck and the girder.

Real dimensions of the members as specified in the as-built plans were used for mesh generation and determination of element dimensions (e.g. thickness and width) in the FEM. The only exceptions to this were the shell elements which were used for the parapets. Constant thickness shell elements were used for approximating the tapered shape of the parapet. The thicknesses of the shell elements used in the FEM for the parapets were determined with respect to the original area and moment of inertia of the parapets obtained using the real dimensions. Thicker shell elements were used at the

bottom and thinner shell elements were used at the middle and the top of the parapet as shown in Figure 2.12. The selected dimensions approximated the original area and moment of inertia values within a tolerable limit. The differences between the approximate dimension properties and the original values were less than 5%.

The elastic modulus and Poisson's ratio for concrete with a specified compressive strength of 4 ksi were taken as 3605 ksi and 0.2, respectively. The elastic modulus of concrete was calculated with respect to the ACI (2005) equation. For structural steel, the modulus of elasticity and Poisson's ratio were taken as 29,000 ksi and 0.3, respectively.

Truck loads applied to the investigated bridges were selected from an extensive permit truck database obtained from the Indiana Department of Transportation (INDOT) and Wisconsin Department of Transportation (WisDOT) by Wood (2004) as described in Chapter 1. Wheel loads were applied to the bridge deck as pressure load patches. The pressure load values were calculated based on axle weight and contact areas on the deck surface. The pressure load patches applied to the US-52 Bridge FEM for the 824-kip Maximum Superload Truck are shown in Figure 2.13. Since the overload trucks usually do not traverse a bridge side by side, the bridges were analyzed for a single lane loading.

The bending moment resisted by each girder was determined by using a special feature of the SAP2000 program (2001). With the "Group Joint Force Sum" feature, it was possible to obtain the resultant forces and moments in the nodes of the elements specified through the graphical user interface. All nodes and elements which were within the effective flange width of a composite beam were specified at a typical girder cross-section. The same procedure was repeated for all beams. For evaluation of the participation of parapets, nodes and shell elements included in the cross-section of a parapet were selected. Only the nodes above the deck were selected for the parapets, since the nodes in the deck had already been selected for the exterior beam. Hence, it can be said that the parapets were treated as beams above the deck for analysis and evaluation purposes.

2.3.1. FINITE ELEMENT ANALYSIS OF THE US-52 BRIDGE

The FEM of the US-52 Bridge was developed according to the aforementioned general modeling principles. Shell elements were used for the deck, parapets, beam web, diaphragm web and flanges. Frame elements were used for the beam flanges. The mesh density was increased for the critical regions of the bridge. For example, the FEM mesh was refined with smaller size elements at the beam web and diaphragm connection. A transition between fine and coarse mesh was provided by triangular shell elements. All five spans of the bridge were modeled as described. Since emphasis was given to the first and second spans in this study, a finer mesh than for the other spans was used. Also, diaphragms of the third, fourth and fifth spans were not included in the FEM, since the diaphragms of these spans did not have any effect on the response of the first span. This modeling simplification decreased the number of elements and run times without altering the results. There were more than 70,000 elements in the FEM of the US-52 Bridge.

Emphasis was given to possible high stress regions in the bridge. The portions of the web between the diaphragm and the top and bottom flanges of the beam are identified as the web gap region. The web gap region is known to be a possible problem region due to high stresses (Roddiss and Zhao, 2003) induced by out-of-plane bending. Hence, a finer mesh was used in this area. A close-up of the diaphragm and beam web connection is presented in Figure 2.14. Note that the intermittent and fillet welds used to connect the diaphragms to girder web were not considered in the FEM, instead, full connectivity was provided at all coincident nodes of the diaphragm and the beam web. Since the flanges and web of the diaphragm were rigidly connected to the beam in the FEM, this provided a model stiffer than the actual structure.

2.3.2. FINITE ELEMENT ANALYSIS OF THE I-65 BRIDGE OVER RIDGE ROAD

Two finite element models (FEM) were constructed to provide a more detailed estimate of actual behavior of the I-65 Bridge. Two separate steps were taken in modeling the selected bridge. First, a preliminary FEM was developed based solely on the as-built drawings of the bridge. This model was used to identify regions of high stress on the bridge. The results from the preliminary FEM provided insight into locations on the bridge that would be instrumented with strain gages. The second step in modeling the bridge utilized the results from the strain gages placed on the bridge to modify the FEM. The main difference between these two FEMs is the girder support conditions.

2.3.2.1. Development of Preliminary Finite Element Model

The preliminary finite element model developed for the I-65 Bridge was built using SAP2000 (FEM #1). In this model, shell elements (having six degrees of freedom at each node) were used to represent the steel girder flanges and webs, as well as the concrete deck, parapets, and web stiffener plates. The entire bridge model consisted of over 132,000 rectangular shell elements, each approximately 5 in. by 5.5 in. in size (Figure 2.15 and 2.16). An aspect ratio of 1:1 for shell elements was enforced wherever possible.

Although the deck was constructed using metal deck pans with grooves which may increase the thickness of concrete actually provided, the deck was modeled using 9 in. uniform thickness shells. This model does not take into account any cracking of the concrete deck, as it was modeled elastically. The deck acts compositely with the girder top flanges via shear studs, which were modeled using rigid beams or link elements (rigid link elements), to link the deck to the top flange.

The cross-frames were modeled with beam elements (having six degrees of freedom at each end). Due to the coarseness of the model (5 in. by 5.5 in. elements) the gusset plates connecting the web stiffener plates to the cross-frame members were not modeled. Cross-frame members were rigidly connected to the nodes of the web stiffener plates (Figure 2.17).

The integral end abutment supports were initially modeled as roller supports located at the bottom of the bottom flanges. This was accomplished by restraining deflections in the vertical and transverse directions with rigid springs, then partially restraining deflections in the longitudinal direction with a spring of negligible stiffness. The longitudinal spring was required to maintain stability. The initial assumption to model the bridge as rollers was to obtain the most conservative response possible via maximum possible longitudinal girder stresses and deflections. Adjustments to the end support conditions based on the measured bridge response will be discussed in a later section.

Pressure load patches, corresponding to the wheel weights of each superload truck, were assigned to the deck of the FEM. Each load patch was about 16.5 in. by 20 in. and represented tandem wheels. The longitudinal placement of each truck was determined by using a one-dimensional beam line analysis. Influence lines were developed, and trucks were positioned to produce maximum positive bending moments on a simply supported beam, as reported in Table 2.1. These same longitudinal locations for each truck group were then used in the three dimensional FEMs. The maximum positive bending moments reported in Table 2.1 were not always located directly at midspan, but were generally within 15 ft of the midspan.

Because of the large space which may exist between major axle groups (for Groups A, B, and C), the positions of the trucks that cause maximum positive bending moments on this 118 ft-1 in. span bridge is occasionally such that only one-half of the superload is on the bridge. Thus, only one-half of the GVW is on the bridge. This is

thought to be a major cause for the relatively low longitudinal stresses observed in the analyses.

Selected trucks were positioned with the entire load on the bridge concentrated at the ends of the span to maximize shear forces near the supports. Furthermore, the transverse truck location (lane of travel) was also varied. Each truck group was modeled at the exit lane (Figure 2.18), the far right travel lane (Figure 2.19), and the geometric center of the bridge (Figure 2.20). The exit lane was selected rather than the left lane since the exit lane is located closer to the parapet.

As will be demonstrated later in this study, this full bridge model did a good job of predicting the overall behavior of the bridge. However, due to the shell element's mesh density (5 in. by 5.5 in. elements) this model had certain limitations. Therefore a second model (FEM#2) was constructed by Akinci (2006) using FEA software called ANSYS. This model incorporated all of the same modeling assumptions described for the first FEM, but used a smaller mesh density (approximately 1 in. by 1.5 in.) to model the web gap area and web stiffener plates more accurately. The FEM#2 was developed in a manner similar to that of the US-52 Bridge and the aforementioned FEM of the I-65 Bridge. However, the detailed FEM of the I-65 Bridge required more elements. The deck, girder web and flanges, parapets and the stiffener plates were all modeled with shell elements. The cross-frame angles were modeled with frame elements with six degrees of freedom at each node. The cross-frame connections were simplified for modeling purposes. The gusset plates connecting the angles to the stiffener plates and to each other were not included in the model.

The stiffener plates were welded to the top flange and girder web but bolted to the bottom flange (Figure 2.9). The web gap of this bridge was 1.6-in deep at the chamfer in the stiffener plate at the intersection of flange and web. In the development of the detailed finite element model of the I-65 Bridge over Ridge Road, the dimension of the web gap region (1.6 in) was one of the major controlling parameters. A fine mesh was essential at

the location where the stiffener plates were connected to the girders, since this was a stress concentration location. A model which was completely made up of small elements (1.6 in by 1.6 in) could result in longer run times and some other computational problems. A practical approach for unifying accuracy and computational efficiency was found to be utilizing the symmetry principle.

For a model with more than 120,000 elements and nodes, restrictions of the available academic version of ANSYS on the number of elements were exceeded. Therefore, symmetry was utilized in addition to a change in the mesh density along the span. The symmetry principle enabled modeling of one-half of the bridge extending from the abutment to the midspan. Accordingly, the appropriate symmetry boundary conditions were assigned to the nodes at the midspan of the bridge and the truck loads were modified somewhat to obtain a symmetric loading. An isometric view of the symmetric FEM of the I-65 Bridge over Ridge Road (FEM#2) is presented in Figure 2.21.

For the symmetric FEM of the I-65 Bridge over Ridge Road, a finer mesh was used near the midspan and a coarser mesh was used near the support. The coarse mesh region includes shell elements whose dimensions are as large as 13 in by 13 in. Dimensions of the shell elements were 1.6 in by 1.6 in at the fine mesh region. The fine mesh region covers the middle portion of the bridge between 40 ft and 59 ft (mid-span); between the end and 36 ft, the coarse mesh was used. Close-up views of the FEM#2 are presented in Figures 2.22 and 2.23, which show the mesh transition, deck, girders, stiffeners and cross-frames. In addition to having different element sizes, the fine and coarse mesh regions differ from each other with respect to the cross-frame and girder connections. In the coarse mesh region, the cross-frame angle beam elements were connected directly to the intersection point of flange and web. The stiffener plates and web gaps were modeled in the fine mesh region and the angles were connected to the stiffener plates directly as shown in Figures 2.24 and 2.25. All beam elements used to model to cross-frame angles were rigidly connected to the stiffener plate or the

intersection point of flange and web. It should also be noted that the angle attachment plates were not included in the model. The 2% grade of the deck was also included in the FEM#2 (Figure 2.11). Since the girders were spaced 11.2 ft apart, the 2% grade led to an elevation difference of 2.7 in between the girders. The girders were stepped on the I-65 Bridge.

The most critical loading for the I-65 Bridge was obtained when the heavy axle groups of the superload trucks were placed at the midspan. Since the superload trucks were longer than the span length of 118 ft, it was not possible to position all of the axles on the bridge. Half of the 824-kip Superload Truck's trailer, required for symmetry, was applied to the symmetric FEM of the bridge as pressure loads (Figure 2.21). The trucks were placed at several transverse locations, such as the center of the bridge, the exit lane, and the shoulder at about 2 ft from the parapet. The load patches were calculated based on the approximate wheel dimensions, axle spacing and weights, and dimensions of the shells used for the deck. The bending moments resisted by the girders and the parapets, displacements of the nodes at the bottom of the girders and cross-frame, and stiffener plate stresses were evaluated.

Support conditions due to the integral end abutment were idealized as roller supports for the preliminary analysis using FEM#2, but then calibrated based on load test results. Therefore, no rotational restraint or constraint action due to the abutment wall was considered in the preliminary FEM #2. This assumption was conservative, but consistent with the traditional approach.

2.4. INSTRUMENTATION

Instrumentation of steel bridges has been used as a powerful tool to determine the actual behavior and nominal stresses in critical locations. Limitations of analytical approaches with regards to accurate prediction of actual stresses in high stress regions

and variations in the anticipated behavior compared to the expected behavior of bridges increase the importance of field measurements. Therefore, the I-65 Bridge over Ridge Road was instrumented for a load test in addition to long term monitoring. The US-52 Bridge was instrumented by Canna and Bowman (2002) in an effort to investigate the fatigue behavior of beam-diaphragm connections with intermittent fillet welds. Results of the load test conducted by Canna and Bowman (2002) were used to evaluate the performance of the FEM of the US-52 Bridge. Instrumentation plans of both bridges are summarized below.

2.4.1. INSTRUMENTATION AND LOAD TEST OF THE US-52 BRIDGE

The US-52 Bridge Westbound structure was instrumented by Canna and Bowman (2002) for field evaluation of beam-diaphragm connections. A load test was conducted, and strains due to regular traffic were monitored. Both static and dynamic measurements were recorded. Resistance type strain gages were used for the instrumentation of Beams 3 and 4, and Diaphragms 2, 3 and 4 (Figure 2.26). Furthermore, deflections of Beams 3 and 4 were measured during the load test. A tandem axle dump truck weighing 52.2 kips was used for the load test. The axle weights of the loaded dump truck were 13.5 kips, 19.3 kips and 19.4 kips. The space between the 13.5 kips and 19.3 kips axles was 17.5 ft, whereas the 19.3 kips and 19.4 kips axles were spaced at 4.5 ft. Details of the instrumentation program and the load test are presented by Canna and Bowman (2002).

The loaded dump truck was positioned at predetermined locations near the middle of the first span for the load test. Two positions of the test truck were considered for comparison purposes in this study. In Canna and Bowman's study (2002), these loading positions are called LC #2A and LC #3A. "A" denotes the transverse position of the truck, and the numbers "2" and "3" indicate the longitudinal position of the test truck. For both cases, the test truck was located at the center of the right travel lane. For LC #2A, the middle axle of the dump truck was located directly above the diaphragm at the middle

of the bridge (Figure 2.26). For LC #3A, the diaphragm at the middle of the bridge corresponded to a point halfway between the tandem axles. These two longitudinal positions were approximately 2 feet apart.

2.4.2. INSTRUMENTATION AND LOAD TEST OF THE I-65 BRIDGE OVER RIDGE ROAD

2.4.2.1. Instrumentation

Based on the results of the preliminary FEM, an instrumentation plan for the I-65 Bridge over 37th Ave. (Ridge Road) was developed. Strain gages, thermocouples, survey equipment, and inclinometers were used to capture various live load responses. The data loggers used for this project allowed a maximum of 80 channels to record strains, temperatures, and rotations. This instrumentation plan utilized 75 channels for strain gages, 3 channels for thermocouples and 2 channels for inclinometers (total of 80 channels). Deflections were measured using survey equipment; this data was not recorded using the data loggers.

Four different cross sections and one abutment were selected for instrumentation. Each of these cross sections was selected to efficiently capture a different effect. The general bridge cross section is shown and a plan view of the bridge showing the longitudinal positions of the four different cross sections that were instrumented are shown in Figure 2.8. Section C, which is at the midspan of the bridge, was selected since it represented a line of symmetry for the bridge, and because it was expected to be the location of maximum bending moment. Section A, which is 6 in. from the face of the abutment, and Section D, which is 54 in. from the face of the abutment, were selected to measure the end restraints provided by the abutment. Section B was selected as the most critical line of cross-frame members which was also closest to the midspan of the bridge.

The measurements collected from this instrumentation plan were used to infer the following: the lateral distribution of live loads; the amount of lateral live load distribution provided by the cross-frames; the stress concentration at the weld connecting the transverse web stiffeners to the top flanges of the girders; the stress concentration at the bottom web gaps; the amount of live load rotational restraint provided by the integral abutments; the maximum girder deflection; and the effects of temperature changes.

2.4.2.2. Lateral Distribution of Live Loads

Finite element analysis results suggest that the five or six girders closest to the superload carry almost the entire load. To confirm this load distribution, cross section C, which is located at midspan of the bridge and expected to be very near to the location of maximum positive moment, was instrumented. It was assumed that any superload trucks crossing the bridge would travel either in the right or center lane. For this reason, it was expected that girders 6, 7, and 8 would be the most heavily loaded girders and, therefore, they were instrumented more extensively.

The locations of strain gages on the ten girders at Section C are shown in Figures 2.27 and 2.28. Two strain gages were placed on the top flanges, web, and bottom flanges of girders 6, 7, and 8. This strain gage configuration was selected to provide the stress distribution through the cross section of the girder, and thus allow the neutral axis to be located. This configuration also strengthened the probability of obtaining accurate readings at both flanges and the web. In the unfortunate event that one of the gages became defective, the redundancy of having two gages at each location still allowed strains to be measured. Girders 1 through 5, 9, and 10 were instrumented as shown in Figure 2.28. Only three strain gages were used at each of these girders' cross sections. Three gages were adequate for capturing the transverse load distribution and inferring the neutral axis location of each girder. The location of the neutral axis and the load

distribution could be captured with a combination of any two gages shown, but the third gage was provided for redundancy. Photographs of selected gages are shown in Figures 2.29 and 2.30.

In addition to the girders at Section C, the parapet on the East side of the bridge was also instrumented (Figure 2.31). The East side parapet was selected because it is closer to the travel lanes than the West side parapet, which is closer to the exit lane. The parapet was instrumented to quantify its effect on the live load distribution. Four concrete surface gages were used at the locations shown along the face of the parapet to capture the strain distribution in the parapet. A photograph of the parapet gages is shown in Figure 2.32.

2.4.2.3. Cross-Frame Load Distribution

Finite element analysis of the bridge also suggests that the three cross-frames closest to the location of the superload contribute to the lateral load distribution, while the remainder of the cross-frames are essentially unaffected. Furthermore, the FEA suggests that the cross-frames closest to the midspan of the bridge (Section B) are more heavily stressed than the cross-frames closer to the abutments. Because superloads crossing the bridge are expected to be traveling in the right or center lane, the cross-frames under these two lanes were selected for instrumentation. The cross-frames between girders 6 and 7 and between girders 7 and 8 were instrumented with strain gages as shown in Figure 2.33. Finite element analysis also suggests that the bottom horizontal angle is the most highly stressed member of the cross-frame system. For this reason, only the horizontal angle was intended to be instrumented. Two strain gages, one on each leg of the angle, were placed so as to capture the axial strains in the angle, and to identify any strains caused by bending of the angle. Bending stresses were expected to be very small. During the installation of strain gages, two of the stiffener plate gage locations were found to be inaccessible, and therefore, the gages were relocated to two cross-frame

diagonals. The two diagonals connected to the top of girder 7 were instrumented with one strain gage each. Photographs of the cross-frame gages are shown in Figure 2.34 to 2.36.

2.4.2.4. Transverse Web Stiffener Stress Concentrations

Finite element analyses of the bridge also indicate stress concentrations at the outer edges of the transverse web stiffeners, where the web stiffeners are welded to the top flanges. This stress concentration is expected to be the most severe at Section B, where cross-frames are connected to the girders. Due to the limitations on the number of data channels available, only one girder was instrumented. Girder number 7 was selected because it was expected to be the most heavily loaded girder (since it is between the right lane and the center lane). Stresses in the vertical direction are thought to be the most severe component of this stress concentration. For this reason, strain gages were positioned vertically at the top outside edge of the web stiffeners. These vertical gages were placed on one side of the web stiffeners. It was desirable to place vertical gages on both sides of the stiffeners, but the presence of the gusset plates limited accessibility on one side of the stiffeners. Because stress concentrations radiate out from their origin in all directions, a strain gage positioned horizontally was also placed adjacent to the vertical strain gage on one face of each web stiffener. The combination of vertical and horizontal gages at these locations provided an indication of the magnitude and direction of each stress concentration. The locations of these gages are shown in Figure 2.37, as well as in photographs found in Figures 2.38 to 2.40.

2.4.2.5. Lower Web Gap Stress Concentration

Although the finite element analyses conducted for this bridge did not indicate that the lower web gap region was a critical location, it is widely known that this region is often susceptible to cracking caused by out of plane bending of the web. Because the

bottom of the web stiffener plate is welded to a small plate which is bolted to the bottom flange, the stresses at the lower web gap were expected to be low. Strain gages were however placed in this region for the purpose of confirming finite element results. One vertical and one horizontal strain gage was placed directly at the lower web gap region on each side of the girder web. All gages were placed for redundancy and so that the directionality of the stresses could be determined. Also, due to limitations on the number of data channels available, and the belief that this location was not critical, only one girder was instrumented. Girder number 7 at Section B was selected since it was expected to be the most heavily loaded girder. The locations of these gages and the location of the web stiffener plate gages can be seen in Figure 2.41 and 2.42. Figures 2.43 and 2.44 show photographs of the web gap gages.

2.4.2.6. Integral Abutment Restraint

The amount of live load rotational restraint provided by integral abutments is not fully understood, since little experimental research has been published on the topic. Finite element models used to analyze this bridge were conservatively modeled with zero rotational restraint at the abutments. To quantify the live load rotational restraint provided by the integral abutments, certain girders near the abutment were instrumented. Since girder 7 is expected to be the most heavily loaded girder, the instrumentation was centered around that girder. At Section A (6 in. from the face of the abutment), two strain gages were placed on each of the flanges and on the web of girder 7, as seen in Figure 2.45. Two gages were used at each location to provide redundancy and ensure the accuracy of the readings. Girders 6 and 8 were instrumented as shown in Figure 2.46. One gage was placed at each of the flanges and at the web. This configuration allowed the strain distribution through the cross section to be determined even if one of the gages was lost. Furthermore, by instrumenting girders 6 and 8 as well as girder 7 at Section A, the lateral load distribution could be inferred, and compared with the lateral load distribution at the center of the span (Section C). By instrumenting these three girders at

Section A, it was also possible to identify the integral abutment's contribution to lateral load distribution.

In order to fully capture the longitudinal moment distribution along the bridge, girder 7 was also instrumented at Section D (54 in. from the face of the abutment). Strain gages were placed on Girder 7 as shown in Figure 2.47. Once again, for purposes of redundancy, ensuring accurate readings, and capturing strain distributions through the cross section, two gages were placed on both of the flanges as well as the web. Since the longitudinal bending moments at Sections C, D, and A could be determined for Girder 7, the restraining moment at the integral abutment could be inferred. Figures 2.48 and 2.49 are photographs of the gages at Section A and D.

In addition to using strain gages to determine the rotational restraint of the integral abutment, inclinometers were also used to measure the angle of rotation of the face of the abutment wall. One inclinometer was placed on the face of the abutment wall next to girder number 7, to measure the live load rotation of the abutment. The other inclinometer was placed on the web of girder number 7, as close to the wall as possible (6 in.), to measure the live load rotation of the girder. Based on the rotation of the abutment, and the differential rotation between the girder and the abutment, the restraint of the abutment could be inferred. The inclinometers were placed adjacent to each other on the abutment and the girder as shown in Figures 2.50 and 2.51.

2.4.2.7. Girder Deflection

Girder deflections are important measurements in that they provide another check for girder stresses and degree of rotational restraint provided by the abutments. It was decided to obtain deflection measurements during the load test of the bridge, and to forgo measurements of deflection due to live traffic. The procedure used will be discussed further in a later section.

2.4.2.8. Thermal Effects

Temperature was monitored to distinguish between live load strains and strains caused by thermal effects. Since strain gages were not able to differentiate the causes of strains, it was necessary to identify and quantify the strains caused by environmental sources (change in temperature), so that the live load strains could be isolated. Three thermocouples were used to measure air temperatures. One thermocouple was located at midspan on the top of the deck, near the East parapet. The other two thermocouples were located under the bridge on girder number 7 at the North abutment and at midspan.

2.4.2.9. Instrumentation Equipment and Data Acquisition System

Strain, angular, and thermal data were collected using four Campbell Scientific CR5000 data acquisition units. Strains were measured using precision strain gages from Vishay Micro-Measurements. Gage types CEA-06-250UN-350, CEA-06-125UN-350, and N2A-06-20CBW-350 were used. Schaevitz LSOC inclinometers were used to measure angle rotations at one of the abutments. Thermal data was measured using Type T, 24 AWG Teflon Neoflon FEP insulated thermocouple wire (FF-T-24). Deflection measurements during the load test were taken with a DL-101 digital level from Topcon, which has an accuracy of better than 2 mm.

2.4.2.10. Data Sampling and Recording

A program for data sampling and recording was written using CRBasic software created by Campbell Scientific for CR5000 and CR9000 data acquisition systems. Calculations for the unloaded bridge suggest that the fundamental period of the bridge is

approximately 0.3 sec, corresponding to a natural frequency of approximately 3.33 Hz. A sampling frequency of 50 Hz was selected. Sampling of all data logger channels on all four data loggers was initiated by a trigger. A trigger was set based on the two strain gages on the bottom flange of girder 7 at Section C (midspan). A moving average of 1000 scans was kept for each of the two trigger channels. When strains on either of these two channels exceeded $30\mu\epsilon$ above the 1000 scan moving average, all channels on all data loggers began recording. The $30\mu\epsilon$ trigger was later changed to $60\mu\epsilon$ to capture only the heaviest truck traffic. Based upon the expected speed of trucks traveling over the bridge and the time needed to record the full event, a pre-trigger of two and one-half seconds (125 scans) and a post-trigger of three and one-half seconds (175 scans) were used in storing data. A sample of the CRBasic program written for recording was presented by Wood (2004).

2.4.2.11. Description of Load Test

In order to validate the finite element model (FEM) used to simulate the various superload truck loadings with the actual response of the bridge, a static load test of the bridge was conducted. The load test was conducted on Saturday, June 26, 2004. Traffic control began at 4:00 a.m. and was in place by approximately 5:00 a.m. All three lanes of interstate traffic were merged into a single lane, and routed across the bridge via the exit lane and shoulder while the load test was being conducted. From 5:00 a.m. until approximately 6:00 a.m. the top surface of the bridge deck was marked and prepared for measurements. Measurements for the load test began at 6:00 a.m. and were concluded at approximately 8:30 a.m. Traffic control was removed and traffic resumed normal flow by 9:00 a.m.

The load test included parking four-tandem dump trucks, loaded with sand, which were provided by the Indiana Department of Transportation. The dump truck dimensions and axle weights can be found in Figure 2.52. The axle weights of each loaded dump

truck were measured at a local weight station the afternoon of Friday, June 25, 2004. The gross vehicle weights of each dump truck ranged from 41,820 lbs to 44,940 lbs. The dump trucks were stored in an enclosed garage over night. Since all four trucks were the same make and model, the axle spacings were all identical. These dimensions were hand measured on Friday, June 25, 2004.

Six different Load Cases were devised to simulate potential superload axle configurations and to generate the maximum stresses at the instrumented locations. Five of these Load Cases are shown in Figures 2.53 to 2.61. Load Cases 1-5 were static loadings where the trucks were parked in the positions shown while measurements were made. Load Case 6 was a dynamic loading where the trucks drove in unison over the bridge as close to the shown configuration as possible (Figure 2.62). Load Case 1 was devised to maximize longitudinal stresses in girder 7 at Section C (midspan). Load Case 2 was selected to compare with Load Case 1 and to quantify the effects of parapets on load sharing. Load Cases 3 and 4 were selected to simulate longer-single lane superload trucks similar to Groups B or C driving in the right or center lane. Load Cases 5 and 6 were devised to both maximize longitudinal stresses and quantify the impact of the trucks entering the bridge. Load Case 6 was repeated two times due to the difficulty of maintaining uniform truck spacing at high speeds.

The measurements taken for each static Load Case included strain measurements of all strain gages placed on the bridge, angle measurements from the two inclinometers, and vertical deflection measurements using a DL-101 digital level. The strain and angle measurements were manually triggered to record once the trucks were parked in their respective location for each Load Case. The data acquisition system was set up to scan each strain gage and inclinometer simultaneously at 50 Hz, and to record 350 records. The measurements were triggered to record three times (in approximately one minute intervals) for each Load Case (totaling 1050 data points per channel per Load Case).

Deflection measurements using the digital level were also taken on top of the deck while the strain and angle measurements were being recorded (Figures 2.63 and 2.64). Deflections were measured at midspan of the bridge at girder lines 6-10 and half-way between each girder, as well as the top of the parapet. Figure 2.65 shows the location of deflection measurements. Stainless-steel acorn nuts (Figure 2.66) were attached with epoxy to the bridge deck at each measurement location to ensure that each measurement was taken at the exact same location for each Load Case. Although it was desirable to take measurements at each of the girder lines (girders 6-10) and mid-way between each girder, visibility and/or access to these locations were occasionally blocked by the dump trucks parked on the bridge. In these cases, the deflection measurements were conceded.

Traffic in the exit lane and shoulder of the bridge was stopped for approximately 30 seconds so that all strain gages and inclinometers could be “zeroed”. Traffic was completely stopped for “zeroing” in order to eliminate noise in the data caused by the vibration of the bridge. The “zero” deflection measurements were also taken while the dump trucks were parked off of the bridge, but while traffic was allowed to pass in the exit lane and shoulder.

2.5. EVALUATION OF LOAD TEST DATA AND ANALYSIS RESULTS

Available load test data are compared with the analysis results in this section. In the FEMs, the test trucks were positioned according to their actual locations during the load test. Comparison of the load test and analysis results demonstrates the accuracy and reliability of the FEMs.

2.5.1. COMPARISON OF LOAD TEST AND ANALYSIS RESULTS FOR THE US-52 BRIDGE

There were only a limited number of measurements available for the US-52 Bridge, since the main objective of instrumentation was to measure strains in the diaphragms. The beam and diaphragm stresses calculated from the measured strains are compared with the FEA values, as were the beam deflections. The elastic modulus of steel used for the strain stress conversion was 29,000 ksi. Labeling of the beam and diaphragms and the transverse position of the test truck for load cases LC #2A and LC #3A are presented in Figure 2.26.

In Table 2.2, the measured midspan deflections are compared with the results obtained from the FEA for the two load cases. These deflections were measured at the bottom of Beam 3. The measured and calculated deflections presented in Table 2.2 agree with each other within a tolerable limit. The difference between the FEA deflection and the test data was less than 10% for LC #2A. The FEA predicted a slightly larger deflection. This can be attributed to unintended composite action over the piers due to the friction between the beams and the deck. The unintended composite action was not considered in the FEM. It could also be due to variations in the modulus of elasticity of concrete.

The beam and diaphragm stresses calculated from the measured strains were also compared with the analytical results. The longitudinal stresses at the bottoms of the top and bottom flanges of Beams 3 and 4 are presented in Tables 2.3 and 2.4. The strain measurements were recorded at the middle of the first span. This was also a diaphragm location. Positive stresses indicate tension and negative stresses indicate compression in the tables. The bottom flange stress of Beam 3 could not be obtained due to malfunctioning of the strain gauge on the bottom of this beam.

The error with respect to measured stress for the top flange was about 40% for both beams. However, the differences between the analytical and the experimental results

were as small as 0.14 ksi and 0.06 ksi. The top flange is close to the neutral axis of this bridge; therefore, smaller strains were recorded. Also, modeling idealizations can influence the neutral axis location attained by the FEM. This difference in predicted and measured neutral axis may not affect the overall behavior and stresses at locations far enough from the neutral axis, but near the neutral axis, it can make a significant difference. Also, the accuracy of the strain measurements becomes a critical issue for smaller strains such as the ones obtained for the top flanges, which were on the order of five to ten micro strains. The bottom flange stress of Beam 4 calculated from the FEM was almost identical with the stress calculated from the measured strain, with a difference of less than 2%.

Strains in the diaphragm located at the middle of the first span were also measured. Strain gauges were attached to the top and bottom flanges of the diaphragms at the middle along the length between the beams. The measured and computed diaphragm stresses along the diaphragm are presented in Tables 2.5, 2.6 and 2.7 for Diaphragms 2, 3 and 4 (Figure 2.26), respectively. Stresses calculated from measured strains are called “measured stresses.”

The complex stress state of the diaphragms and the modeling idealizations of the diaphragms (i.e. the actual connection behavior) are believed to be the major factors creating the errors seen in Tables 2.5 to 2.7. The mesh density of the model along the diaphragms is another factor which could lead to the observed discrepancy. A finer mesh could yield more accurate stress values in this region. However, the results obtained from the FEM can be considered to be satisfactory considering that the general behavior was represented well.

2.5.2. EVALUATION OF LOAD TEST DATA FOR THE I-65 BRIDGE OVER RIDGE ROAD

As mentioned earlier, the entire duration for measurements in the load test for the I-65 Bridge over Ridge Road was approximately 2.5 hr (6:00-8:30 a.m.). During this time, temperatures of the both the girders and deck were monitored by means of thermocouples. It was found that the girders and deck temperatures did not fluctuate by more than 3°F from start to finish of the load test and, therefore, no significant drift in strain gage readings due to temperature were observed.

2.5.2.1. Girder Deflections

As described earlier, girder deflections were obtained using digital display surveying equipment. Measurements were taken to obtain a transverse deflected shape of the bridge under various loadings, and to provide an added check for other experimental data. The measured deflections for each Load Case can be seen in Figure 2.67. Measurements were not always continuous across the bridge since the trucks on the bridge occasionally were located directly over the markers or blocked the line of sight. The obtained measurements are fairly consistent, with exception for Load Case 2, during which difficulties were experienced with the surveying equipment. For Load Case 2, measurements were made by manual readings of the surveying rod, and therefore were not as accurate as the remainder of the readings. The problems were resolved for the readings of the remaining Load Cases. Chapter 7 compares this deflection data with FEA deflections, where a better representation of the transverse deflected shape of the entire cross section can be observed.

2.5.2.2. Individual Girder Stress/Strain Distribution

Strain gages were placed on all girders to determine the strain distribution through the depth of each individual girder at given cross sections. These gage locations allow

maximum tensile and compressive stresses to be determined, as well as the location of the neutral axis. Plots were generated at selected girder cross sections for all Load Cases. Figures 2.68 to 2.72 show the stress distribution over the depth of girder 7 at midspan (Section C) for all Load Cases. The maximum tensile stress in the bottom flange of girder 7 was found to be 2.5 ksi for Load Case 1. The X's represent stress values inferred from the measured strains. A fairly linear stress distribution can be observed for each of the Load Cases. Small deviations from the linear distribution, such as the bottom web (WB-E) gage, are likely a result of misalignment of the gage.

Figures 2.73 to 2.77 display the stress distribution of girder 7 at Section A, a position 6 in. from the face of the abutment. It is evident from these plots that there is substantial negative moment at the abutments. Compressive bottom flange stresses at this region of girder 7 are approximately equal in magnitude to the tensile stresses at midspan for Load Cases 1 and 2, suggesting a nearly fixed end condition. Load Cases 3-5 were not symmetric loadings, so a similar trend would not be as prevalent for these loadings. Nonetheless, it is apparent that there is a nearly fully-fixed-end support condition at the integral abutments.

Figures 2.78 to 2.82 show the stress distribution over the depth of girder 7 at Section D (54 in. from the face of the abutment). The stress distribution at this location further supports the fixed-end support condition claim. Girders 6 and 8 demonstrate the same trend of having compressive longitudinal stresses near the abutment and equivalent tensile longitudinal stresses at midspan.

2.5.2.3. Lateral Distribution of Load

The lateral distribution of load can be evaluated by looking at the collective girder stress distribution. Figures 2.83 to 2.87 show the lateral stress distribution as a function of top flange stresses and bottom flange stresses. The top flange gage reading from girder 5

and the bottom flange gage readings from girders 2 and 9 were not included in these plots due to their inconsistency with deflection data and the distribution across each respective girder. Girder distribution factors (GDFs) were computed for each of the Load Cases by dividing the maximum girder stress by the sum of the girder stresses as shown (Equation 2.1). The GDFs based on top flange and bottom flanges stresses are shown in Table 2.8.

$$GDF = \frac{\sigma_{\max}}{\sum_{i=1}^{10} \sigma_i} \quad (\text{Eqn. 2.1})$$

Girder Distribution Factors were found to be significantly lower when computed based on top flange stresses. This is thought to be a function of the very low stresses measured at the top flange. Consequently, GDFs computed based on the bottom flange stresses are thought to be a better representation of the actual load distribution. Despite Load Cases 1, 2, and 5 having multiple lane loadings, the GDFs remain fairly consistent with the single lane loadings. GDFs were found to range from 0.29 to 0.38.

In comparison with computed design GDFs, it is shown that the actual bridge has a more uniform load distribution than predicted. Equation 2.2 shows the AASHTO LRFD lateral load distribution factor for bending moment in interior girders of a concrete slab on steel girder bridge:

$$g = 0.06 + \left(\frac{S}{14}\right)^{0.4} \left(\frac{S}{L}\right)^{0.3} \left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1} \quad (\text{Eqn. 2.2})$$

g = girder distribution factor (single lane)

S = girder spacing (ft)

L = span length of beam (ft)

t_s = thickness of deck (in)

K_g = longitudinal stiffness parameter

$$= n(I + Ae_g^2)$$

n = modular ratio between beam and deck material

I = moment of inertia of beam (in^4)

A = cross-sectional area of beam (in^2)

e_g = distance between center of gravity (COG) of beam
and COG of deck (in)

Equation 2.2 yields a girder distribution factor of 0.53 for a single lane and 0.80 for multiple lanes for this particular bridge. This bridge was designed based on the AASHTO Standard Specification, which indicates a *wheel* distribution factor of $S/5.5$, or 2.01 for this bridge. This *wheel* distribution used by the Standard Specification would equate to a 1.0 *girder* distribution factor based on axle weights. Table 2.8 shows that both of these formulas underestimate the lateral load distribution of the actual bridge. Experimentally evaluated girder distribution factors are on the order of 0.29 to 0.38.

It was also observed from Section A, that the lateral load distribution followed the same trend as at Section C. Although only three girders were instrumented at this section, a similar load distribution was observed based on bottom flange stresses.

2.5.2.4. Parapet Effects

It was anticipated that from the four concrete surface gages on the face of the parapet, the parapets' contribution to load distribution could be quantified. Unfortunately,

during the instrumentation process, one of the concrete surface gages did not work properly, therefore only three gages were left to infer the stress distribution across the parapet. The strains measured on the parapet will be further discussed in Chapter 4.

2.5.2.5. Integral Abutment Effects

One of the objectives in analyzing this particular bridge was to determine the restraint provided by integral abutments. Strain gages were placed at three different longitudinal positions to extrapolate a longitudinal girder stress distribution. Furthermore, inclinometers were used to obtain live load rotations of both one girder, and the abutment wall.

Based on the individual girder stress distributions for Girder 7 at Sections A, D, and C, a longitudinal stress distribution along the member length can be extrapolated for Load Cases 1 and 2. Figures 2.88 and 2.89 show the longitudinal stress distribution along girder 7 for Load Case 1 and 2, respectively. The two measurements (top and bottom flange) at the left end of the span represent top and bottom flange stresses measured from Section A, 6 in. from the face of the abutment. The next two measurements represent stresses at Section D, 4.5 ft (54 in.) from the face of the abutment. The measurements at midspan (59 ft) represent stresses at Section C. Since Load Case 1 and 2 were very close to being symmetric loadings, the stress distribution is also assumed to be symmetric about the midspan (Section C). Therefore, the measurements shown at span locations of 113.5 ft and 117.5 ft mirror the measurements at Section D and A, respectively.

From these two figures, it can be inferred that a nearly fully fixed support condition exists at the integral abutments. This inference is based on equal moments at both the center of the span and at the supports for a beam line analysis with concentrated loads centered about the midspan of the bridge. Based on measured bottom flange stresses from girder 7, it is estimated that the integral abutments provide between 90-

100% fixity. Similar data obtained from girders 6 and 8 further support this trend, and it suggests that the integral abutments provide between 80-100% fixity.

2.5.2.6. Abutment Rotations

Angular measurements from a girder and an abutment wall also provided some insight into the effects of the integral abutments. As noted before, inclinometers were placed on girder 7, at a position 6 in. from the bottom of the bottom flange at Section A (6 in from the face of the abutment), as well as on the abutment at a position 6 in. from the bottom of the top flange and 6 in. from the web of girder 7. Both inclinometers were oriented to measure rotations in the longitudinal direction. The measurements from both inclinometers are reported in Table 2.9. The measurements from these two inclinometers had very large noise-to-measurement ratios, and therefore are not believed to be very reliable. Nonetheless, from this table, it is evident that both the abutment wall and the girders are rotating under live loads. Both the rotations as well as the relative rotation are extremely small, which further supports the condition of nearly full fixity of the integral abutments. Rotations based on a simply supported (0% fixity) beam line analysis were calculated to be approximately 0.002 radians based on Load Case 1 loading scenario, which is 10 times as large as the measured rotations in Table 2.9.

2.5.2.7. Cross-frames

Four cross-frame members were instrumented with strain gages, to identify their participation in the lateral load distribution, and to check for potential overstressing. Figure 2.90 shows the measured stresses for the instrumented cross-frame members for Load Cases 1-6.

From the measured data, a couple of conclusions can be made. First, it is apparent that horizontal members are subjected to axial forces, bending moments, and possibly torsion. This combination of loadings is a result of both the differential deflections in adjacent girders and the manner in which angles are connected to the girders. Secondly, although measured stress values are well below the design yield strength, they are similar in magnitude to measured longitudinal girder stresses, signifying a significant contribution to the lateral load distribution. It can also be concluded that the cross-frame members positioned directly under the applied wheel loads are the most highly stressed.

2.5.2.8. Web Stiffener Plates and Bottom Web Gap Region

The preliminary FEM indicated a possible stress concentration at the weld between the girder top flanges and the web stiffener plate. The measured data from this location confirms this critical location. Figure 2.91 provides both vertical and horizontal stresses at a location of less than 1 in. from the weld. All measurements are already corrected for transverse sensitivity effects. One of the horizontal gages (Horizontal 1) was found to be nonfunctioning, and therefore no measurements could be obtained from it. As evident from Figure 2.91, the vertical component of the concentration is the most critical. Furthermore, it can be observed that when one side of the web is in compression (-), the other side is in tension (+). A maximum tensile stress of 4.8 ksi was measured, while an even higher stress is expected at the location of the weld, since that is the origin of the stress concentration. With the strain gage configuration used, it was not possible to determine how quickly the stress concentration diminished away from its origin. A fatigue analysis of this region will be discussed further in a later section.

The web gap region was also instrumented with strain gages in two orthogonal directions. The measurements obtained from these strain gages can be found in Figure 2.92. Again, all measurements are already corrected for transverse sensitivity effects. The measured data seems to suggest that there may be a small stress concentration in the

longitudinal (horizontal) direction as a result of the termination of the weld between the girder web and the web stiffener plate. The horizontal gages measure longitudinal stresses and, therefore, the measurements appear to be more a result of longitudinal bending of the girder, rather than a significant stress concentration. By extrapolating longitudinal stresses from the bottom flange gages and the bottom web gage of girder 7 at Section C, it is expected that approximately 2.0 ksi at this location for Load Case 1 is due to longitudinal bending. The remainder of stress (0.0 ksi for Horizontal 1 and 0.7 ksi for Horizontal 2) is predicted to be caused by the stress concentration (Figure 2.92). As with the strain gage configuration used on the web stiffener plates, it was not possible to determine how quickly the stress concentration diminished away from the origin. Furthermore, the signs (tension/compression) of Horizontal 1 and Horizontal 2 are consistent with each other, suggesting that there is not any out of plane bending of the web in this region. The magnitudes of the stresses at this location are very small, and are not expected to be a problem.

2.5.3. COMPARISON OF LOAD TEST AND ANALYSIS RESULTS FOR THE I-65 BRIDGE OVER RIDGE ROAD

2.5.3.1. Modification of Support Conditions

In order to determine the most accurate means of modeling the selected bridge, several comparisons were made between load test data and different FEMs. The FEMs used in this comparison differed only in the types of support conditions. The three different support conditions investigated were roller-roller supports, pinned-pinned supports, and fixed-fixed supports.

The first support condition modeled was roller-roller. The roller-roller supports were generated by using spring elements. Very stiff springs (1×10^{12} kip/in) restrained both vertical and transverse translation. Springs of negligible stiffness (1×10^{-3} kip/in) restrained longitudinal translations. The longitudinal springs were required for longitudinal stability, even though their effects in restraining the longitudinal translation were negligible. Each combination of springs was located at the bottom flange nodes at the centerline of the bearing support (Figure 2.93). Although the bearing plates as well as the ends of the girders are embedded 15 in. into the abutment, this support condition completely neglects any restraint from the integral abutment.

The second support condition modeled was pinned-pinned. The pinned-pinned supports were generated using translational restraints in SAP2000. These translational restraints did not allow translation in the vertical, transverse, or longitudinal directions, but did allow rotation in all directions. Like the roller-roller supports, these restraints were located at the bottom flange nodes at the centerline of the bearing supports which were embedded 15 in. into the abutment (Figure 2.94). This support condition assumes there is no longitudinal displacement of the abutment, but allows the entire abutment to rotate freely.

The third support condition modeled was fixed-fixed. The fixed-end supports were generated using translational and rotational restraints in SAP2000. These restraints did not allow translation or rotation in any direction. This support condition was used to model the interface of the girders and the face of the abutment, therefore this model is 30 in. shorter than the other two models. Recall that the supports for the other two models were located at the centerline of the bearings, which were embedded 15 in. into the abutment. This support condition assumes there is absolutely no movement of the abutment due to live loads. Furthermore, this support condition assumes the full depth of the girder, as well as the concrete deck, are restrained (Figure 2.95).

2.5.3.2. Comparisons between Load Test data and FEM#1 Results

In order to determine the most accurate FEM, a comparison was made among the different models. The measured strains and deflections were compared with analytical stresses and deflections of the various FEMs.

2.5.3.2.1. Girder Longitudinal Stresses and Deflections

Each of the static load cases from the load test were analyzed in each of the three models. Strains and deflections were compared with measured data from the load test to determine which model most accurately predicted the response of the bridge. Deflections were compared first, as seen in Figures 2.96 to 2.100. As evident from these figures, the actual bridge deflection response falls somewhere between the fixed-fixed support condition and the pinned-pinned support condition. The roller-roller support condition overestimated the deflection response by about a factor of two. It should be noted that although the deflection measurements from Load Case 2 reflect the analytical deflected *shape*, the magnitude of the measurements do not fall between the fixed-fixed and pinned-pinned support conditions as all the other Load Cases did. This discrepancy is thought to be a function of errant measurements as described previously.

A comparison with stress/strain data was also studied to determine the most accurate analytical model. The stress distribution over the depth of individual girders was first analyzed. Figures 2.101 through 2.105 show the stress distribution comparison for girder 7 at midspan (Section C) for each load case. An identical comparison was made for all instrumented girders at Sections A, C, and D. All instrumented locations demonstrated a similar trend as shown in Figures 2.101 to 2.105. The fixed-fixed support condition was observed to be the best estimate for locating the neutral axis and identifying top flange stresses, whereas both the fixed-fixed support condition and pinned-pinned support condition seemed to be equally estimating bottom flange stresses. This can further be

shown in Figures 2.106 to 2.115, which show the lateral distribution amongst all girders. From these figures several outlying gages can be identified. Several of the top flange gages seem to have drifted during the course of the load test. Top flange measurements for Load Case 1 and 2 correlate very well with the fixed-fixed support condition, but in Load Cases 3-5 the top flange gages do not seem to match well with any of the support conditions.

The discrepancy between the pinned-pinned support condition and fixed-fixed support condition is further noticed in examining cross sections close to the support. Figures 2.116 and 2.117 show the stress distribution of Girder 7 at Sections A and D, respectively. Varying degrees of stress concentrations occur in all of the models near the supports, but they are functions of the numerical integration of SAP2000, and are not representative of the behavior of the bridge. Furthermore, the pinned-pinned support condition does not even produce a linear stress distribution in this region. At this region, the fixed-fixed support condition was found to overestimate longitudinal girder stresses, but did provide the most accurate prediction of all the support conditions

2.5.3.2.2. Cross-frame Angle Stresses

A comparison with cross-frame stresses was also made to determine which model was most effective. Figure 2.118 shows the results of this comparison. Based on longitudinal stresses and deflections, the roller-roller support condition was discarded, and therefore is not shown in this figure. The analytical stresses from both the fixed-fixed and pinned-pinned load cases generally overestimate (two to three fold) the measured stresses. Furthermore, the precision was not very high. The lack of both precision and accuracy is believed to be due to the manner in which the cross-frames were connected to the web stiffener plate in the models. In addition, it is apparent from the measured data that the horizontal cross-frames experience a substantial amount of bending, but the axis

of bending could not be determined based on only the two gages per cross-frame member.

Since the axis of bending could not be determined, it was assumed for the analytical stress calculations. In the FEM, the cross-frames were modeled by connecting the centroid of cross-frame angles to a node of the web stiffener plates. In order to make a direct comparison between this modeling assumption and the actual geometry of this connection detail, further computations were required. These computations included accounting for an additional constant moment on the cross-frame angle as a result of the eccentricity of the force obtained from the FEA. This eccentricity consisted of one-half of the thickness of the web stiffener plate, the thickness of the gusset plate, and the distance from the centroid of the cross-frame angle to the edge of the cross-frame angle (Figure 2.119). The stresses from both the axial force and the moment caused from the eccentricity of the axial force constituted the computed cross-frame angle stresses shown in Figure 2.118. In addition, the FEA results indicated that negligible moments were transferred from the single node of the web stiffener plate to the cross-frame angle. Although neither analytical support condition model was capable of accurately predicting the measured stresses, the fixed-fixed support condition did provide a slightly better estimate.

Although the results from the fixed-fixed support condition model overestimates the stresses near the abutment and in the cross-frame members, it was the most accurate model overall and, therefore, was used in the analysis of the Superload truck groups. Further adjustments to the FEM to exactly match the measured data could have been applied, but the measured and predicted stresses were small enough that they did not warrant a more refined model.

2.5.3.3. Comparisons between Load Test data and FEM#2 Results

2.5.3.3.1. Girder Longitudinal Stresses and Deflections

Bottom flange stresses obtained at the midspan of the bridge from FEM#2 for Load Cases 1 and 2 are presented in Figures 2.120 and 2.121. Also, the measured deflections for Load Cases 1 and 2 are presented in Figure 2.122 and 2.123. Results obtained from FEMs for simple and fixed support conditions are also presented in Figures 2.120 to 2.123. Both FEMs included all secondary members, such as the cross-frames and the parapets, which resulted in a better agreement between the measurements and calculations. The contribution of the secondary members resulted in a better load distribution for the I-65 Bridge over Ridge Road. The GDFs were up to 30% lower for the case with all secondary members than for the case without any secondary members.

Comparison of the analytical results and measurements shows that the FEM#2 with fixed support condition match almost perfectly with the measurements (see Figures 2.120 to 2.123). The integral abutment bridge behaves as a fixed-fixed span rather than a simple-span. Furthermore, the FEA predicts the bridge response accurately except for the deflections of Load Case 2 presented in Figure 2.123. This was due to the problems encountered in deflection measurements during Load Case 2. Load Case 2 deflections were obtained without using the digital features of the surveying equipment. The accuracy of the measurements was therefore limited, since automated readings could not be obtained. This problem was fixed after Load Case 2 and the other measurements were obtained with a higher sensitivity, which was approximately 0.001 in.

Overall comparison of the calculated values with the measurements indicates that the symmetric FEM of the I-65 Bridge over Ridge Road captures the general behavior well. A total load of 170 kips generated stresses less than 2.5 ksi in the bottom flanges of the girders, which have a minimum yield stress of 50 ksi. Also, the maximum deflection

was about 0.20 in. This value is significantly lower than the traditional deflection limit of span length divided by 800 (AASHTO, 2004), which was 1.8 in for the I-65 Bridge over Ridge Road.

Data from Load Cases 1 and 2 enabled computation of GDFs experimentally. Assuming that the bending moment resisted by each girder will be proportional to the bottom flange strain, the maximum bottom flange strain recorded at the midspan was divided by the summation of strains recorded for each girder. Consequently, the maximum GDF for two trucks side by side was calculated as 0.37. The highest GDF was obtained for Load Case 2, which imposed more demand on the girders since the test trucks were close to the edge of the bridge. For a beam line analysis conducted by running a single truck influence line analysis, this value has to be doubled in order to be consistent with the beam line analysis procedure of AASHTO LRFD (2004). The GDF calculated according to AASHTO (2004) equation for two lanes loaded was 0.80, a value 8% higher than the GDF inferred from the measurements, which is 0.74.

2.5.3.3.2. Girder Stresses at the Abutment

For Load Cases 1 and 2, stresses calculated from the strain measurements at the abutment (Section A) and the FEA are presented in Tables 2.10 and 2.11. The agreement between the measurements and the calculations was not as good as for the midspan stresses for Section A. This might be due to the small stress range or some modeling idealizations. The abutment and the actual connection of the girders with the abutment were not modeled. However, one notable point about the measurements at the abutment was that the stresses are comparable to the midspan stresses. This is regarded as a clear indication of nearly fixed support condition.

2.5.3.3.3. Stiffener Plate, Cross-Frame and Web Gap Stresses

Despite the low longitudinal strains and vertical deflections obtained at the midspan, some notable strains were measured in the stiffener plates and the cross-frame angles. The experimental stresses in the cross-frame angles are presented in Figure 2.124 along with the calculated stresses. The average of the stresses measured in each horizontal angle leg is also presented in the figure. Comparison of the calculated and measured values indicates that the cross-frame angles may have different stress values in each leg. However, the FEA predicted only axial force in the angles and the bending moments were negligible. On the other hand, the average stress for each horizontal angle results in a stress comparable to that from the FEA. Therefore, it can be concluded that the net axial force in the horizontal angles is reasonably approximated by the FEA. The bending stresses are not captured by the FEA due to the modeling idealizations. Possible misalignment problems during the construction might be another reason for the considerable difference between the expected and analytical behaviors. Also, the FEM did not include the gusset plates connecting the stiffener plates to the stiffeners. If the stiffener plates were not lined-up in the same plane as each other, this condition may generate secondary moments. Meanwhile, the stresses in the diagonal members were not approximated well by the FEA. There was only one strain gage on each diagonal member, and the average stress could not be computed. Consequently, the effect of bending stresses on the diagonal members is not known.

The strains measured at the top corners of the stiffener plates at both sides of Girder 7 are presented in Tables 2.12 and 2.13 for Load Cases 1 and 2. Tables 2.12 and 2.13 indicate that the horizontal stresses in the stiffener plates are negligible. However, the highest stresses among the bridge are obtained from the gages measuring strains in the vertical direction in the stiffener plates. Furthermore, the FEA stresses are significantly lower than the experimental values. This underestimate of the stresses in the stiffener plate is considered to be a result of modeling idealizations and simplifications. An FEM with gusset plates modeled and finer mesh could possibly approximate the

actual stresses better. However, the limitations of the FEA tools and the computational resources did not allow this.

Bottom web gap stresses due to Load Cases 1 and 2 are presented in Tables 2.14 and 2.15. Although this region was identified in the literature as a possible fatigue problem area due to out-of-plane bending of the web, negligible stresses were observed at the bottom web gap for the I-65 Bridge over Ridge Road. This is due to the connection of the stiffener plates to the bottom flange. This bolted connection restrains the rotation of the bottom flange with respect to the girder web. Hence, it minimizes the out-of-plane bending problem. The FEA results were comparable with the measurements for the bottom web gap.

The load test of the I-65 Bridge over Ridge Road confirmed that both FEMs represent the general behavior reasonably well. The high stress regions predicted by the FEA, such as the stiffener plates and the cross-frame angles, were subjected to relatively high measured strains in the load test as well. However, the FEA was unconservative with regards to the stresses in the top of the stiffener plates. The maximum bottom flange stress at the midspan due to a total load of 170 kips was significantly lower than the yield stress.

2.6. SUPERLOAD ANALYSIS AND CAPACITY EVALUATION

The superload trucks presented in Chapter 1 were applied to the FEMs of the investigated steel bridges. Gross vehicle weights of the superload trucks used in the analysis are presented in Table 2.16. The superload trucks were positioned to generate the maximum bending moments in the members. The FEMs included parapets and other secondary members such as the diaphragms and the cross-frames. Distribution of load and localized high stress regions were the main issues investigated. The GDFs obtained

from the 3D FEA were used for the calculation of maximum positive moments due to the superloads. For the calculation of shear forces generated by superload trucks, the AASHTO (2004) GDF equation was used. Support fixity due to the integral end abutment of the I-65 Bridge over Ridge Road was not included in the capacity evaluation, since its long term behavior and high magnitude load response were not known. Ratings presented in this section are based on the expected behavior of the analyzed bridges. An influence line analysis was conducted to obtain the maximum moment and shear forces generated by the superload trucks. Afterwards, the maximum moments and shear forces were distributed to the girders using the GDFs.

Only the main members were considered in the capacity evaluations. The secondary member stresses and the out-of-plane bending problems due to superloads are investigated in the later section on damage evaluation. Fatigue of the web was not considered in the capacity evaluations. According to AASHTO LRFR (2003) “existing bridges which have experienced many truck passages, if uncracked, may be deemed insensitive to distortion-induced cracking, even under heavier permit loads.”

The rating equation (AASHTO, 2003) used in capacity evaluation was a general expression as follows:

$$RF = \frac{C - A_1 D}{A_2 (1 + IM) L} \quad (\text{Eqn. 2.3})$$

where RF is the rating factor for the live-load capacity, C is the capacity of the member, D and L are the dead load and live-load effects, A_1 and A_2 are load factors for the dead load and the live load, and IM is the impact factor. There were no capacity reduction factors for flexural and shear capacity of steel bridges.

The Strength II and Service II Limit States of AASHTO LRFR (2003) were considered in the capacity evaluations. The Service II Limit State is intended to control

the stresses in steel girders and to prevent yielding or excessive deformation of members. For the Strength II Limit State, A_1 and A_2 were taken as 1.25 and 1.15 for the dead load and the live load, respectively. These load factors can be used, according to AASHTO LRFR (2003), when a permit is issued for a single trip of an escorted heavy vehicle. Furthermore, the impact factor (IM) could be eliminated provided that the permit vehicle crosses the bridge at crawl speed, which should be less than 5 mph. All load factors were 1.0 for the Service II Limit State. In the tables of this section, load factors for the dead load and the live load are denoted by γ_{LL} and γ_{DL} , respectively. Also, M_{LL} and M_{DL} denote live load and dead load moments. Similarly, V_{LL} and V_{DL} denote live load and dead load shear forces. Furthermore, the load reduction factors are represented by ϕ , which was one for both moment and shear resistance of steel bridges. The live load stress is denoted by σ_{LL} , whereas the dead load stress and the maximum allowable stress in the steel girders are denoted by σ_{DL} and σ_{all} , respectively. The maximum allowable stress is 95 and 80% of the yield stress for composite and noncomposite sections, respectively.

An acceptable value of rating factor (RF) should be greater than 1. If the rating of a bridge is lower than 1 for a permit truck, it indicates that there is a possibility of failure due to that truck. On the other hand, a rating factor much greater than 1 indicates that the bridge has more than enough strength to accommodate that permit truck.

The analysis and rating programs used by INDOT were also investigated. VIRTIS and LARS were the software considered. These programs evaluate the capacity of girders as well as the maximum demands on the girders. Accuracy of the aforementioned programs was not checked by the authors. However, the general principles employed in the calculation of live load demand per girder were examined. The GDFs calculated with respect to AASHTO LRFR or AASHTO Standard Spec are used in the evaluation and rating. The difference in the evaluation using VIRTIS and LARS compared to a detailed analysis or load test is believed to be a result of having different GDF values for each tool. Use of more accurate GDFs in the rating software can significantly improve the rating of bridges for superload trucks. GDFs calculated according to AASHTO are

compared with the detailed analysis results in a following chapter. Furthermore, the beam line analysis approach of AASHTO (2004) does not include the contribution of the secondary members. Secondary members can contribute to load resistance and this reserve strength can be utilized during superload passages. However, the beam line approach does not allow this and a detailed analysis might be necessary. The capacity calculations of these VIRTIS and LARS are expected to be consistent with the AASHTO (2004) provisions. Therefore, use of more accurate GDFs in conjunction with the capacity calculation components of these programs can improve the accuracy of rating process.

2.6.1. US-52 BRIDGE

The superload trucks were applied to the FEM of the US-52 Bridge which included parapets and diaphragms. Distributions of the bending moments at 34.75 ft from the beginning of the first span due to superload trucks and the HL-93 Loading positioned at the center of the right lane are presented in Figure 2.125. It was assumed that a superload would typically traverse the bridge close to the center. Note that bending moment resisted by each member was obtained using the “Group Joint Sum” feature of SAP2000. Each data point in Figure 2.125 corresponds to a composite beam or a parapet.

Figure 2.125 shows that the maximum bending moments due to the Maximum Truck and the Group D Superload are significantly higher than those due to the other superload trucks. Superload Groups A, B and C generate moments comparable to the moment due to HL-93 Loading. For all trucks, the parapet located at 20.75 ft resisted more than 30% of the load resisted by the most heavily loaded beam located at 3 ft. The GDFs due to the superload trucks and HL-93 loading positioned at the center of the right lane are presented in Table 2.17. The GDF values ranged between 0.17 and 0.21. A lower GDF was obtained for the Maximum Truck due to presence of eight wheels per axle. For a single lane loading, the AASHTO (2004) equation gave a GDF value of 0.29, when the

multiple presence factor was factored out. The AASHTO (2004) equation overestimated the GDFs. The difference was more than 50% for some superload groups.

Bottom flange stress is another important parameter for the behavior of a steel bridge and the demand on steel beams. Bottom flange stresses due to superload trucks and the HL-93 loading positioned at the center of the right lane are presented in Figure 2.126. The bottom flange stresses were obtained at the maximum positive moment location, which was at 34.75 ft from the beginning of the first span. The distribution of the bottom flange stresses was similar to that of the girder moments. The maximum live load stress in Beam 4, which was the most heavily loaded member, was between 6.5 and 5 ksi for the Superload Groups A, B, C and the HL-93 loading. However, for the Maximum Truck the live load stress was close to 20 ksi. The dead load stress in the bottom flange of the same beam was about 14.2 ksi, based on an unfactored dead load moment of 588 ft-kip. The bottom flange stress due to the Maximum Truck and dead load totaled about 34 ksi, which was below the minimum yield stress of 50 ksi. Therefore, yielding of the bottom flanges of the US-52 Bridge is not expected to be an issue even for the Maximum Superload Truck. However, other superload truck configurations which were not considered in this study may have the potential to yield the girders in the positive moment region, if the weight is increased or an unfavorable axle configuration is used.

Deflections of the beams of the US-52 Bridge are presented in Figure 2.127. These deflection values were obtained at the first steel span at 34.75 ft from the support. This was also the location of the maximum positive moment. The HL-93 Loading resulted in a maximum deflection of 0.6 in, which was comparable with the deflections due to Group A and B superload trucks. Group C, Group D and the Maximum Superload Trucks led to maximum deflections of 0.8, 1.9 in and 2.8 in. The traditional limit of span length divided by 800 gave a maximum permissible deflection value of 1.4 in for the 92 ft span. Since this criterion was not enforced by AASHTO LRFD (2004) or LRFR (2003), it should not affect the decision on permitting superload vehicles. However, it is

notable that only the deflections due to the Maximum Truck and the Group D Superload Truck exceed the traditional deflection limit.

The capacity of the US-52 Bridge was checked with respect to the Strength II and Service II Limit States per AASHTO LRFR (2003) for superloads traveling in the center of the right lane. The maximum positive bending moments generated by the superload trucks were compared with the live load capacity. The live load capacity and the demand due to superloads as well as the rating at the maximum positive moment location for the Strength II Limit State are presented in Table 2.18. The US-52 Bridge meets the Strength II Limit State for positive moments generated by all superload groups.

The shear resistance of the beams was also checked in accordance with AASHTO LRFR (2003). The shear GDF value calculated according to AASHTO (2004) was 0.47. The shear forces generated by the superloads, the live load capacity and the rating obtained from the ratio of these two quantities are compared in Table 2.19. The maximum shear force was obtained at the second internal support for the Maximum Superload Truck and at the first internal support for all other trucks. Therefore, the shear capacity of the critical section was different for the Maximum Truck. Table 2.19 shows that the US-52 Bridge has adequate shear capacity to accommodate the demands due to all investigated superload trucks. The minimum shear rating for the HL-93 Loading was significantly higher than the minimum shear ratings for the other superload trucks; this was not the case for positive moment (Table 2.18).

The negative moment resistance of the US-52 Bridge was checked at the piers. The maximum negative moment was obtained at the first internal support for all trucks, except the Maximum and the Group D Superload Trucks for which the second internal support was the most critical location. The demand due to each truck and the negative moment capacity for live load at the critical sections are presented in Table 2.20. Note that WF36X230 and WF36X194 sections were used at the first and second internal supports, respectively. Therefore, different negative moment capacities were calculated

for each internal support location. Furthermore, these rolled shapes were not composite with the deck at the negative moment region. The rating factors presented in Table 2.20 show that the US-52 Bridge has adequate negative moment strength to resist the demands due to all applied superload trucks except the Maximum Truck. Also, the negative moment rating factors for the Strength II Limit State were lower than the rating factors for positive moment and shear.

Satisfying the Strength II Limit State checks ensures that the US-52 Bridge has adequate strength to accommodate Group A, B, C and D superload trucks. The Maximum Superload Truck resulted in a demand lower than the positive moment capacity, but the negative moment capacity was exceeded at the negative moment region. Furthermore, the long term effects of superloads should also be checked per the AASHTO LRFR (2003) Service II Limit State. The flange stresses in bending are compared with the limiting stresses for composite and noncomposite sections in Tables 2.21 and 2.22, respectively. The negative moments over the internal supports were checked for the calculation of noncomposite stresses. Tables 2.21 and 2.22 indicate that yielding of the beam flanges in the positive and moment regions is not probable. Therefore, requirements of the Service II limit state are met for all investigated superload trucks.

Superload analysis and capacity evaluation of the US-52 Bridge in accordance with AASHTO LRFR (2003) demonstrated that the controlled passage of all superload trucks included in the database except the Maximum Truck can be allowed. Yielding of the beams or crushing of the deck is not probable for superload trucks with a gross vehicle weight (GVW) less than the Maximum Truck. This was also confirmed with a nonlinear analysis conducted using ANSYS. Yielding or buckling of the beams was not detected in the nonlinear analysis which included geometric and material nonlinearities. The rating factors for moment could be lower than the values presented in this section if the AASHTO LRFD (2004) GDF equation was used for the positive and negative moment regions.

2.6.2. I-65 BRIDGE OVER RIDGE ROAD

2.6.2.1. Refined FEM#1 Results for Superload Truck Groups

From the preliminary FEA for the I-65 Bridge over Ridge Road, it was found that the transverse location of the Superloads did not significantly affect the magnitude of the maximum stresses. Comparisons between various load cases within the load test also support this trend. Load Case 1 and 2 have very comparable maximum bottom flange stresses. Similarly, Load Case 3 and Load Case 4 have very comparable maximum bottom flange stresses, as well as maximum deflections. The change in support conditions did not change this trend. Consequently, only one transverse truck location was analyzed with the final model. All superload trucks were modeled as traveling in the right lane, similar to Load Case 3.

The FEM was analyzed with the design trucks (HS20 and Toll Loading) and the results were compared with those of Load Cases 1 and 3. In addition, the bridge was analyzed with the Maximum Truck to provide an upper-bound of deflections and stresses. Then, the four different Superload Group loadings were analyzed with the FEM. The girder deflections from the design trucks, Maximum Truck, and Superload Groups can be seen in Figures 2.128 and 2.129. From these figures, it is apparent that there is little difference in the effects of the two design trucks (HS20 and Toll Loading). Furthermore, Load Case 3 of the load test matches the response of the two design trucks very well. Based on the Maximum Truck, the maximum predicted girder deflection is approximately 0.9 in. (L/1570), which is much less than the AASHTO recommended L/800 limit (AASHTO LRFD, Section 2.5.2.6.2, 2004).

The maximum longitudinal stresses shown in Figures 2.130 and 2.131 show a similar trend as with the deflections. Little difference can be seen between the two design

trucks and Load Case 3. From Figure 2.131, it is apparent that there is little difference in the effects of the Group A and B trucks. The absolute maximum girder deflections and longitudinal bending stresses for the Group C trucks, which consisted of 193 kips on the bridge, match fairly well with Load Case 1 of the load test, which consisted of 171 kips on the bridge. The maximum longitudinal girder stress for the Maximum Truck is approximately 7 ksi, 15% of design yield strength. As expected from the preliminary FEA, none of the Superload trucks included in this analysis cause damaging deflections or longitudinal girder stresses. Maximum non-composite longitudinal dead load stresses were found to be approximately 11 ksi.

2.6.2.2. Refined FEM#2 Results and Capacity Evaluation for Superload Truck Groups

Superload analysis of the I-65 Bridge over Ridge Road was conducted using the previously described symmetric FEM (FEM#2). The fixity due to the integral end abutment was not considered in the capacity analysis. Although nearly full fixed support conditions were encountered during the load test, response of the abutment at the ultimate load level was not known. Therefore, fully fixed support condition is considered to be an upper bound for capacity evaluation; it may not be safe to rely on this fixity for superloads. However, the secondary members were included in the analysis. The live load test of the I-65 Bridge over Ridge Road and the detailed analysis results indicated that the parapets and cross-frames would participate in live load resistance even for heavy loads.

For the simple span bridge model, only the positive moments at the midspan and the shear forces at the support were considered in the capacity evaluation. The distributions of the positive moments at the midspan due to all investigated superload trucks and the HL-93 loading obtained using the “Group Joint Sum Feature” of SAP2000 are presented in Figure 2.132. In this analysis it was assumed that a superload truck would traverse the bridge following the centerline for the transverse direction.

Furthermore, the controlled passage of the superload truck would cross the bridge at a crawl speed in order to eliminate the effects of impact. Figure 2.132 shows that the demand due to the HL-93 loading and the Group A Superload truck are comparable. Superload Groups B and C generated relatively higher bending moments and the moments due to the Maximum and Group D Superload Trucks were significantly higher than those of the other superloads. The GDFs for the superloads and the HL-93 loading are presented in Table 2.23. The Maximum Truck, with eight wheels per axle, resulted in a lower GDF than other trucks.

The distribution of the bottom flange stresses at the midspan of the I-65 Bridge over Ridge Road due to superload trucks and the HL-93 loading are presented in Figure 2.133. The bottom flange stresses at the midspan were obtained from the 3-D FEA. The live load stresses in the flanges were between 3 and 4 ksi for Superload Groups A, B and C, and the HL-93 loading. The Maximum Truck led to a bottom flange stress of 12 ksi. Self weight of the bridge generated a tensile stress of 19.5 ksi in the bottom flanges, providing a total bottom flange stress of about 32 ksi at the midspan for the Maximum Truck. This value was significantly lower compared to the minimum yield stress of 50 ksi.

Deflections of the girders at the midspan of the I-65 Bridge over Ridge Road due to superloads and HL-93 loading are presented in Figure 2.134. The largest deflection, obtained for the Maximum Truck, was about 2 in. The maximum girder deflection slightly exceeded the traditional limit of span length divided by 800, which gave a maximum permissible deflection value of 1.8 in. All other trucks resulted in deflections much lower than the traditional limit.

The capacity of the I-65 Bridge over Ridge Road was checked in a similar manner as for the US-52 Bridge. Strength II and Service II Limit States (AASHTO, 2003) were checked for controlled passages of superload trucks traversing the bridge at the center. The maximum bending moments generated by the superload trucks were determined

through an influence line analysis and distributed to the girders using the GDFs presented in Table 2.23. The maximum positive moments generated by superload trucks are compared with the live-load capacity in Table 2.24. The comparison shows that the I-65 Bridge over Ridge Road has adequate positive moment capacity to accommodate all superload trucks included in the database.

Shear capacity of the girders at the abutment were also checked for all superload trucks. As for the shear evaluation of the US-52 Bridge, the AASHTO (2004) shear GDF equation, which gave a GDF value of 0.68, was used in the calculation of the maximum shear forces. Comparison of the maximum shear forces generated by the superload trucks and the HL-93 loading are presented in Table 2.25. Rating of the I-65 Bridge over Ridge Road for superload trucks shows that the bridge has adequate shear strength for all superload trucks included in the database except for the Maximum Truck. The bridge could support a permit truck which has a GVW corresponding to 90% of the GVW of the Maximum Truck. Furthermore, it is likely that the use of a more accurate GDF for shear could result in a lower demand and a higher rating factor for the Maximum Truck.

The maximum bottom flange stresses at the midspan were checked to obtain the bridge ratings for the Service II Limit State. None of the superload trucks resulted in yielding of the girders, as shown in Table 2.26. Therefore, no permanent deflection is expected due to the superloads included in the database.

Superload analysis and capacity evaluation of the I-65 Bridge over Ridge Road shows that all superload trucks included in the database except the Maximum Truck can safely traverse the bridge provided that speed and transverse location of the truck is controlled and no any other trucks are allowed on the bridge during the superload passage. Response of the I-65 Bridge over Ridge Road was also checked through a nonlinear analysis conducted using ANSYS. When the superload trucks were positioned to generate maximum positive moment, no yielding or buckling was observed in the steel girders.

2.7. LONG TERM MONITORING OF THE I-65 BRIDGE OVER RIDGE ROAD

Once the load test was completed, the data acquisition system remained in place to monitor normal truck traffic and to measure the response to any potential superloads traveling over the bridge. The I-65 Bridge over Ridge Road was monitored for more than two years using different trigger levels. No superload passages were reported by the Permits Division of INDOT in this period of time. The data loggers were set up to record all channels for six seconds (two and one-half second pre-trigger and three and one-half second post-trigger) with a scan rate of 50 Hz (300 data points per event). A typical event is presented in Figure 2.135. The two bottom flange gages of girder 7 at Section C were designated as the trigger channels since girder 7 is between the right and center lanes. The trigger was initiated when a strain reading from either of the two bottom flange gages of girder 7 exceeded $60\mu\epsilon$ greater than a 1000 point moving average. Unfortunately, in the four months this bridge was monitored no reported superloads crossed the bridge as anticipated. However, while monitoring normal truck traffic, several events which caused stress levels similar to those expected by a single superload were observed.

All dynamic data presented in this section were subjected to a low-pass filter. Examples of pre-filtered data and post-filtered data are shown in Figures 2.136 and 2.137, respectively.

2.7.1. MAXIMUM GIRDER STRESSES/STRAINS

Data were recorded continuously for 49 days with the $60\mu\epsilon$ trigger. After this period of time, the data logger programs were adjusted to look at different aspects of the bridge. During the 49 days of continuous recording several events were observed to cause stresses equivalent to those of the Group A, Group B, and Group C Superloads. Figure

2.138 shows three of these events which produced midspan stresses equivalent to the Group C Superload and Load Case 1. During this period of time, no superload trucks were reported by INDOT to be traveling over this bridge, so the axle configuration which caused these events are unknown. For some of the events shown in Figure 2.138, the noise-to-measurement ratio was too high to accurately discern the measurements for girders 4 through 1, and therefore zero stress was reported for these girders.

An attempt was made to predict a single truck or a group of trucks that may have caused these comparatively large girder stresses. It was first assumed that only one truck caused the stress distributions found in Figure 2.138. Weigh-in-Motion (WIM) sensors, which measure axle weights and spacings of trucks driving over the sensors, were coincidentally located approximately two miles south of the instrumented bridge. These WIM sensors were part of a separate research project coordinated by the Purdue University Civil Engineering-Transportation Department. Unfortunately, the WIM data for the period of time in which the bridge was instrumented were not available. WIM data for the previous year however, were available. Several months worth of this data were obtained and analyzed. An assumption was made that the trucks causing the stresses shown in Figure 2.138 were Class-9 trucks, which were one of the most common truck configurations observed. The maximum GVW of any Class-9 trucks recorded by the WIM data available was 115 kips.

The axle weights and spacings of all Class-9 trucks with a GVW of 80 kips or greater were then compiled to obtain an “average” Class-9 truck which represented average axle weight and spacing distributions. The relative axle weights with respect to the GVW and the average axle spacings for a “Hypothetical Class-9 Truck” can be seen in Figure 2.139. With this axle weight and spacing configuration, a GVW which would cause stresses equivalent to the Group C Superload was needed. In order to determine this GVW, a bending moment analysis was performed. A simply-supported beam-line bending moment analysis was conducted for each of the Superload Groups as shown in Table 2.27. Although the simply-supported beam-line moment analysis contradicts the

fixed-fixed the support conditions used in the FEM, it can still be used to compare relative moment magnitudes among different truck groups. These moments do not represent the actual moments experienced by the bridge.

Using this simply-supported beam-line moment analysis, the GVW of the Hypothetical Class-9 Truck was adjusted while maintaining the axle weight ratios, until a maximum midspan moment equivalent to the Group C Superload was obtained. This moment can also be seen in Table 2.27. For comparison, the load test trucks are also presented in this table. The GVW of the Hypothetical Class-9 Truck which produced an equivalent moment to the Group C Superload was 218 kips, and consisted of four axles over 40 kips each (Figure 2.140). This truck configuration constitutes an illegal truck, both due to the GVW which would be classified as a Superload, and to the axle weights which are well above legal limits. Regardless of this fact, this Hypothetical Class-9 Truck was analyzed using the same FEM, which confirmed that the maximum girder stress could be predicted based on the simply-supported beam-line moment analysis.

The Hypothetical Class-9 Truck with a GVW of 218 kips was not thought to be realistic. Therefore, it was then assumed that two Class-9 trucks driving side-by-side may have caused the stresses in Figure 2.138. Two Class-9 trucks each having a GVW of 109 kips (Figure 2.140) were positioned in adjacent lanes and analyzed using the same FEM. The girder stresses for the two Hypothetical Class-9 trucks were found to be lower than the single Hypothetical Class-9, but they do provide a legal alternative to the causes of the stresses shown in Figure 2.138. Figure 2.141 shows the comparison between the single Hypothetical Class-9, the two Hypothetical Class-9s, and three of the measured events.

During the 49 days of continuous recording, 270 events causing $60\mu\epsilon$ (1.75 ksi) or greater were recorded. Figure 2.142 shows the number of events which produced various stress ranges greater than the trigger value. Based on 2004 A.A.D.T., approximately 7,200 trucks pass over this particular bridge everyday. Based on this projection, the 270

measured events which caused stresses greater than 1.75 ksi, constitute less than 0.08% of the average annual daily truck traffic. Furthermore, trucks which produce greater than 2.0 ksi longitudinal girder stresses constitute less than 0.01% of the average annual truck traffic for this specific bridge, and have a recurrence interval of approximately 1.4 days.

2.7.2. STRESS CONCENTRATIONS (STIFFENER PLATES)

Since the stiffener plate stresses were found to be the highest stresses measured on the bridge, a more thorough analysis was desired for this potential fatigue critical detail. The capabilities were available to obtain an inventory of stress ranges for the web stiffener plates. At the conclusion of the 49 days of continuous recording for longitudinal stresses, the program on one of the data loggers was changed to solely collect stiffener plate stress data. The program for one data logger was compiled to trigger off of the two vertical strain gages on the web stiffener plates. The trigger value was set at $30\mu\epsilon$ above or below (since the stiffener plates experienced both tension and compression) a 1000 point moving average. Once again, the data loggers were set up to record for six seconds (three second pre-trigger and three second post-trigger) with a scan rate of 50 Hz (300 data points per event). With the lower trigger value, an extreme amount of data would be collected, and for this reason only the web stiffener plate gages were recorded. Data was recorded at the $30\mu\epsilon$ trigger level for eight consecutive days.

In the eight days of recording, nearly 34,000 events were measured. Depending upon the lane of travel of each of these events, tensile or compressive stresses were generated in the web stiffener plates. A distribution of the measured tensile stress ranges is shown in Figure 2.143. Of the almost 34,000 recorded events, 13,362 measured tensile stresses of greater than 1.0 ksi ($34\mu\epsilon$). The maximum tensile stresses recorded consisted of one event that measured 7.9 ksi, a second event that measured 7.5 ksi, and a third event that measured 6.5 ksi. The remainder of the events measured less than 5.5 ksi.

2.8. EFFECTS OF SUPERLOADS ON STEEL BRIDGES

Capacity evaluation of the investigated steel bridges revealed that the superload groups presented in Chapter 1, or some other comparable superload trucks, can probably traverse the investigated steel bridges without causing any damage in the main members. The Maximum Truck can be regarded as an exception, but the ratings obtained for this truck were close to one. Therefore, a similar superload configuration with a slightly lower GVW could safely traverse both of these bridges. The capacity evaluation according to AASHTO LRFR (2003) focused on main members, and secondary members were not considered. Furthermore, no fatigue analysis was needed according to AASHTO LRFR (2003) for the rating calculations, since the frequency of loading was low and there were not any details in the main members of either bridge that were particularly fatigue prone. Therefore, only flexural and shear resistances were checked. However, a detailed FEA of the investigated steel bridges indicated that stresses significantly higher than those of the main members can be attained in the secondary members or the web gap area.

The US-52 Bridge and the I-65 Bridge over Ridge Road had two common types of bracing members: diaphragms and cross-frames, respectively. Both of these bracing systems were subjected to live load stresses, although their designs were not based on any live-load effects. Furthermore, they were generating some considerable stresses in the main members to which they were connected. The effect of superload trucks with respect to the bracing type will be presented in the following subsections.

2.8.1. EFFECTS OF SUPERLOADS ON DIAPHRAGMS AND WEB GAP

The US-52 Bridge is a typical example of bridges with welded diaphragms. The diaphragms are used to laterally brace the longitudinal rolled shapes. This type of bracing

has been used in many bridges throughout Indiana. The FEA of the US-52 Bridge revealed possible damage locations for the US-52 and similar bridges.

Superload trucks were positioned at the center of the right lane of the US-52 Bridge for the evaluation of stresses related with the secondary members. The deflected shape of the US-52 Bridge subjected to live load clearly shows out-of-plane bending of the web. The cross-section of the US-52 Bridge subjected to live-load is shown in Figure 2.144. Differential deflections of the beams result in bending of the deck in the transverse direction. The top flanges of the beams align with the deck, but the diaphragms pull the webs of beams. This action leads to out of plane bending of the beam web as shown Figure 2.144. Also, an isometric view of the diaphragms and beams is presented in Figure 2.145 (note that the deck elements are hidden).

The diaphragm behaves like a floor beam. Furthermore, the diaphragms together can be considered as one floor beam, across the width of the bridge, acting compositely with the deck. The webs of the beams act to enforce curvature compatibility between the deck and the diaphragm in the transverse direction. High stresses in the web area between the diaphragm and the top flange of the longitudinal beam, or the web gap, considerable bending stresses in the diaphragms, and the welds connecting the diaphragms to beam web, are the result of the interaction between the main members and the diaphragms.

Stresses generated by superload trucks in the diaphragm-to-web connection and in the web gap were obtained from the finite element analyses conducted using ANSYS and SAP2000. Stresses parallel to the direction of the diaphragm indicate that the diaphragm stresses are dominated by bending as shown in Figure 2.145 for the Maximum Truck. Stresses calculated at the top and the bottom of the diaphragm were comparable. It is notable that the stresses in the diaphragms for the Maximum Truck were close to the minimum yield stress of 36 ksi. The highest stresses in the diaphragms and the web gap were obtained due to the Maximum Truck. The maximum stresses in the bottom flanges of the diaphragms due to superload trucks and the Fatigue Truck are presented in Table

2.28. The stresses presented in Table 2.28 were obtained from a linear analysis conducted using SAP2000. Mesh density of the FEM is expected to affect the stresses computed in this stress concentration region.

Except the Maximum Truck, all trucks resulted in a stress range significantly lower than the yield stress in the diaphragm-beam connection. If the effective stress due to regular truck traffic is assumed to be represented by the fatigue truck, it can be said that the superload trucks generate stresses 2 to 9 times the regular traffic stresses. However, rare superload passages and stresses lower than the yield stress level suggests that typical superload trucks may not cause additional damage to the diaphragm-beam connection for the US-52 Bridge. On the other hand, some other bridges with similar details may have stresses higher than the yield stress in the diaphragms. This can result in residual stresses in the connection or the diaphragm after a superload passage, and possibly exhaustion of the connection ductility if large permit loads are repeated frequently.

Cracking of the web in the web gap region was another form of damage observed in bridges with welded diaphragms. Barth and Bowman (2002) observed cracking in the top and bottom web gap regions due to out-of-plane bending in their tests. The out-of-plane distortion mechanism shown in Figure 2.144 generated considerably high stresses in the vertical direction. A linear analysis indicated that stresses higher than the yield stress were obtained in the web gap due to the Maximum Truck. The Von Mises stresses obtained in the top and bottom of the top web gap from a nonlinear analysis are presented in Figure 2.146. The top of the web gap corresponds with the intersection of the top flange and the web, whereas the bottom of the web gap is at the bottom flange of the diaphragm. The maximum vertical stresses obtained in the top and bottom of the top web gap from a linear analysis are presented in Tables 2.29 and 2.30. The Maximum Truck generated a vertical stress of 130 ksi in the top web gap in a linear analysis. The Superload Groups A, B and C generated 2 to 3 times the vertical stress generated by the fatigue truck, which was 14 ksi. Stresses at the bottom of the top web gap were

approximately 40% lower than the stresses at the top. The Maximum Truck generated 63 ksi vertical stresses in the bottom of the top web gap.

A nonlinear analysis was conducted in addition to the linear analysis of the US-52 Bridge for evaluating the extent of nonlinearity observed in the top web gap. Only the Maximum Truck was considered in the nonlinear FEM prepared using ANSYS. Both geometric and material nonlinearities were included in this analysis. Distribution of the equivalent plastic strains in the webs of the beams is presented in Figure 2.147. Nonlinear analysis of the US-52 Bridge gave an equivalent plastic strain magnitude of 0.0014 due to the Maximum Truck. This strain value corresponds to approximately 80% of the yield strain. Therefore, the plastified region is not expected to experience any strain hardening due to the limited number of overload cycles caused by typical superload trucks. Furthermore, the strains generated by the Maximum Superload Truck were well below the rupture strain. The fillet between the web and the top flange was not included in the FEM. Therefore, the actual stresses in the top of the top web gap might be smaller. However, yielding took place only in the top web gap as shown in Figure 2.147.

Jajich and Schultz (2003) developed a simple technique to predict the stresses in the bottom of the top web gap. A range for the applicability of the proposed equation was not specified. Differential deflection of the girders was found to be directly proportional to the maximum stresses observed in the web gap. The maximum stress in the bottom of top web gap (σ_{wg}) can be calculated with the following equation according to Jajich and Schultz (2003):

$$\sigma_{wg} = \frac{2Et_w\Delta}{gL} \quad (\text{Eqn. 2.4})$$

where E is the elastic modulus of steel, t_w is the thickness of web, Δ is the relative deflection of the beams, g is the web gap length, and L is the spacing of the beams. Use of Equation 2.4 gave the web gap stresses presented in Table 2.31 for the maximum

relative deflection values, Δ_d , also presented in Table 2.31. The maximum stresses computed using Equation 2.4 were approximately 30% lower than the stresses computed from the FEA. This can be due to the variability of the stresses in the connection with respect to mesh density. Accuracy of the FEA stresses depends on the mesh density. Furthermore, accuracy of Equation 2.4 is expected to be limited. Therefore, stresses computed using Equation 2.4, can be regarded as a quick and effective way of checking the stresses in the web gap. The actual stress ranges in the web gap can only be determined through instrumentation.

2.8.2. EFFECTS OF SUPERLOADS ON CROSS-FRAMES

Bridges made of plate girders like the I-65 Bridge over Ridge Road may experience high stresses in the secondary members or in the web gap region if the vertical stiffener plates are not attached to the top and bottom flanges. The problems observed due to out-of-plane bending of the web gap can be evaluated using an approach presented in the previous section. However, assessment of the possible problems due to stresses observed in the cross-frame components have not been addressed in the literature. FEA and instrumentation of the I-65 Bridge over Ridge Road indicated that top of the stiffener plates and the cross-frame angles might be subjected to stresses considerably higher than those of the primary members. The deflected shape of the I-65 Bridge over Ridge Road due to a superload truck positioned at the center is presented in Figures 2.148 and 2.149. Figures 2.148 and 2.149 show the vertical stresses generated in the stiffener plates as well as the overall deflected shape.

The top edge of the stiffener plate was welded to the top flange and the side of the plate was welded to the web of the girder. The bottom edge of the stiffener plate was bolted to the bottom flange. As a result of this configuration, the stiffener plates effectively resisted out-of-plane bending of the girder web. Out-of-plane bending in the web and stiffener plates is generated because of the restraint provided by the secondary

members. If there were no cross-frames, the web would be perpendicular to the top flange. The top flanges would follow the curvature of the deck in the transverse direction, and the webs of the girders would be completely perpendicular to the deck. However, the angles connecting the bottom flanges to each other do not allow the webs to remain straight. The bottom flanges are pulled together. The web and the stiffener plates at the two sides of the web act like beam elements subjected to bending when they are resisting out-of-plane bending of the web. This action generates significant stresses in the top of the stiffener plates and the angles. The most critical locations were found to be the top corners of the stiffener plates and the web gaps.

Results of the FEA indicated a stress concentration at the top of the web stiffener plate welded to the top flange. The vertical stresses in the stiffener plates and the web gap can be seen in Figures 2.150 and 2.151, and the stiffener stresses are tabulated in Table 2.32. At one side of the web, the maximum vertical tensile stress was obtained (Figure 2.150), and the maximum compressive stress was obtained at the other side. The absolute values of the stresses at two opposite corners were very close to each other (the difference was less than 1 ksi for all cases). The Maximum Truck generated stresses close to the yield stress of 36 ksi in the stiffener plate.

Another location that should be considered with respect to out of plane bending is the web gap region. In Figures 2.150 and 2.151, the vertical stress contours of the stiffener plates and the web gap region due to the Group D truck are presented. From Figures 2.150 and 2.151, it can be inferred that the web gap region is bending in double curvature. In these figures, stresses at the top and bottom of the top web gap region have opposite signs. The maximum tensile stresses calculated in the top web gap are presented in Table 2.33. The maximum tensile stress was calculated as 3.5 ksi in the top web gap. The stresses calculated in the bottom web gap were much lower. Moreover, the web gap stresses are not nearly as large for the I-65 Bridge as that observed for the US-52 Bridge due to the positive connection of the stiffener to the girder web and flange.

The cross-frame angles were also subjected to considerable stresses due to superloads. The maximum stresses calculated in the horizontal angle between Girders 5 and 6 are presented in Table 2.34. The horizontal angle was resisting a live-load demand close to the yield stress of 36 ksi due to the Maximum Truck loading.

Positioning the superload trucks at the shoulder resulted in lower distortional stresses in the web gap and relatively lower stresses in the secondary members, since the curvature of the deck was reduced. The decrease in elastic stresses was about 30%. However, positioning the superload trucks close to the edge was not favorable, since the demand on the main members was higher for this loading position.

Superload analysis of the I-65 Bridge over Ridge Road showed that the web gaps, both top and bottom, were not subjected to considerable stresses due to superload trucks. However, the stiffener plates and cross-frame angles had the largest stresses among all members. Although these members were not designed to resist live-load demands, they were contributing to live-load resistance. For the I-65 Bridge over Ridge Road, yielding of the secondary members is not probable for most of the superload trucks, but could be an issue for superload trucks like the Maximum Truck. Comparison of the FEA results with the load test results showed that the FEM has the potential to underestimate the actual stresses for the stiffener plates. Therefore, stresses higher than the FEA predictions may be obtained in the secondary members.

2.9. DAMAGE EVALUATION

AASHTO LRFR (2003) does not allow any damage to main members such as the girders and the deck of the slab-on-girder bridges. Therefore, a truck which could cause immediate damage to main members is not expected to obtain a permit. However, damage to the secondary members or damage to the main members due to interaction with the secondary members was not addressed by AASHTO LRFR (2003). Therefore,

immediate or long term damage to the secondary members or the web gap can be probable for some bridges. For example, AASHTO LRFR (2003) does not have any limitations on the stresses in the web gap area or the diaphragm-to-beam connection. The diaphragm-to-beam connections or the cross-frames are not addressed in the rating manual. Long term damage related with the secondary members of steel bridges is evaluated in this section. Notable studies on the effects of overloads on fatigue life are also summarized. Furthermore, a relationship to evaluate the fatigue life of the stiffener plate-top flange connection is presented.

2.9.1. DIAPHRAGMS AND WEB GAP

The web gap region and the diaphragm-to-beam connection were shown to be high stress regions with respect to the detailed FEA results. Cracking of the beam web due to out-of-plane bending was observed in many bridges with welded or bolted diaphragms across the country. This damage mechanism has been studied extensively by many researchers and notable studies are summarized below.

Barth and Bowman (2002) performed a series of tests to evaluate the fatigue behavior of welded diaphragm-to-beam connections. Cracking of the girder web or fracture of the welds connecting the diaphragms to the webs of the beams were some damage mechanisms observed during their tests. Fracture of the welds was less critical compared to cracking of the web of the main beam. It was also stated that the system did not fail immediately, stable crack growth was observed, and several millions of cycles of load could be applied. Furthermore, the magnitude of the load applied in the tests was considered to be higher than what would typically be observed at a bridge due to regular traffic. Furthermore, most of the specimens resembled Category D characteristics in terms of S-N relationships, and some specimens even approached the Category C S-N curve. Barth and Bowman's study (2002) suggests that fatigue behavior of diaphragm-to-beam connections might be comparable to behavior of a Category C or D detail as well.

Therefore, effects of the superloads can be conservatively evaluated for intermittently welded diaphragm details by using a Category D detail.

Cousins et al. (1998) investigated reasons of fatigue cracking observed in diaphragm-girder connections of hundreds of bridges in Maryland. Cracks were observed in the base material and the weld. Distortion induced cracking was indicated as the main reason for cracking, and a bolted-angle retrofit was recommended as an effective retrofit technique. The economic impact of the fatigue problems due to distortion was also reported in their study. The total repair costs over a 4-year period were more than \$8,000,000 at the time of their research.

Roddis and Zhao (2003) conducted finite element analyses to investigate the distortion-induced fatigue in steel bridges. They have noted that the current specifications do not address any methods for prediction of secondary stresses. A two girder bridge was investigated as a case study. Although the girder type and the web gap depth were different than that of the US-52 Bridge, the same out-of-plane bending mechanism was causing cracks in the bridge studied by Roddis and Zhao (2003). Welding stiffener plates to the top flange to minimize the distortional stresses was recommended as a retrofit strategy.

It is evident that superload trucks may exacerbate possible problems observed in bridges with diaphragm-to-beam connections. Since the fatigue problems observed in this area may occur due to regular traffic, effects of the superloads might be similar to those of the overloads on Category C or D details. Therefore, overload response of these details subjected to cyclic loading should govern the long-term behavior of this region. Overload response of fatigue prone details will be presented in a separate subsection below.

2.9.2. FATIGUE RESISTANCE OF STIFFENER PLATES WELDED TO THE TOP FLANGE

There have been numerous studies investigating the stress concentration problems at the web gap region and the mechanism of the out-of-plane bending problem (Fisher and Keating, 1989). A common type of failure reveals itself as cracking of the girder web due to out of plane bending stresses. The crack usually starts at the weld toe of the stiffener-to-web weld in the tension zone; this type of failure is covered by the AASHTO Fatigue Category C. However there have not been any studies on the fatigue resistance of stiffener plates welded to the top flange in the compression zone.

Another possible limit state is the fracture of the welded connection between the top flange and the vertical stiffener plate. Under certain loading conditions and geometric properties, this mode of failure can be critical. This type of failure has not been considered thoroughly in the current specifications. However, increasing load spectrum for highways and the future improvements of the analysis methods are expected to result in a higher demand on those types of details. Therefore, fatigue resistance of secondary members should be checked to prevent any possible damage. Damage to a secondary member may not jeopardize the overall integrity of a bridge by itself, but its influence on the primary members might be of concern. Fatigue life evaluation of the stiffener plate elements with welds susceptible to cracking will be presented in the following sections.

As shown in Figures 2.148 and 2.149, the stiffener plate behaves like a cantilever beam which is connected to a fixed support (top flange) with the load applied at the bottom by the horizontal angle. There is also some axial force effect for the stiffener plate for some truck locations, but the stresses are generally due to the bending of the member. For evaluation purposes, the stiffener plate to top flange connection can be conservatively classified as a cruciform joint which is subjected to in-plane bending.

The fatigue strength of cruciform joints subjected to axial loads has been extensively studied by Frank and Fisher (1979). Two types of failure have been

identified: the load- carrying fillet weld cruciform may fail by fatigue cracks starting at the weld toe or from the weld root (Figure 2.152). The weld penetration has been shown to be a major factor affecting the mode of failure. Weld toe failure is categorized as a Category C detail. The nominal stress in the vertical stiffener plate controls the fatigue life with respect to this failure mode. On the other hand, cracking of the weld due to shear stresses is controlled by the provisions of Category F (Frank and Fisher, 1979). The crack starts propagating parallel to the stiffener plate initially (Figure 2.152). Then, it rotates about 45 degrees and propagates through the weld throat. This is a severe and highly penalized detail in terms of stress limitations.

The lack of penetration creates a crack-like condition at the weld root. The fracture mechanics relationships developed by Frank and Fisher (1979) provide a reasonable estimate of the fatigue strength. The fatigue life is approximated with the following equation.

$$N = \int dn = \frac{1}{C} \int_{a_i}^{a_f} \Delta K^{-3} da \quad (\text{Eqn. 2.5})$$

in which a_i and a_f are the initial and the final crack lengths, C is the material crack growth constant and ΔK is the range of stress intensity factor. Based on a finite element analysis solution Frank and Fisher (1979) developed an expression for the ΔK equation. Furthermore, numerical integration of the ΔK equation resulted in the following expression for fatigue life calculation based upon the assumption of no weld penetration:

$$N = \frac{S_R^{-3}}{C} \frac{\left(0.06 + 0.79 \frac{H}{t_p} \right)^3}{\sqrt{t_p}} \quad (\text{Eqn. 2.6})$$

where S_R is the nominal stress range in the plane of the stiffener, H is the weld leg height and t_p is the thickness of the plate (Figure 2.152). The lower-bound value of C for weld root failures was taken as 3.6×10^{-10} for common low carbon steels. This value provides a conservative estimate for life when compared with the experimental data. Therefore, the same value will be used throughout this study.

Frank and Fisher's study (1979) is applicable to a crack starting at the lack of penetration zone and propagating towards the weld under constant tension. Therefore, it can be accepted as a one dimensional crack propagation problem for which the crack propagates through the thickness of the weld. However, in-plane bending of the stiffener plate is expected to drive the crack in the weld in two directions, starting at the edge of the stiffener plate and progressing through the thickness and along the length of the weld. Although the crack in this study propagates in the same plane with that of Frank and Fisher's study (1979), the same stress intensity factors may not be used to compute the growth rate. The stress intensity factor needs to be corrected for the stress gradient and free surface effects.

Dividing the growth of crack into two phases simplifies the calculations. The first phase is idealized as an edge crack starting at the corner of the weld and propagating as a quarter circle through the thickness of the weld as shown in Figure 2.153. Therefore, the effect of an additional free surface for a two-dimensional crack propagation problem can be obtained by multiplying the readily available K factor with the free surface correction factor 1.12 (Barsom and Rolfe, 1999). From equations 2.5 and 2.6, using the free surface correction, first phase of life can be approximated as:

$$N_1 = 0.7 \frac{S_{RE}^{-3}}{C} \frac{\left(0.06 + 0.79 \frac{H}{t_p}\right)^3}{\sqrt{t_p}} \quad (\text{Eqn. 2.7})$$

in which S_{RE} is the maximum stress generated at the edge of the stiffener plate due to in plane bending. Note that the stress gradient effect is not considered in this phase of the crack propagation and it is assumed that the quarter circle is subjected to the same stress range. Furthermore, contribution of the weld portion at the edge of the plate is conservatively neglected (Figure 2.153). This conservative approach has also been employed by Frank and Fisher (1979) and Lawrence et al. (1981).

At the second phase of crack propagation, the crack propagates parallel to the direction of the weld after the quarter circle radius approaches the weld height H as shown in Figure 2.153. The initial crack size at this phase is assumed to be equal to the size (height) of the weld and it propagates up to the termination of the weld, which gives a final crack size of approximately half of the top flange width (w_{flange}). Therefore, the final crack size is equal to the length of the weld. For a cracked beam in bending, the stress intensity factor (K) is calculated as follows by Bannantine et al. (1990):

$$K = F_G S_R \sqrt{\pi a} \quad (\text{Eqn. 2.8})$$

where F_G is the correction factor that depends on the specimen and crack geometry, S_R is the stress range at the edge of the bending beam, and a is the crack size. Bannantine et al. (1990) provide the following expression for the calculation of correction factor:

$$F_G = 1.122 - 1.40(a/L) + 7.33(a/L)^2 - 13.08(a/L)^3 + 14.0(a/L)^4 \quad (\text{Eqn. 2.9})$$

in which a is the crack length and L is the dimension of the beam along which there is a stress gradient. For a stiffener plate welded to a top flange, L corresponds to the sum of the stiffener plate widths (at both sides of the web) and the web thickness. The crack is assumed to be propagating along the weld throat in this case, and the maximum stress occurring at the edge of the stiffener plate, S_{RE} , will be used to compute the shear stress at the weld throat. From the equilibrium of a finite length along the stiffener plate, the nominal stress due to bending (which is assumed to be constant for that finite length)

equals the sum of the shear stresses resisted by the weld. Also, it can be conservatively assumed that the maximum tensile stress along the critical plane (weld throat) is equal to the division of the applied force to the throat area (Salmon and Johnson, 1996). This assumption results in the following relationship between S_{RE} and the approximate stress normal to the critical plane, which is along the weld throat and will be called S_{RT} :

$$S_{RT} = S_{RE} \frac{t_p}{1.41H} \quad (\text{Eqn. 2.10})$$

Substituting 2.10 and 2.9 into 2.8 gives the stress intensity factor for a crack along the throat of a weld propagating from the edge of the stiffener plate to the web. Note that the same maximum stress value due to bending (S_{RE}) is used for the second phase of the life calculations. The number of cycles for the second phase will also be calculated from Equation 2.5 by substitution of 2.8. Numerical integration of Equation 2.5 after substitution of Equations 2.8, 2.9, and 2.10 results in a function of a/L . This function can be approximated by the following polynomial for an a/L interval between 0.05 and 0.5. Note that 0.5 represents cracking of the entire weld at the tension side and the value of crack size a in this equation should be greater than $0.05L$ and smaller than $0.5L$.

$$N_2 = \frac{1}{C} \left(S_{RE} \frac{t_p}{1.41H} \sqrt{\pi} \right)^{-3} \left[-373 \left(\frac{a}{L} \right)^4 + 523 \left(\frac{a}{L} \right)^3 - 278 \left(\frac{a}{L} \right)^2 + 70 \left(\frac{a}{L} \right) - 2.52 \right] \quad (\text{Eqn.2.11})$$

where N_2 is the number of cycles for the second phase of the crack propagation. In order to further simplify Equation 2.11, it can be assumed that Phase I creates a crack size of about $L/20$, and Phase II starts with this initial crack size and increases to $L/2$. The reduction in the length of the weld due to web gap is neglected, since it does not have a considerable effect on Equation 2.9. After a crack length greater than $0.3L$, the expression in brackets approaches 5. If these values are substituted into Equation 2.11 as the initial and final crack lengths, Equation 2.9 can be simplified to

$$N_2 = \frac{2.5}{C} \left(\frac{H}{t_p} \right)^3 S_{RE}^{-3} \quad (\text{Eqn.2.12})$$

Equation 2.12 enables calculation of the second phase of life in terms of cycles as a function of the bending maximum stress in the vertical direction. The total fatigue life N_T is approximated by

$$N_T = N_1 + N_2 \quad (\text{Eqn.2.13})$$

Fatigue life of the stiffener plate-top flange connection of the I-65 Bridge over Ridge Road was calculated using the relationships developed in this section. There might be variations in the dimensions for other bridges, but the fatigue life calculations conducted using the properties of the I-65 Bridge over Ridge Road are expected to be representative of typical S-N relationships for a cruciform detail subjected to in-plane bending. The geometry dependent parameters used in Equations 2.7 and 2.12 are as follows for the I-65 Bridge over Ridge Road:

Weld height, $H = 0.25$ in

Plate thickness, $t_p = 0.50$ in

Consequently the S-N relationship can be approximated from Equations 2.5, 2.10 and 2.11:

$$N_T = 0.0933 \frac{S_{RE}^{-3}}{C} + 0.3125 \frac{S_{RE}^{-3}}{C}$$

The first part of Equation 2.13 covers the first phase of life, and the second part of the equation corresponds to the second phase of the total life. For a C value of 3.6×10^{-10} , the S-N relationship for this example can be estimated as

$$N = 11.3 \times 10^8 S_{RE}^{-3} \quad (\text{Eqn. 2.14})$$

The constant term of Equation 2.14 (11.3×10^8) is almost identical to the Category E Detail constant (11×10^8) of AASHTO (2004). Therefore, a crack emanating from the weld root can be classified as a Category E Detail (AASHTO, 2004) and the fatigue life and the endurance limit may be approximated accordingly. For other plate thicknesses and weld sizes the S-N relationship can be different. However, considering that Equation 2.12 was derived with a conservative approach, the Category E Detail classification can be regarded as a lower bound for stiffener plates subjected to in-plane bending. Variability of the dimensions is expected to have an effect less significant than that of the scatter of the fatigue data. On the other hand, it can be possible to obtain a fatigue response worse than a Category E Detail depending on the geometry.

The fatigue life of the stiffener plate-top flange connection for the I-65 Bridge over Ridge Road was estimated as 70 years for an effective stress range of 3 ksi. The effective stress obtained experimentally was lower than the maximum stress of 5 ksi generated by the fatigue truck (Table 2.32). In fatigue life calculations, it was assumed that the strains recorded for eight days were representative of the past and future load spectrums. Since the bridge was in service for 5 years, the remaining fatigue life of the stiffener detail was estimated as 65 years with a conservative approach.

Initiation of a crack at the weld toe is a less critical condition than weld cracking at the weld root. Hence, this mode of failure was considered as a Category C detail by Frank and Fisher (1979). If the following relationship is satisfied for a given cruciform detail (Figure 2.152), it can be stated that the mode of failure will be weld toe cracking (Frank and Fisher, 1979):

$$\left(\frac{0.71 - 0.65 \frac{2a_i}{t_p} + 0.79 \frac{H}{t_p}}{1.10 t_p^{1/6}} \right) \geq 1 \quad (\text{Eqn. 2.15})$$

where $2a_i$ equals the plate thickness (t_p) minus two times the amount of penetration as shown in Figure 2.152. If the relationship given by Equation 2.15 is fulfilled by having an adequate weld size or penetration, then the S-N relationships and endurance limit for Category C detail can be used in life and endurance limit calculations. Conservatively, the effect of stress gradient for weld root failure is not considered. Category C classification can be adopted as a lower bound for a cruciform joint subjected to in-plane bending.

A simple method for predicting the fatigue life assessment of a stiffener plate connected to the top flange of a plate girder and the cross-frame bracing has been presented. Two modes of failure were identified: weld root failure and weld toe failure. Available literature and approximate analysis conducted indicates that weld root failure classifies the cruciform joint subjected to in-plane bending as a Category E Detail, whereas the weld toe failure is governed by the Category C criteria. Identification of the failure mode is achieved by an Equation 2.15 developed by Frank and Fisher (1979). Due to absence of the experimental data for cruciform joints subjected to in-plane bending, a conservative approach has been developed in this study. Testing of typical details subjected to in-plane bending may give a higher fatigue life. However, the approach presented herein can be used as a safe method for fatigue life calculations.

There has not been any documented failures at details similar to the one investigated here. This can be in part due to conservative designs, load limits on bridges, and conservative girder distribution factors. If the beneficial effects of secondary members are considered in the load rating of a bridge, the secondary element performance becomes as important as that of the primary members. Furthermore, losing the connectivity between the top flange and the vertical stiffener plate may induce high stresses in the web gap region which may lead to out-of-plane cracking of the web.

2.9.3. EFFECTS OF PERIODIC OVERLOADS

FEA analyses of the investigated steel bridges have shown that superloads generate stresses 2 to 9 times higher than those for the fatigue truck, which is expected to generate stresses comparable to the effective stress for fatigue prone details. Since the number of superloads would be limited throughout the life-span of a bridge, the large load cycles due to superloads would be followed by many cycles of effective stress. Therefore, low-cycle fatigue behavior is not expected to govern unless a bridge component is subjected to several consecutive overload cycles and ductility of the section decreases as a result of this. Furthermore, Ferreira et al. (1998) concluded that S-N relationships developed for high-cycle fatigue assessment of cruciform welded joints can also be applied in the low-cycle fatigue range. However, the effects of overloads can be approached in two ways. The overload cycles can either be considered as a part of the variable amplitude load spectrum, or the load interaction effects due to overloads can be taken into account through a nonlinear damage accumulation model.

Effects of periodic overloads on the fatigue life of steel components have been studied extensively by several researchers in the past. Skorupa (1999) summarized many notable studies on the load interaction effects during crack growth under variable amplitude loading. Plastic straining at the crack tip and plasticity-induced crack closure were the main concepts presented as the reason of the load interaction effects in Skorupa's literature review (1999).

Load interaction effects in specimens subjected to variable amplitude loading including overload cycles have been observed in the tests conducted by Skorupa et al. (1999) with central crack tension specimens made of structural steel. As the magnitude of the overload increased, then the amount of the crack retardation under periodic single overloads increased.

Fatigue behavior of actual bridge components subjected to periodic overload cycles may be more complicated than central crack tension specimens due to the presence of residual stresses. The fatigue behavior of a stiffener detail subjected to periodic overloads has been studied extensively by Abtahi et al. (1976). Abtahi et al. (1976) applied cyclic load and periodic overloads to stiffener plates, which were classified as Category C details. The overload stress range was 1.67 times the live load stress range. The specimens were loaded up to 75% of the yield stress for the overload cycles. Fatigue tests of the stiffener plates indicated that periodic overloads spaced at 10^3 cycles and farther apart had beneficial effects on the fatigue life. However, overload cycles spaced every 100 cycles or less decreased the fatigue life. It was also concluded that the increased overload ratio increased the fatigue life further. Therefore, it can be concluded that the superloads are not expected to have a negative effect on the fatigue life of typical steel bridges. Also, the superloads are not expected to generate tensile residual stresses as a result of high compressive stresses in fatigue details of the investigated bridges. Since the tensile residual stresses at the weld toe due to welding can be as high as the yield stress, the compressive overload should be approximately twice the yield stress in order to introduce tensile residual stresses. This type of loading may not be possible because of capacity limitations.

Abtahi et al. (1976) also concluded that “similar overload effects on welded structural details with a more severe stress concentration, such as the cover plate end weld, may be anticipated.” Since fatigue response of the diaphragm-to-beam connection was found to be comparable to those of the Category C and D details by Barth and Bowman (2002) with respect to the test results, overload response of the connection can also be expected to be comparable to those of the Category C and D details. Details with lower stress concentrations were not expected to experience any delay effects according to Abtahi et al. (1976).

The ratio of the maximum stress range due to a superload truck to the stress range due to regular traffic, which was represented by the fatigue truck, was significantly

higher than the ratio of 1.67 used in Abtahi et al.'s experiments (1976). Moreover, the stress ranges due to superloads were approaching or exceeding the yield stress for the heavier superload trucks in this study. Although the data by Abtahi et al. (1976) does not cover overload stress ranges such as those due to overloads, from their conclusions and the general trend of experimental data, it can be concluded that superloads spaced at 1000 or more cycles of effective stress can still be beneficial. Due to the presence of the residual stresses, the actual stresses in the hot spot might be quite different than the predicted values both in Abtahi et al.'s specimens and in an actual bridge subjected to superloads. Requiring a certain number of days (with respect to average daily truck traffic) between passages of heavy superloads can be useful for preventing fatigue damage due to superloads. Abtahi et al. (1976) also investigated the frequency of overloads for the I-695 Highway in Maryland. In 1974, one out of 40,000 trucks traveling on this highway was classified as an overload. Frequency of overloads in actual traffic was relatively low according to their observation. The weigh-in-motion data on the route including the considered bridge can be used to estimate the number of days between heavy superload passages to prevent possible damage due to superloads.

Although it has been shown that periodic single overloads may not have a negative effect on fatigue life, passage of a superload truck can lead to brittle fracture of a weld which might be already damaged due to regular traffic. Diaphragms and their connections or similar bridge components should be inspected regularly in order to prevent failure. Furthermore, diaphragms should be checked during or after a superload passage. If a diaphragm were to fall off from a bridge, it could cause serious damage to the vehicles, property, or people underneath the bridge.

The variable amplitude fatigue life of bridges subjected to superloads can be approximated with an approach consistent with the current provisions (AASHTO, 2004). The effect of overload cycles due to superloads can be taken into account using Miner's Rule without considering the beneficial effects of superloads. This approach was used to evaluate the effects of overloads on fatigue life of steel bridges by Dicleli and Bruneau

(1995), Mohammadi and Polepeddi (2000) and Wang et al. (2005). Therefore, a limited number of superload crossings is not expected to significantly alter the anticipated effective stress level or the corresponding overall fatigue life.

2.10. FINDINGS AND CONCLUSIONS

2.10.1. FINDINGS FOR THE I-65 BRIDGE OVER RIDGE ROAD

Through rigorous finite element and experimental analyses, several observations regarding superloads in general and one specific case study bridge have been developed. The preliminary FEA provided a number of observations related to the positioning of superloads on bridges. This FEA suggested that the transverse location (lane of travel) of the superload did not significantly affect the lateral load distribution or the maximum girder or secondary member stresses. This trend is thought to be due to a number of factors, the first of which is the considerable width of the selected bridge, which proved to effectively distribute load laterally. A second factor is that the exterior girders were of the same stiffness as the interior girders. With the contribution of the parapets fully composite with the deck, the exterior girders were able to laterally distribute load with the same magnitude as interior girders.

The load test along with a comparison of various FEMs also provided a number of relevant observations. Perhaps the most significant conclusion was that the fully integral abutments provided a nearly rigid support condition for live loads. It is estimated that the integral abutments provide between 80 and 100% rigidity under live loads. This is a significant finding, since integral abutment bridges are typically designed as having no rigidity under live loads. The load test also confirmed the FEA-predicted stress concentration in the web stiffener plates at the toe of the weld connecting to the girder top flange. It was observed however, that the FEA underestimated the magnitude of this

stress concentration. In addition, the load test demonstrated a lack of significant stress concentration at the bottom web gap region. The parapets were also shown to contribute to the load carrying capacity of the bridge. The Load Test also indicated that the cross frame members, in addition to the deck, contributed to the lateral load distribution.

Finite element analyses were used to evaluate the I-65 bridge for various superload configurations. Based upon these analyses, several observations were made. First of all, longitudinal girder stresses and girder deflections are not expected to be a major concern for superloads. For the Maximum Truck, live load longitudinal bending moment stresses were predicted to be less than 15% of the girder's design yield strength. Deflections for the Maximum Truck were found to be less than $L/1570$. The FEM does, however, suggest that Superload Group D and the Maximum Truck have the potential for inducing yielding of the cross frame members. The horizontal cross frame angles were found to be the most heavily stressed members. The FEM also suggests that the Superload Group D, Maximum Truck, and possibly Superload Group C trucks could generate stress concentrations at the top of the web stiffener plates which may cause yielding of the stiffener plates. The lower web gap region was not found to be a critical location even when subjected to Superloads. This is primarily due to the bolted detail that connects the bottom of the web stiffener plate to the girder bottom flange. The bolted connection was found to minimize stress concentrations and out of plane web bending in this region.

From the measured truck traffic, a number of events which caused stresses comparable to predicted stresses caused by superload trucks were observed. As many as 34 events were recorded that caused longitudinal girder stresses (> 2.0 ksi) comparable to those predicted to be caused by Superload Groups A, B, and C. A fatigue analysis of the web stiffener plate to girder top flange connection detail was also conducted based on measured truck traffic. Although three events were recorded to have caused stresses in excess of 6.0 ksi, it was predicted that this detail will not likely experience fatigue

damage due to normal truck traffic, even with demands comparable to those expected for some superload trucks.

Although cross frame members are not explicitly designed to participate in lateral live load distribution, they do contribute to load transfer. Due to the differential displacements of adjacent girders, cross frame members transfer forces to the adjacent girders. Although these cross frames can be beneficial in increasing the lateral load distribution, they can also impart unintended stresses on other more critical load carrying members, or become overstressed themselves. The heaviest and most compact Superloads, like Group D Superloads, which can cause large differential displacements between adjacent girders, are expected to generate stresses in the horizontal cross frame members in excess of the design yield strength (36 ksi) of the angles.

The differential displacement of adjacent girders and the presence of cross frame members with similar details to those of this selected bridge also generate stress concentrations in the web stiffener plates. As the girders deform under live load, the girder top flange aligns with the transverse curvature of the deck. The girder web and bottom flange attempt to align themselves with the girder top flange, but are partially restrained by the cross frame members. This restraint provided by the cross frame members causes a “pinching action” (compressive) between the girder top flange and web stiffener plate on one side of the girder web. On the other side of the girder web, a “prying action” (tensile) of nearly equal magnitude is caused between the girder top flange and web stiffener plate. The heaviest and most compact Superloads, similar to the Group D Superloads and possibly Group C Superloads, are expected to generate stress concentrations in the web stiffener plates in excess of the yield strength (36 ksi) of the plates. The weld was found to have adequate strength so as not to fracture under such stressing.

2.10.2. GENERAL FINDINGS AND CONCLUSIONS

Analysis and instrumentation of two steel bridges and a comprehensive literature survey led to the following findings and conclusions:

- The instrumented plate girder bridge behaved as a nearly fully fixed span because of the abutment fixity.
- The GDFs computed using the AASHTO (2004) equation gave approximately 100% conservative results for the instrumented bridge when one lane was loaded.
- Both steel bridges have enough capacity to accommodate the 500-kip Group D Superload or similar trucks. However, the demand due to the 824-kip Maximum Truck exceeded the capacities of both bridges. However, the rating for the Maximum Truck was 0.9 for both of the investigated bridges. This indicates that a 740-kip truck which has the same axle configuration with the Maximum Truck generates demands lower the capacities of the investigated steel bridges. Serviceability requirements of AASHTO LRFR (2003) are met for all superload trucks. Yielding of the beams or girders is not expected for the superloads included in the database.
- Diaphragms and the top web gap are high stress regions in bridges with diaphragm-beam connections. Yielding of the top and bottom of the top web gap region or stressing the diaphragms and the diaphragm-beam connection is possible for some superload trucks.
- Superload trucks generated stresses 3 to 10 times the stresses generated by the Fatigue Truck (AASHTO, 2004) in the secondary members and web gap of the investigated bridge with diaphragm-beam connection.
- Cross-frames, diaphragms and web gap are possible damage locations for the investigated steel bridges. Cracking of the welds connecting the stiffener plate to the top flange or the diaphragm to beam web, and cracking in the beam web in bridges with diaphragm-beam connections are probable damage mechanisms.

- Depending on the geometry, typical stiffener plate top flange connections can be classified as Category E if fatigue cracks develop in the weld rather than the weld toe. This results in finite life.
- Overload effects due to superloads are not expected to have negative effects on the fatigue life of steel bridges provided that they do not cause brittle fracture. Overload cycles may rather have a beneficial effect on the fatigue life of details if they are spaced 1,000 or more cycles apart. Furthermore, the beneficial effects of overload cycles due to superloads can be conservatively neglected and these cycles can be incorporated into the stress spectrum to obtain the life consumed by a superload passage. Miner's Rule of Cumulative Damage can be used in the evaluation of damage.

Table 2.1 Superload truck data

Loading	GVW (kip)	Weight on Bridge (kip)	Total Number of Axles	Number of Axles on Bridge	Total Truck Length	Maximum Moment (kip-ft)
HS-20	72	72	3	3	28'-0"	1850
Toll Road Loading	90	90	5	5	28'-0"	2280
Group A	201	201	9	9	80'-2"	3250
Group B	247	130	13	7	125'-8"	2940
Group C	366	193	17	9	152'-8"	4260
Group D*	500	500	16	16	95'-0"	9230
Max Truck*	824	778	21	19	127'-7"	14020

* Trucks have 8 tires per axle

Table 2.2 Measured and calculated deflections of the Beam 3 due to the test truck

Loading	Deflection (in)		Error (%)
	FEA	Measurement	
LC #2A	-0.243	-0.224	8.5
LC #3A	-0.244	-0.213	14.6

Table 2.3 Longitudinal flange stresses of Beam 3 for LC #3A loading

	Long. Flange Stress (ksi)		Error (%)
	FEA	Measurement	
Top Flange	-0.19	-0.33	42.4
Bottom Flange	2.20	N/A	N/A

Table 2.4 Longitudinal flange stresses of Beam 4 for LC #3A loading

	Long. Flange Stress (ksi)		Error (%)
	FEA	Measurement	
Top Flange	-0.21	-0.15	40.0
Bottom Flange	2.59	2.63	1.5

Table 2.5 Flange stresses of Diaphragm 2

	Loading	Long. Flange Stress (ksi)		Error (%)
		FEA	Measurement	
Top Flange	LC #2A	-0.40	0.08	600.0
Bottom Flange	LC #3A	1.60	1.09	46.8

Table 2.6 Flange stresses of Diaphragm 3

	Loading	Long. Flange Stress (ksi)		Error (%)
		FEA	Measurement	
Top Flange	LC #3A	-1.80	-1.29	39.5
Bottom Flange	LC #3A	3.50	2.87	22.0

Table 2.7 Flange stresses of Diaphragm 4

	Loading	Long. Flange Stress (ksi)		Error (%)
		FEA	Measurement	
Top Flange	LC #2A	-1.50	-0.85	76.5
Bottom Flange	LC #3A	3.05	2.37	28.7

Table 2.8 Girder distribution factors computed from load test data based on measured top and bottom flange stresses

	Top Flange	Bottom Flange
Load Case 1	0.17	0.29
Load Case 2	0.14	0.37
Load Case 3	0.22	0.38
Load Case 4	0.25	0.35
Load Case 5	0.22	0.35

Table 2.9 Rotation measurements from inclinometers located at North Abutment

	Angle 1 Girder 7 (radians)	Angle 2 Abutment Wall (radians)	Relative Rotation (radians)
Load Case 1	0.0005	0.0003	0.0002
Load Case 2	0.0003	0.0001	0.0002
Load Case 3	0.0003	0.0001	0.0002
Load Case 4	0.0004	0.0002	0.0002
Load Case 5	0.0005	0.0003	0.0002

Table 2.10 Longitudinal flange stresses at the abutment (Section A) for Load Case 1
(FEM#2)

LC1	Stress from FEA (ksi)		Stress from Measurement (ksi)	
Girder Number	Bottom Flange	Top Flange	Bottom Flange	Top Flange
6	-3.0	0.4	-2.3	0.0
7	-3.4	0.4	-2.2	0.0
8	-2.3	0.7	-1.2	0.0

Table 2.11 Longitudinal flange stresses at the abutment (Section A) for Load Case 2
(FEM#2)

LC2	Stress from FEA (ksi)		Stress from Measurement (ksi)	
Girder Number	Bottom Flange	Top Flange	Bottom Flange	Top Flange
6	-0.45	0.15	-0.7	-0.1
7	-1.3	0.3	-0.5	0.0
8	-2.6	0.78	-2.3	0.0

Table 2.12 Vertical stresses in the stiffener plate for Load Case 1 (FEM#2)

LC1	Stress from FEA (ksi)		Stress from Measurement (ksi)	
Stiffener Plate Location	Vertical	Horizontal	Vertical	Horizontal
East	0.7	0.0	2.2	-
West	-0.7	0.3	-3.4	0.0

Table 2.13 Vertical stresses in the stiffener plate for Load Case 2 (FEM#2)

LC2	Stress from FEA (ksi)		Stress from Measurement (ksi)	
Stiffener Plate Location	Vertical	Horizontal	Vertical	Horizontal
East	-1.7	0.5	-6.1	-
West	1.9	-0.5	2.1	-0.6

Table 2.14 Vertical stresses in the bottom web gap for Load Case 1 (FEM#2)

LC1	Stress from FEA (ksi)		Stress from Measurement (ksi)	
Web Gap Location	Vertical	Horizontal	Vertical	Horizontal
East	-0.3	1.9	-0.7	2.0
West	-0.3	1.9	-0.4	2.7

Table 2.15 Stresses in the bottom web gap for Load Case 2 (FEM#2)

LC2	Stress from FEA (ksi)		Stress from Measurement (ksi)	
Web Gap Location	Vertical	Horizontal	Vertical	Horizontal
East	0.2	0.7	-0.3	0.7
West	0.1	0.7	-0.2	0.7

Table 2.16 Gross vehicle weights of the superload trucks used in the analysis

Truck	Weight (kips)
Max Truck	824
Group D	500
Group C	348
Group B	248
Group A	201

Table 2.17 GDFs for the US-52 Bridge due to trucks positioned in the right lane

Truck	GDF
Max Truck	0.17
Group D	0.18
Group C	0.18
Group B	0.17
Group A	0.21
HL-93	0.20

Table 2.18 Strength II Limit State positive moment check for the US-52 Bridge

Truck	$\gamma_{LL}M_{LL}$ (ft-k)	$\phi M_n - \gamma_{DL}M_{DL}$ (ft-k)	Rating
Max Truck	1320	2756	2.1
Group D	982		2.8
Group C	467		5.9
Group B	322		8.6
Group A	399		6.9
HL-93	383		7.2

Table 2.19 Strength II Limit State shear check for the US-52 Bridge

Truck	$\gamma_{LL}V_{LL}$ (kips)	$\phi V_n - \gamma_{DL}V_{DL}$ (kips)	Rating
Max Truck	246	550	2.2
Group D	172	539	3.1
Group C	84		6.4
Group B	88		6.1
Group A	126		4.3
HL-93	55		9.8

Table 2.20 Strength II Limit State negative moment check for the US-52 Bridge

Truck	$\gamma_{LL}M_{LL}$ (ft-k)	$\phi M_n - \gamma_{DL}M_{DL}$ (ft-k)	Rating
Max Truck	1783	1623	0.9
Group D	1190		1.4
Group C	671	2321	3.5
Group B	524		4.4
Group A	468		5.0
HL-93	362		6.4

Table 2.21 Service II Limit State composite flange stress check for the US-52 Bridge

Truck	σ_{LL} (ksi)	$\sigma_{all}-\sigma_{DL}$ (ksi)	Rating
Max Truck	19	33	1.7
Group D	15		2.2
Group C	6		5.1
Group B	5		6.9
Group A	5		6.1
HL-93	5		6.9

Table 2.22 Service II Limit State noncomposite flange stress check for the US-52 Bridge

Truck	σ_{LL} (ksi)	$\sigma_{all}-\sigma_{DL}$ (ksi)	Rating
Max Truck	28	27	1.0
Group D	19		1.4
Group C	9	27	3.2
Group B	7		4.1
Group A	6		4.5
HL-93	5		5.9

Table 2.23 GDFs for the I-65 Bridge over Ridge Road due to trucks positioned at the center of the bridge (from FEM#2)

Truck	GDF
Max Truck	0.21
Group D	0.22
Group C	0.22
Group B	0.23
Group A	0.25
HL-93	0.24

Table 2.24 Strength II Limit State positive moment check for the I-65 Bridge
over Ridge Road

Truck	$\gamma_{LL}M_{LL}$ (ft-k)	$\phi M_n - \gamma_{DL}M_{DL}$ (ft-k)	Rating
Max Truck	3352	10794	3.2
Group D	2342		4.6
Group C	1076		10.0
Group B	784		13.8
Group A	937		11.5
HL-93	807		13.4

Table 2.25 Strength II Limit State shear check for the I-65 Bridge over Ridge Road

Truck	$\gamma_{LL}V_{LL}$ (kips)	$\phi V_n - \gamma_{DL}V_{DL}$ (kips)	Rating
Max Truck	375	321	0.9
Group D	256		1.3
Group C	119		2.7
Group B	95		3.4
Group A	106		3.0
HL-93	80		4.0

Table 2.26 Service II Limit State composite flange stress check for the I-65 Bridge over
Ridge Road

Truck	σ_{LL} (ksi)	$\sigma_{all} - \sigma_{DL}$ (ksi)	Rating
Max Truck	12	28	2.3
Group D	7		3.8
Group C	4		7.3
Group B	3		10.8
Group A	3		10.6
HL-93	3		9.1

Table 2.27 .Maximum simply supported moment and stress comparisons for Superload Trucks and Hypothetical Class-9 Trucks.

Loading	GVW (kip)	Weight on Bridge (kip)	Total Number of Axles	Number of Axles on Bridge	Total Truck Length	Maximum Simply Supported Moment (kip-ft)	Maximum Long-Stress (ksi)	GDF
HS-20	72	72	3	3	28'-0"	1850	1.4	0.39
Toll Road Loading	90	90	5	5	28'-0"	2280	1.6	0.36
Group A	201	201	9	9	80'-2"	3250	1.9	0.35
Group B	247	130	13	7	125'-8"	2940	1.9	0.35
Group C	366	193	17	9	152'-8"	4260	2.6	0.34
Group D	500	500	16	16	95'-0"	9230	5.0	0.33
Max Truck	824	778	21	19	127'-7"	14020	7.1	0.32
1-Hyp. Class 9	218	218	5	5	60'-0"	4260	2.6	0.39
2-Hyp. Class 9 Reduced	109	218	10	10	60'-0"	4260	2.3	0.34
Load Case 1	171	171	6	6	44'-1"	4130	2.4	0.33
Load Case 2	171	171	6	6	44'-6"	4100	2.5	0.37
Load Case 3	171	171	12	12	101'-3"	2710	1.3	0.33
Load Case 4	171	171	12	12	101'-3"	2710	1.3	0.33
Load Case 5	171	171	6	6	55'-4"	3482	1.6	0.30

Table 2.28 Maximum stresses in the diaphragms of the US-52 Bridge

Truck	Long. Stress (ksi)
Max Truck	34
Group D	24
Group C	10
Group B	8
Group A	7
Fatigue Truck	4

Table 2.29 Maximum stresses in the top of the top web gap of the US-52 Bridge

Truck	Vertical Stress (ksi)
Max Truck	130
Group D	90
Group C	38
Group B	29
Group A	26
Fatigue Truck	14

Table 2.30 Maximum stresses in the bottom of the top web gap of the US-52 Bridge

Truck	Vertical Stress (ksi)
Max Truck	63
Group D	45
Group C	19
Group B	15
Group A	12
Fatigue Truck	7

Table 2.31 Maximum relative beam deflections and vertical web gap stresses of the US-52 Bridge computed using Jajich and Schultz's (2001) equation and FEA

Truck	Δd (in)	σ_{wg} (ksi)	
		Jacich and Schultz (2001)	FEA
Max Truck	0.67	39	63
Group D	0.47	27	45
Group C	0.20	11	19
Group B	0.15	9	15
Group A	0.13	7	12
Fatigue Truck	0.07	4	7

Table 2.32 Maximum stresses in the stiffener plates of the I-65 Bridge

Truck	Vertical Stress (ksi)	
	Tension	Compression
Max Truck	35	-34
Group D	22	-21
Group C	12	-11
Group B	9	-10
Group A	8	-9
Fatigue Truck	5	-5

Table 2.33 Maximum stresses in the top web gap

Truck	Vertical Stress (ksi)
Max Truck	3.5
Group D	2.2
Group C	1.2
Group B	0.8
Group A	0.7
Fatigue Truck	0.5

Table 2.34 Maximum stresses in the horizontal angle

Truck	Long. Stress (ksi)
Max Truck	30
Group D	20
Group C	10
Group B	7
Group A	7
Fatigue Truck	4

US-52 Bridge

I-65 Bridge

Figure 2.1 Cross-sections of the US-52 and I-65 Bridges **Figure 2.2** General view of the first and second steel spans of the US-52 Bridge

Figure 2.3 Diaphragms of the US-52 Bridge

Figure 2.4 Fixed support over the second pier of the US-52 Bridge

Figure 2.5 Rocker bearings at the beginning of the first steel span of the US-52 Bridge

Figure 2.6 I-65 Bridge over Ridge Road

Figure 2.7 I-65 over Ridge Road Bridge overhang and parapet

Figure 2.8 Framing plan of the I-65 Bridge over Ridge Road

Figure 2.9 Cross-frame of the I-65 Bridge over Ridge Road

Figure 2.10 Plate girders, cross-frames and integral end abutment of the I-65 Bridge

Figure 2.11 Cross-section of the FEM of the I-65 Bridge over Ridge Road

Figure 2.12 FEM of the US-52 Bridge (end of the 5th span)

Figure 2.13 Load patches for the Maximum Superload Truck on the US-52 Bridge

Figure 2.14 Beams and diaphragms of the US-52 Bridge

Figure 2.15 SAP2000 Model top view (FEM#1)

Figure 2.16 SAP2000 Model bottom view (FEM#1)

Figure 2.17 SAP2000 Model showing girder-to-deck connections with Rigid Link Elements (FEM#1)

Figure 2.18 Maximum Truck load patches at Exit Lane (FEM#1)

Figure 2.19 Maximum Truck load patches at Right Travel Lane (FEM#1)

Figure 2.20 Maximum Truck load patches at Center of Bridge (FEM#1)

Figure 2.21 Symmetric FEM of the I-65 Bridge over Ridge Road (FEM#2)

Figure 2.22 FEM#2 of the I-65 Bridge over Ridge Road shown with real thicknesses

Figure 2.23 Underside of the FEM#2 of the I-65 Bridge over Ridge Road

Figure 2.24 Top web gap

Figure 2.25 Bottom web gap

Figure 2.26 Numbering of the beams and diaphragms of the US-52 Bridge

Figure 2.27 Section C-Girders 6, 7, and 8 (18 gages)

Figure 2.28 Section C-Girders 1-5, 9, and 10 (21 gages)

Figure 2.29 Typical Section C-Girders 6, 7, and 8

Figure 2.30 Typical Section C-Girder 1-5, 9, and 10

Figure 2.31 Strain gages on the East side parapet at Section C of the I-65 Bridge over Ridge Road

Figure 2.32 Surface gages on the East side parapet at Section C of the I-65 Bridge over Ridge Road

Figure 2.33 Strain gages on cross-frames between Girders 6 and 7 of the I-65 Bridge over Ridge Road

Figure 2.34 Section B-Typical Cross Frame diagonal gage

Figure 2.35 Section B-Typical Cross Frame horizontal gages

Figure 2.36 Section B-Typical Cross Frame horizontal gages

— Strain Gage

Figure 2.37 Girder 7 stiffener plate strain gages at Section B of the I-65 Bridge over Ridge Road

Figure 2.38 Strain gages on the stiffener plate of Girder 7 of the I-65 Bridge over Ridge Road (Wood et al., 2006)

Figure 2.39 Typical Section B-Girder 7 transverse stiffener plate gages

Figure 2.40 Typical Section B-Girder 7 transverse stiffener plate gages

Figure 2.41 Section B-Girder 7 (cross section) lower web gap strain gages (4 gages)

Figure 2.42 Section B-Girder 7 (elevation) lower web gap strain gages (4 gages)

Figure 2.43 Typical Section B-Girder 7 web gap gages

Figure 2.44 Typical Section B-Girder 7 web gap gages

Figure 2.45 Section A-Girder 7
(6 gages)

Figure 2.46 Section A-Girders 6
and 8 (6 gages)

Figure 2.47 Section D-Girder 7
(6 gages)

Figure 2.48 Section A-Girder 7 gages

Figure 2.49 Section D-Girder 7 gages

Figure 2.50 Cross Section at face of North abutment

Figure 2.51 Elevation View of Girder 7 at North abutment

Truck 1
GVW: 42.9kip
Gage Width: 7'-10"

Truck 2
GVW: 44.94kip
Gage Width: 7'-10"

Truck 3
GVW: 41.82kip
Gage Width: 7'-10"

Truck 4
GVW: 42.3kip
Gage Width: 7'-10"

Figure 2.52 Axle configurations of the test trucks

Figure 2.53 Load Case 1 schematic

Figure 2.54 Test trucks positioned on the I-65 Bridge for Load Case 1

Figure 2.55 Load Case 2 schematic

Figure 2.56 Test trucks positioned on the I-65 Bridge for Load Case 2

Figure 2.57 Load Case 3 schematic

Figure 2.58 Test trucks positioned on the I-65 Bridge for Load Case 3

Figure 2.59 Load Case 4 schematic

Figure 2.60 Load Case 5 schematic

Figure 2.61 Test trucks positioned on the I-65 Bridge for Load Case 5

Figure 2.62 Load Case 6 schematic

Figure 2.63 DL-101 digital level from Topcon for deflection measurements

Figure 2.64 DL-101 digital level rod for deflection measurements

Figure 2.65 Locations of deflection measurements

Figure 2.66 Acorn Nut used to mark locations of deflection measurements

LOAD CASE 1

LOAD CASE 2 **Figure 2.69** Stress distribution at midspan of Girder 7 for Load Case 2**LOAD CASE 3** **Figure 2.70** Stress distribution at midspan of Girder 7 for Load Case 3**LOAD CASE 4** **Figure 2.71** Stress distribution at midspan of Girder 7 for Load Case 4

LOAD CASE 5 **Figure 2.72** Stress distribution at midspan of Girder 7 for Load Case 5**LOAD CASE 1** **Figure 2.73** Stress distribution at Section A of Girder 7 for Load Case 1**LOAD CASE 2** **Figure 2.74** Stress distribution at Section A of Girder 7 for Load Case 2

LOAD CASE 3 **Figure 2.75** Stress distribution at Section A of Girder 7 for Load Case 3**LOAD CASE 4** **Figure 2.76** Stress distribution at Section A of Girder 7 for Load Case 4**LOAD CASE 5** **Figure 2.77** Stress distribution at Section A of Girder 7 for Load Case 5

LOAD CASE 1 **Figure 2.78** Stress distribution at Section D of Girder 7 for Load Case 1**LOAD CASE 2** **Figure 2.79** Stress distribution at Section D of Girder 7 for Load Case 2**LOAD CASE 3** **Figure 2.80** Stress distribution at Section D of Girder 7 for Load Case 3

LOAD CASE 4 **Figure 2.81** Stress distribution at Section D of Girder 7 for Load Case 4**LOAD CASE 5** **Figure 2.82** Stress distribution at Section D of Girder 7 for Load Case 5

Figure 2.83 Lateral stress distribution at midspan for Load Case 1

Figure 2.84 Lateral stress distribution at midspan for Load Case 2

Figure 2.85 Lateral stress distribution at midspan for Load Case 3

Figure 2.86 Lateral stress distribution at midspan for Load Case 4

Figure 2.87 Lateral stress distribution at midspan for Load Case 5

Figure 2.88 Longitudinal stress distribution of Girder 7 for Load Case 1 (118ft span)

Figure 2.89 Longitudinal stress distribution of girder 7 for Load Case 2 (118ft span)

Figure 2.90 Cross frame member measurements

Figure 2.91 Web stiffener plate measurements

Figure 2.92 Bottom web gap measurements

Figure 2.93 Roller-Roller support condition

Figure 2.94 Pin-Pin support condition

Figure 2.95 Fixed-Fixed support condition

Figure 2.96 Load Case 1 deflection comparison.

Figure 2.97 Load Case 2 deflection comparison.

Figure 2.98 ·Load Case 3 deflection comparison.

Figure 2.99 ·Load Case 4 deflection comparison.

Figure 2.100 Load Case 5 deflection comparison.

Figure 2.101 Load Case 1-Girder 7 stress distribution at Section C

Figure 2.102 Load Case 2-Girder 7 stress distribution at Section C

Figure 2.103 Load Case 3-Girder 7 stress distribution at Section C

Figure 2.104 Load Case 4-Girder 7 stress distribution at Section C

Figure 2.105 Load Case 5-Girder 7 stress distribution at Section C

Figure 2.106 Load Case 1-Top Flange stress distribution at Section C

Figure 2.107 Load Case 1-Bottom Flange stress distribution at Section C

Figure 2.108 Load Case 2-Top Flange stress distribution at Section C

Figure 2.109 Load Case 2-Bottom Flange stress distribution at Section C

Figure 2.110 Load Case 3-Top Flange stress distribution at Section C

Figure 2.111 Load Case 3-Bottom Flange stress distribution at Section C

Figure 2.112 Load Case 4-Top Flange stress distribution at Section C

Figure 2.113 Load Case 4-Bottom Flange stress distribution at Section C

Figure 2.114 Load Case 5-Top Flange stress distribution at Section C

Figure 2.115 Load Case 5-Bottom Flange stress distribution at Section C

Figure 2.116 Load Case 1-Girder 7 stress distribution at Section A

Figure 2.117 Load Case 1-Girder 7 stress distribution at Section D

Figure 2.118 Comparison of cross frame angle stresses

Figure 2.119 Eccentricity of axial force for cross frame angle stress computations

Figure 2.120 Bottom flange stresses at the midspan of the I-65 Bridge for Load Case 1 (FEM#2)

Figure 2.121 Bottom flange stresses at the midspan of the I-65 Bridge for Load Case 2 (FEM#2)

Figure 2.122 Deflections at the midspan of the I-65 Bridge for Load Case 1 (FEM#2)

Figure 2.123 Deflections at the midspan of the I-65 Bridge for Load Case 2 (FEM#2)

Figure 2.124 Cross-frame stresses for Load Cases 1 and 2 (FEM#2)

Figure 2.125 Distribution of the bending moments across the US-52 Bridge

Figure 2.126 Distribution of the bottom flange stresses for the US-52 Bridge

Figure 2.127 Distribution of the beam deflections for the US-52 Bridge

Figure 2.128 Mid span deflections for design trucks and Load Test configurations (FEM#1) for I-65 Bridge

Figure 2.129 Mid span deflections for Superload Groups (FEM#1) for I-65 Bridge

Figure 2.130 Maximum girder stresses for design trucks and Load Test configurations (FEM#1) for I-65 Bridge

Figure 2.131 Maximum girder stresses for Superload Groups (FEM#1)

Figure 2.132 Distribution of the bending moments across the I-65 Bridge over Ridge Road (FEM#2)

Figure 2.133 Distribution of the bottom flange stresses for the I-65 Bridge over Ridge Road (FEM#2)

Figure 2.134 Distribution of the beam deflections for the I-65 Bridge over Ridge Road

Figure 2.135 Data sampling and recording with $60\mu\epsilon$ trigger value (50 Hz scan rate)

Figure 2.136 Unfiltered data event [Stress (ksi) = 0.029*Micro-Strain (µε)],
[1 second = 50 scans]

Figure 2.137 Filtered data event [Stress (ksi) = 0.029*Micro-Strain (µε)],
[1 second = 50 scans]

Figure 2.138 Stress comparison between Hypothetical Class-9 truck, measured events, and Group C Superload.

Average Class-9

Axes Weights are % of GVW
Total Length: 60'-0"
4-tires per axle.

Figure 2.139 Axle weight distribution and spacing for Average Class-9 trucks

Hypothetical Class-9

GVW: 218 Kip
Total Length: 60'-0"
4-tires per axle.

Two Hypothetical Class-9s

GVW: 109 Kip each
Total Length: 60'-0"
4-tires per axle.

Figure 2.140 Axle configurations for Hypothetical Class-9 trucks

Figure 2.141 Stress comparison between one Hypothetical Class-9 truck and two Hypothetical Class-9 trucks.

Figure 2.142 Frequency of girder stress ranges for 49 days of recorded truck traffic

Figure 2.143 Frequency of tensile web stiffener plate stress ranges for eight days of recorded truck traffic

Figure 2.144 Deflected shape of the US-52 Bridge cross-section subjected to live load (amplified 100 times)

Figure 2.145 Deflected shape of the diaphragm and stresses for the Maximum Truck

Figure 2.146 Von Mises stresses in the top and bottom of the top web gap for the Maximum Truck

Figure 2.147 Equivalent plastic strains for the Maximum Truck

Figure 2.148 Deflected shape of the I-65 Bridge over Ridge Road (amplified 200 times)

Figure 2.149 Close-up of the critical region showing the out-of-plane bending of the web

Figure 2.150 Tensile stress at the edge of the stiffener plate

Figure 2.151 Compressive stress at the edge of the stiffener plate

Figure 2.152 Cruciform joint and dimensions (Frank and Fisher, 1979)

Figure 2.153 Crack growth in the fillet weld connecting the stiffener plate to the top flange (cross-section of the stiffener connection at one side of the web is shown)

CHAPTER 3. EFFECTS OF SUPERLOADS ON PRESTRESSED CONCRETE BRIDGES

Effects of superloads on steel bridges have been discussed in Chapter 2. A similar investigation has been carried out for typical prestressed concrete (PC) bridges. Two slab-on-girder bridges have been selected from an extensive database of bridges which were expected to be traversed by heavy permit trucks. These two bridges are believed to represent typical practice for similar types of slab-on-girder bridges. The first bridge has bulb-tee girders and the second bridge has AASHTO Type-II girders. With regards to deck width, girder depth, and girder capacity they show significantly different characteristics.

In order to capture the behavior of an existing structure, both finite element analysis (FEA) and structural monitoring techniques were utilized. One of the prestressed concrete bridges was instrumented for this study to provide a better understanding of the live load response of typical prestressed concrete bridges subjected to superloads. Details of the instrumentation program and the finite element analyses are presented in the following sections. Detailed finite element models were developed, and a load test was conducted to evaluate the finite element model in addition to long term monitoring of one of the investigated bridges. Furthermore, capacities of two bridges were calculated and compared with the demand due to superloads. Possible damage mechanisms due to superload trucks were also evaluated.

There have been several studies investigating the behavior of prestressed concrete bridges. However, due to special challenges of instrumenting concrete structures, the scope of these studies was limited and strains were monitored typically at midspan only.

Predicted and measured response of a highway bridge can be significantly different. Simplifications and some design assumptions may not be consistent with the actual behavior of the bridge. Furthermore, behavior of a prestressed concrete bridge may be altered due to time dependent effects. Therefore, an instrumentation program is an essential tool for a thorough understanding of actual bridge behavior and response to superloads.

3.1. BACKGROUND

There are a limited number of comprehensive studies on measured behavior of existing prestressed concrete bridges. In almost all available studies at the time, one or more bridges were instrumented by strain gages attached to the girders at the midspan, and finite element models (FEMs) using the eccentric beam modeling technique were developed. Prestressed concrete girders and parapets were modeled with beam elements and shell elements were used for the deck typically. Behavior at the abutment and the negative moment region for continuous bridges were investigated in detail only in one particular study by Lawver et al. (2000). Some notable experimental and analytical studies on behavior of prestressed concrete bridges are presented below in chronological order. Also, two studies by Wood (1992) and Weiss (2006) on long term properties of concrete are summarized to present variability of actual properties of concrete.

Shenoy and Frantz (1991) tested 27-year old prestressed box beams removed from a deteriorated bridge. The load history of these beams was not known, but there were some signs of deterioration such as spalling and minor cracking. However, both beams showed sufficient strength and ductility when loaded up to failure. Flexural cracking occurred at about two times the 1989 AASHTO service load, and the measured strength exceeded factored loads.

Shahawy and Issa (1992) tested transversely prestressed double tee bridges in Florida. Strains and deflections were measured. The measured deflections were observed to be more than 20% lower than the analytical values obtained by finite element analysis.

Massicotte and Picard (1994) monitored a prestressed segmental box girder bridge during strengthening. Several sensors including foil type electrical strain gages, vibrating wire gages and mechanical strain gages were used. It was stated that long-term reliability of foil gages under field conditions is not certain. Vibrating wire gages were found to be more reliable and accurate than foil gages. However, high cost and low measurement frequency were stated as the main disadvantages for vibrating wire strain gages. Mechanical strain gages were observed to be reasonably accurate, but manual readings with these devices were cumbersome according to the authors. Since this study focused on thermal and long term effects, there was no information on the live load response of the bridge.

Cai et al. (1998) conducted an extensive study on non-destructive testing to evaluate the load capacity of existing bridges. Finite element analysis and non-destructive field testing were used. A detailed description of the field test program conducted by the Florida Department of Transportation was given. The test vehicles were specially designed semi trailers. The test vehicles, loaded with concrete blocks, weighed 204 kips each when fully loaded. It was concluded that the testing program provided not only accurate load ratings, but also gave other useful information on load distribution and impact factors.

Lawver et al. (2000) investigated field performance of a 3-span integral abutment bridge in Minnesota, from the construction stage through several years of service life. More than 180 sensors were installed in the girders and deck to study the bridge. Live load tests were conducted to measure the live load response. Effects of the seasonal trends were found to be higher than the effects of the live load with respect to measured response of the bridge. The live load test indicated that the integral abutment bridge

behaved as three independent spans. Behavior of the end spans were between simple and fixed support behaviors. Bending moments, calculated from strains at the end and the midspan, together with the deflection measurements, revealed that there was approximately 60% fixity at the abutments due to the rotational restraint.

Mirmiran et al. (2001) carried out an analytical study to determine the effect of positive moment reinforcement in continuity diaphragms on the performance of continuity connections. It was found that cracking of the diaphragm in the absence of positive moment connection significantly reduces continuity effect for service loads and may cause some durability concerns. However, a minimum amount of positive moment reinforcement was recommended to control cracking of the diaphragm and to prevent loss of continuity. Positive moment capacity of more than 120% of the cracking moment of the continuity diaphragm was found to be redundant. There was no significant improvement in performance beyond this moment capacity when the diaphragm was analyzed as a T-section with its web equal to the bottom flange width. It was concluded that better continuity behavior can be achieved if continuity is established at later ages of the girder.

Barr et al. (2001) investigated live load distribution factors for a continuous prestressed concrete bridge. Finite element analysis and a load test program were conducted to evaluate load distribution characteristics of the bridge. Eccentric beam modeling technique was used for the finite element model. It was found that the flexural GDFs calculated with the AASHTO LRFD (2004) equation were up to 28% greater than the GDFs computed using the finite element model calibrated with respect to the load test. Furthermore, it was concluded that the same bridge could accommodate 39% higher live loads if the finite element model GDFs were used.

Cai and Shahawy (2003) discussed critical issues for capacity rating of bridges from load tests. The significance of unreliable factors observed during load tests was noted. Factors such as bearing restraint or unintended composite action were some of

these beneficial effects. Therefore, it was recommended to remove the effect of unreliable factors from the rating calculations. Importance of engineering judgment for the interpretation of load test results was emphasized.

Chun and Shahawy (2004) evaluated the performance of six existing prestressed concrete bridges in Florida using finite element models and load test results. Strain gages were attached along the girders at the predicted critical sections. For example, the strains were monitored at the bottom flanges at the maximum moment location. An eccentric beam modeling technique was used for the developed finite element models. Diaphragms were also included in the model. The stiffening effect of the top flange on the deck was simulated by adding flange stiffening elements to the deck. These stiffening beam elements were normal to the longitudinal axis of the girders and were located in the same plane with the deck. Parameters such as bearing stiffness, diaphragm stiffness, contribution of parapets and top flange stiffening were considered as different analyses parameters, and the results of different finite element models were compared with the measurements. The combined contribution of these factors to live load resistance of the bridges was found to be a major source of the difference between the predicted and measured performance. However, it was also stated that the distribution of load among the girders was less sensitive than the magnitude of the strains.

Barr et al. (2006) evaluated the load rating capacity of a prestressed concrete bridge in New Mexico using finite element modeling, instrumentation and a live load test. The results of this study indicated that the load distribution factors calculated with respect to AASHTO (2004) were conservative compared to the calculated and measured distribution factors.

Weiss (2006) summarized elastic properties, creep and relaxation of concrete. Stress-strain response of concrete was illustrated in a figure. A line was superimposed on the stress-strain diagram for strains up to 1000 microstrains. Different moduli definitions were specified with respect to the regions of the stress-strain curve. Slope of the stress-

strain curve at the origin was defined as the initial tangent modulus. Although the techniques used to measure and estimate the elastic modulus neglect the initial part of the stress-strain curve up to 50 microstrains, this portion might be critical for low stresses obtained in a field study. There was no relationship specifically applicable for estimation of moduli in the low strain region. Estimation of elastic properties and variations in the measured modulus up to 20% were also discussed. The ACI (2005) expression for the calculation of the elastic modulus was presented as a useful relationship, but sensitivity of the elastic modulus to the modulus of the aggregate was emphasized as the major factor for differences from the calculated values.

Wood (1992) evaluated long term properties of concrete through an experimental program. Specimens at ages ranging from day 1 to 34 years were tested. After 20 years, the mean compressive strength of the specimens was 30 to 40% higher than the 28-day strengths. The modulus of elasticity was also found to be higher than the value computed using the ACI (2005) equation. However, it was still proportional to the square root of the compressive strength. Therefore, it can be concluded that a considerable increase in the compressive strength influences the elastic modulus and can result in a higher value than predicted.

The most important conclusion that can be drawn from the previous studies is the conservatism of the load distribution model of the AASHTO LRFD (2004). Another common point of the previous studies is related to the limitations of the experimental work. Most often, the bridges were instrumented for a short period of time, less than a week, during which a load test was conducted. Emphasis was given to the load distribution at the midspan. This can be attributed to the difficulties involved with reliable strain measurements on concrete surfaces. Therefore, a cost effective and reliable sensor to measure the concrete strains is needed for a comprehensive experimental study. Having more sensors on a bridge is expected to give a better understanding of the actual behavior and the effect of support conditions, continuity and effect of secondary members on the live load response of bridges. There were only a few available studies on the response of

prestressed concrete bridges subjected to superloads when this study was conducted. Hence, there is need for comprehensive research to investigate the effects of superloads.

3.2. DESCRIPTION OF THE INVESTIGATED BRIDGES

Many bridges on the routes traversed by superload trucks are expected to be slab-on-girder bridges. Therefore, two slab-on-girder bridges are investigated in order to evaluate the performance of these bridges when subjected to superloads. The first bridge, with bulb-tee girders, has a higher capacity compared to the second bridge and represents the upper end of the database with regards to girder depth and bridge width. The second bridge, with AASHTO Type-II girders, has lower capacity and is more vulnerable to damage due to vehicle loads. Properties of both bridges are presented below.

3.2.1. I-65 BRIDGE OVER US-30

The I-65 Bridge over US-30 (Figure 3.1) is located in Merrillville, Indiana. This prestressed concrete bridge was rebuilt in 2003 using the same foundation elements. It is a two-span integral abutment bridge without skew, and the span lengths are 121 ft and 119 ft. The bridge was designed as simple spans for dead load and continuous for live load. Girders were made composite with the 8-in thick deck. Specified 28-day compressive strength of concrete used in the deck was 4 ksi. Eleven, 66-in deep bulb-tee girders made of high strength concrete (8 ksi) were used for the north-bound structure. Spacing of the girders was 9 ft. The south-bound structure is carried by a separate and almost identical structure. There are two diaphragms in each span, and they are located at one third and two third points of the span approximately. Since the Northbound structure was expected to be more frequently traversed by heavy trucks, only the Northbound structure was analyzed and instrumented. A general plan and cross-section of the bridge

are presented in Figures 3.2 and 3.3, respectively. Also, design drawings, reinforcement and prestressing pattern of the I-65 Bridge over US-30 are given in Appendix B.

3.2.2. I-164 BRIDGE OVER WEINBACH AVENUE

The I-164 Bridge over Weinbach Avenue is located in Evansville, Indiana. East and Westbound traffic are carried by two separate, but identical structures. This bridge was constructed in 1992. The three span continuous bridge has seven 36-in deep AASHTO Type-II prestressed concrete girders spaced at 6.3 ft and it does not have any skew. The girders are made of concrete with a specified 28-day compressive strength of 5 ksi. The span lengths are 41.5 ft, 53 ft and 41.5 ft. The 8-in thick deck was made of 4 ksi concrete. Similar to the I-65 Bridge over US-30, this bridge was also made continuous for live load. A general plan and cross-section of the I-164 Bridge are presented in Figures 3.4 and 3.5 respectively. The design drawings of the I-164 Bridge are presented in Appendix B.

3.3. FINITE ELEMENT ANALYSIS

The prestressed concrete bridges were modeled using the FEA program SAP2000. Girders of both bridges were modeled by frame elements. Deck and parapets were modeled using shell elements. Both frame and shell elements have six degrees of freedom. This modeling technique is called the eccentric beam model, since the girders were modeled with frame elements and their eccentricity with respect to the slab as well as the composite action were enforced by rigid link elements (Figure 3.6). The diaphragms were modeled using frame elements (Figure 3.7) and their connection with the girders was expected to be capable of transferring some moment. However, since rigidity of the connection was not known initially, two finite element models, one with

rigidly connected diaphragms and one with no diaphragms, were prepared to simulate the upper and lower bound behaviors.

3.3.1. FINITE ELEMENT ANALYSIS OF THE I-65 BRIDGE OVER US-30

Support conditions, level of continuity, effects of diaphragms and actual material properties were the major unknowns which required refinement according to field measurements for the I-65 Bridge over US-30. Experience from the I-65 Bridge over Ridge Road suggests that fixed support conditions might be closer to actual behavior. However, due to the uncertainties regarding the support conditions, simple supports at the ends were also considered for the preliminary FEM as a lower bound with respect to end fixity. The rotational restraints due to the integral abutments were simulated by fixed support conditions initially, and then calibrated with respect to load test results.

For the preliminary finite element analysis, two spans of the bridge were connected to each other as if there was full continuity between the adjacent spans. However, the level of continuity was later refined with respect to field measurements. A similar approach was also adopted for the diaphragms. For the preliminary analysis, two FEMs with and without diaphragms were developed to examine the effect of diaphragms on load distribution and the diaphragms in the final FEM was modeled according to the observations from the load test. The FEM prepared for the I-65 over US-30 is presented in Figures 3.6 and 3.7.

Several superload trucks, HL-93 Loading and Toll Road Trucks were considered in the analysis. The truck loads were applied in the form of pressure patches at wheel locations as shown in Figure 3.8 for a typical superload truck. Longitudinal truck locations, determined from a influence line analysis, were chosen so that bending moments due to each truck were maximized. Superload trucks and HS-20 Truck were positioned at the center of the center lane and at the shoulder, 2 ft from the parapet, in

order to obtain the maximum and minimum girder distribution factors for single truck loading. The preliminary FEA revealed the most heavily loaded girders and diaphragms. The instrumentation plan was developed accordingly. Bending moments resisted by the composite girders when the HS-20 truck was positioned at the right and center lanes are presented in Figures 3.9 and 3.10, respectively. Note that these bending moments are obtained from the preliminary FEM which had fixed support conditions for the end abutments. Bending moments resisted by Girders 6, 7 and 8 are labeled in Figures 3.9 and 3.10.

It is evident from Figures 3.9 and 3.10 that Girders 6, 7 and 8 are the most heavily loaded girders. Also, the diaphragms between these three girders were subjected to the largest bending moment and axial force (tension) demand. The presence of diaphragms led to a better load distribution.

3.3.2. FINITE ELEMENT ANALYSIS OF THE I-164 BRIDGE OVER WEINBACH AVENUE

The finite element model of the I-164 Bridge over Weinbach Avenue was developed in a similar way as the FEM of the I-65 Bridge over US-30. Eccentric beam modeling technique was used for the FEM of the I-164 Bridge. Shell elements were used for the deck and parapets, and frame elements were used for the girders. Truck loads were applied as pressure patches. Maximum positive and negative moment locations were determined from an influence line analysis.

3.4. DESIGN AND TEST OF SENSORS FOR THE INSTRUMENTATION

Instrumentation of the I-65 Bridge over US-30 was considered as an important tool for evaluating the effects of superloads on PC bridges. Therefore, possible sensors for monitoring of the live load effects were investigated. Instrumentation of prestressed

concrete bridges requires special effort due to the nonhomogenous composition of concrete. Since it is composed of aggregates and cement paste, physical properties may vary throughout the test specimen. Although this does not affect the global behavior and stress distribution, on a local scale the stress distribution may not be as uniform as that for a steel specimen. In order to overcome this measurement difficulty, long gage lengths are preferred for monitoring strains on a concrete surface. The longer gage length can capture the average strain along the gage length. Therefore, as a rule of thumb, the gage length should be five times the diameter of the maximum aggregate size (Measurements Group, Inc., 1995).

For example, recommended uniaxial grid foil gage lengths are 2.5 and 4 in for typical concrete members. Another alternative is use of full bridge transducers for measuring strains. Since the strains are measured in an indirect way, these sensors are called transducers. A strain gage, commonly bonded to a plate also known as the base material or spring, measures the strains on the plate. However, the plate's deformation is controlled by the concrete member, since it is bonded to the member. Usually, the transducers have a long gage length in order to minimize recording locally high or low stresses which may not reflect the global stress distribution. Performance of both the foil gages and the strain transducers were investigated prior to development of the instrumentation plan for the I-65 Bridge over US-30.

3.4.1. TEST BEAM AND EXPERIMENT SETUP

In order to test the performance and installation time of strain gages and available transducers, a full size prestressed concrete beam was used. This beam was cast in the Bowen Lab of Purdue University's School of Civil Engineering. Because of some problems with the formwork, the cross-sectional properties, such as flange widths, were varying up to 25% along the span. The original span length of this AASHTO Type-I beam was 30 ft and its depth was 28 in. It was tested before for another research project

on shear resistance of beams made of high strength concrete (Hoeltke, 2005). Compressive strength of the concrete used for this beam was 10 ksi. The beam was loaded at the midspan. There was no shear reinforcement along the beam except for at the supports. Therefore the shear failure and the major cracks occurred around the load point at the midspan. The beam was broken into two pieces at the end of the test. A remaining 18 ft piece of that beam was used for testing the strain gages and transducers (Figure 3.11).

For the strain gage and transducer test, the remaining segment of the beam was positioned such that the shear cracks were not included in the 13 ft span between the supports (Figure 3.11). Hence, there were no shear cracks between Supports 1 and 2 shown in Figure 3.11. All of the shear cracks were located at the left side of Support 1. However, there were two visible cracks at the top of the beam close to the loading point (Figure 3.12). These cracks were formed when the beam failed suddenly during the previous test and collapsed to the floor. The cracks were at the compression region, and they did not extend to the bottom of the girder. The strain gages and transducers did not cross any of these cracks.

The loading ram used for the test was capable of applying 150 kips. The load point was located at 4 ft from Support 1 and 9 ft from Support 2. The beam was loaded up to 100 kips for testing the gages and transducers several times without causing any additional cracks. A linear voltage displacement transducer was also attached to the bottom of the girder at the load point for measuring the deflections. The load deflection curve of the beam is presented in Figure 3.13.

In Figure 3.13, the loading and unloading responses of the beam are distinct. The beam was not loaded and unloaded dynamically. From Figure 3.13, it can be inferred that the stiffness increases as the beam is loaded. This is believed to be a result of closing cracks in the compression region of the beam. The same type of non-linear response was also observed for the other sensors. Since the beam's cross-sectional properties were not

constant and there were two wide cracks affecting the response, it was not possible to obtain reliable analytical solutions for strains or deflections.

The beam was instrumented with three foil gages, three in-house-developed transducers and a commercial strain transducer. Locations of these sensors on the beam are presented in Figures 3.14 and 3.15, respectively. Installation procedures for these three types of sensors and their performance are discussed in the following sections.

3.4.2. FOIL TYPE GAGE

The foil type strain gages (resistance gages) considered for this study were manufactured by the Measurements Group. These gages were similar to those used to instrument the parapet of the I-65 Bridge over Ridge Road. A typical foil gage is presented in Figure 3.16. These strain gages are used to determine the deformation of the surface to which they are bonded through measured change in electrical resistance. The motivation for using foil gages in the lab environment was to compare their performance and installation times with the transducers. 2.5-in long foil gages were used for this experiment.

Installation of long foil gages to concrete surfaces requires a special procedure (Measurements Group, Inc., 1995). First of all, the surface has to be cleaned and brushed with a wire brush for removing any loose material. Then, the exposed rough surface has to be filled with a special epoxy manufacture by Measurements Group, Inc. (AE-10). After filling the voids, the epoxy coating is sanded until the surface becomes smooth and flat. The curing time of this epoxy is 6 hours at 75° F and it should be sanded when it is cured, preferably. For lower temperatures, the curing time can increase significantly. Finally, the strain gage is bonded to the finished surface with the AE-10 epoxy. This epoxy allows proper alignment of the gage, since it does not set immediately. However, constant pressure should be applied to the gage for a better bond during the curing time of

epoxy. Because of the long gage length, a sizeable normal force has to be applied to the gage area for providing adequate pressure.

Three foil gages were installed to the beam as shown in Figures 3.14 and 3.15. Two of them, denoted as G1 and G2, were at the tension zone and one of them, denoted as G3, was at the compression zone. Load versus strain response was measured at the bottom flange of the test beam at the load point and is presented in Figure 3.17.

3.4.3. IN-HOUSE DEVELOPED TRANSDUCER

In an attempt to find the best solution for the monitoring of strains on a concrete girder, an in-house transducer was developed. The main objective of the transducer design was to make a simple and low-cost sensor. Ease of installation was also a major criterion affecting the design. Basically, the transducer would be composed of a plate with a strain gage attached on it. Two holes were drilled at the ends of the rectangular plate for attaching it to the girder. Dimensions of the in-house transducer are presented in Figure 3.18. For the base material, aluminum was preferred due to its light weight and resistance to corrosion. Due to its superior material properties, Aluminum 2024 was used for the spring structure (base material). This is an aircraft quality material.

The original plan was to attach this transducer to the concrete girder by concrete strike anchors only. However, the shear friction between the washer and transducer was considered to be inadequate for resisting slip. This could cause some creep or hysteresis and affect the measurements. Therefore, epoxy between the transducer and the concrete surface was used in addition to the concrete anchors.

The smallest anchor size was selected in order to minimize the diameter and depth of the drilled hole. The length of the anchor was 1.75 in, and the diameter was 0.25 in. Minimum embedment depth was 1 in for this anchor. The installation procedure for the

concrete strike anchor starts with drilling a hole of sufficient depth. The hole diameter should be the same as the diameter of the anchor being installed. Then, a washer and nut are placed on the anchor. With the nut, washer and set-pin in place, the anchor is inserted through the material to be fastened to the concrete base material. Finally, using a hammer, the pin is set by several sharp strikes on the head of the pin until the pin is flush with the top of the anchor. The concrete strike anchor selected for this test was capable of resisting 900 lbs of pull-out and 1600 lbs of shear forces according to the manufacturer's specifications. The high pull-out strength of the anchors ensures that the transducer will not fall from the instrumented girder.

Application of epoxy between the transducer and the concrete surface was regarded as a solution to possible problems with slip. The surface preparation is similar to but simpler than that of foil gages. Light sanding and a layer of AE-10 were sufficient. Then the transducer was bonded to the concrete surface with AE-10 and the concrete anchors were installed at the same time. The total installation time for a typical homemade transducer is estimated as 1 hour in the field. A photograph of an installed transducer is shown in Figure 3.19.

Using concrete anchors and drilling holes on the girders were not expected to damage the girders, since the embedment depth was 1 inch at most and the minimum cover was at least 1 inch for the girders of the I-65 Bridge over US-30. Also, concrete is not a notch sensitive material (ACI, 1997). Any embedment depth more than 1 inch could result in an interference with the reinforcement. Use of larger concrete anchors may have eliminated the need for epoxy. However, larger anchors would also require larger and deeper holes on the girder.

For the strain gage to be used on the transducer, a full bridge transducer class strain gage manufactured by the Vishay-Measurements Group, Inc. (Figure 3.20) was found to be the best option. The main advantage of this strain gage is having four active arms which amplify the signal. This results in reduced noise levels. Two of the arms

measure the longitudinal strains and the other two measure the transverse strains. In-house developed transducers were manufactured and tested prior to installation on the bridge. Strain gages were attached to base plate using AE-10. Then, lead wires were soldered to the gage and the gage was covered with a protective coating to protect it from moisture (see Figure 3.19). The transducers were tested in bending rather than axial load, because significantly higher strains could be obtained even when a 0.5 lb force was applied. Each transducer was clamped at one end and loaded as a cantilever beam at the other end. The measured strain value was compared with the calculated value. Measured values did not differ from the calculated values more than 5% and all transducers indicated approximately the same value.

The load versus strain response recorded by the Transducer 2 (T2) at the bottom flange of the girder (Figure 3.15) is presented in Figure 3.21. When the data was recorded with the CR9000 data logger, a noise magnitude less than 1 microstrain could be achieved. The amount of noise was much higher for the foil gages due to the lower output signal level compared to that of the full bridge strain gage used in the transducer.

Repeatability of the measurements was tested by loading the beam up to 100 kips and unloading several times. The load versus strain curve followed the same path on all tests for the homemade transducer and the foil gages. The measurements were consistent for each test. In addition, the beam was unloaded suddenly a few times to simulate the dynamic loading and effects of impact. It was concluded that the transducer response was not affected by the rate of loading.

3.4.4. EVALUATION OF OVERALL PERFORMANCE OF GAGES

As mentioned earlier, cross-sectional properties of the beam were not constant throughout its length and there were two apparent cracks in the compression region. However, with the approximate cross-sectional dimensions of an uncracked section, a

beam line analysis was performed and the strains at strain gage and transducer locations were computed. It should be kept in mind that these analytical results are only rough estimates of the existing strains and are not considered to be exact. The measured and computed strains are presented in Table 3.1 for a maximum load level of 100 kips. In Table 3.1, positive strains indicate tension and negative strains indicate compression.

Due to the uncertainties involved in the numerical prediction and measurement of strains, performance of the strain gages and the transducers are believed to be acceptable. When the two alternatives for this instrumentation task are considered, the in-house transducer appears to be the best option. Since it is a custom built device, it combines all desired features, such as fast installation, high sensitivity, low noise to signal ratio, low cost and an acceptable accuracy. The foil type gages could be used. However, due to the difficulties encountered in their installation process, the number of gages would need to be limited. In particular, the 6-hour constant pressure application period makes the foil gages less desirable for a field application. Furthermore, deep voids, up to quarter inch, on the girders of the I-65 Bridge over US-30 would significantly increase the effort needed for surface preparation.

3.5. INSTRUMENTATION PLAN

The instrumentation plan developed for the I-65 Bridge over US-30 will be described in this section. The objectives of the instrumentation program were to obtain the load distribution characteristics of the bridge and to monitor strains at critical spots. Deflections were also monitored during the load test. Locations of interest were based on the preliminary structural analysis. The CR9000 data logger manufactured by Campbell Scientific, which was used for data acquisition, is capable of recording strains at a high scanning rate (more than 1000 Hz) for all of its 80 channels. This data logger is similar to the CR5000 data loggers used at the Ridge Road Bridge. The main differences are the number of channels and scanning rate.

Monitoring strains at the maximum moment section, pier, diaphragm and the abutment was expected to give significant data about the behavior of the instrumented bridge. Installing strain gages close to the maximum moment region enables determination of the lateral distribution of loads in the positive moment region as well as measurement of the maximum tensile strains. Severity of the tensile stresses generated at the precompressed tension region of the girders can be an important damage parameter. Another location of interest is the negative moment region. For example, different girder distribution factors might be expected near the piers than at midspan. Furthermore, performance of the continuity joint can be determined by measuring strains in the negative moment region. Effects of cast-in-place concrete diaphragms on the load distribution and the loads resisted by the diaphragms are the other issues for which field data would be valuable. Also, the amount of fixity due to the integral abutments was evaluated in order to fully understand the behavior of structure.

Due to the issues related to traffic control, the second span (Span B) was selected for instrumentation. General behavior of the first span (Span A) was expected to be similar to that of Span B, although impact effects are undoubtedly different. The sections which were instrumented are presented in Figure 3.22. Most of the load across the bridge is resisted by girders 6, 7 and 8 for regular traffic, since they are located below the center and right lanes (Figure 3.3). Therefore these girders were heavily instrumented. Distribution of transducers for each cross-section will be presented in the following sections.

3.5.1. SECTION A

Section A—shown in Figure 3.22—is located at a girder depth (66 in) away from the face of the abutment. Six transducers were installed to Girder 7 as shown in Figure 3.23. Two transducers were installed on the bottom flange, two transducers at the mid-depth of the girder and two transducers on the top flange. Three transducers through the

depth of the girder enable determination of the strain distribution. Hence, the bending moment resisted by the girder can be inferred. Two transducers at each vertical location improve redundancy of the measurement. The transducers at the bottom flange were installed as close as possible to the bottom fiber.

3.5.2. SECTION B

At section B—shown in Figure 3.22—the diaphragms were instrumented. Since the most heavily loaded members were expected to be Girders 6, 7 and 8 as shown in Figures 3.9 and 3.10, these girders will deflect more than the others. Therefore, interaction of these girders with the adjacent diaphragms was expected to generate the greatest diaphragm stresses for trucks traveling in the center and right lanes. Both ends of the diaphragms were instrumented at the top and bottom as shown in Figure 3.24. Note that two transducers were installed at each location; one was placed at the north side and the other was placed at the south side of the diaphragm to increase redundancy. A total of 16 transducers were attached to the diaphragms.

3.5.3. SECTION C

Section C—shown in Figure 3.22—is located at the middle of the second span. The location of maximum positive moment may shift with respect to the amount of fixity at Abutment 2. However, this location is expected to be close to the midspan if fully-fixed support conditions were encountered. In case of a simple support it would shift to a location around 50 ft from the middle pier. This location is not far away from Section C at 60 ft. Furthermore, the bending moment distribution is not expected to vary considerably in 10 ft. Therefore, the lateral load distribution in the positive moment region can be obtained from Section C. Girders 4 to 11 were instrumented at Section C. Girders 1, 2 and 3 were not instrumented, since these girders carry the ramp lanes and do

not share a sizeable portion of the load due to trucks traveling on northbound I-65. For Girder 7, three vertical locations along the depth of the girder were selected. For the other instrumented girders, transducers were attached to the bottom flanges only. Instrumentation details for Girders 4 to 11 are presented in Figures 3.25, 3.26 and 3.27.

3.5.4. SECTIONS D AND E

Strains at Sections D and E—shown in Figure 3.22—were monitored to capture the negative moment behavior. Section D is located a girder depth (66 in) away from the pier, whereas Section E is located half depth (33 in) away from the pier. At Section D, Girders 5 to 9 were instrumented, since they resist most of the load because of carrying center and right lanes. At section E, only Girder 7 was instrumented with 2 transducers. Locations of transducers at Sections D and E are presented in Figures 3.28 to 3.31.

3.5.5. DEFLECTION MEASUREMENTS

Deflections of Girders 6 to 10 were measured during the load test. Traffic control on US-30 was provided by INDOT for measurement of deflections. The set-up for deflection measurements was prepared right before the load test and was required to be disassembled after the load test. Therefore, long term deflection measurements could not be obtained. Potentiometers (Figure 3.32) were used to measure the vertical deflections at the midspan. Steel angles were attached to the bottom of the girders approximately three feet away from the midspan transducers, towards the abutment. Then, thin steel wires were used to connect the potentiometers to the angles as shown in Figure 3.33. Potentiometers were positioned right under the angles with a plumb. A separate data acquisition system and power provided from a bucket truck's generator were used for the potentiometers. Deflection measurements were recorded electronically. Hence, no additional time was needed for the measurements and dynamic measurements could be

obtained. Compared to the surveying equipment used on the Ridge Road Bridge, potentiometers were more efficient in terms of measurement time needed during the load test.

3.5.6. INSTRUMENTATION SUMMARY

The instrumentation plan consisted of 56 in-house developed transducers and five potentiometers. All sensors were installed as planned. A photograph of a typical transducer installed on the I-65 Bridge over US-30 is presented in Figure 3.34. Transducer locations and the number of transducers at each location are summarized in Table 3.2. Furthermore, photos of the transducers installed at Sections A, B, C and D are presented in Figures 3.35 to 3.38.

Wiring of the transducers and power supply for the data logger were other issues which could be as critical as the selection of sensors. For the wiring of the transducers, four-conductor, shielded wires were used. A 5 ft or longer wire was soldered to the strain gage of the transducer and the transducer was covered with weatherproof protection. The wires coming from the data logger were spliced with the wires of the transducers in the field. The effects of lead-wire and splice resistance on the measurements were checked by splicing the wires in the lab. The same measurement was obtained for all cases regardless of length of the lead wire or having a splice.

Since it was not be possible to use the bottom flanges of the PC girders in the same manner as the flanges of steel girders, the wires were carried by conduit hangers attached to the deck pans. The screws used for connecting the deck pans to each other and to the angles at the edges of the girder were also used, for attaching the conduit hangers to the deck pans. Therefore, no additional drilling or screws were needed.

Power for the data logger was supplied by solar panels. The main advantages of this system were elimination of the cost of running electricity lines and the reusability of solar panels. Furthermore, possible electrical noise due to the alternating current was avoided. For the solar powered system, the main components were the solar panels, charge controller and marine batteries. The solar panels generated electricity, and the charge control unit regulated the electricity generated by the solar panels. The charge controller unit measures the battery voltage and charges them accordingly. Four deep discharge batteries were used to power the data logger. While charged during the day time, they supplied power for the system 24 hours a day. The batteries had enough capacity to power the system for several days without any sunshine.

3.6. LIVE LOAD TEST

Live load tests have been a very effective way of measuring the performance of bridges. Moreover, a load test gives a good understanding about behavior of the investigated bridge. In particular, the effects of the secondary members and level of continuity or abutment fixity can be quantified with experimental data. Measured GDFs have been found to be lower than the analytically predicted values for almost all available studies in the literature. This can be attributed to the limitations of the analytical tools to predict the true load sharing response of bridge girders and/or beams. Therefore, load tests were used to assist in predicting the behavior and response of the I-65 Bridge over US-30 when subjected to superloads.

Traffic control on the day of the load test was provided by INDOT. Northbound I-65 could not be closed to traffic completely, but lane restrictions on all travel lanes and direction of traffic to the ramp lane provided adequate space for positioning the test trucks. All three travel lanes and a ramp lane were available during the load test. Only one ramp lane was occupied by the traffic. For the static load cases, measurements were recorded only when there was no traffic on the bridge.

The load test of the I-65 Bridge over US-30 was designed to simulate superload trucks in addition to simulating regular trucks. Four loaded dump trucks provided by INDOT were used for the load test. Dump trucks were loaded with sand in order to maximize their weight. The axle weights of each tandem truck were measured the day before the load test. Dimensions of the dump trucks are provided in Figure 3.39. Axle weights of the test trucks are provided in Table 3.3. Six static, two quasistatic, and two dynamic load cases were planned for simulating superload trucks and to investigate the bridge response. Locations of the test trucks on the bridge were marked before the load test. The load cases are described in the following sections.

3.6.1. LOAD CASES 1 TO 4

Load cases 1 to 3 were designed to maximize the positive moment at the middle of the instrumented span. Therefore, the transducers at the middle of the span would be recording the maximum possible strains generated by the test trucks. A two by two formation of the trucks was positioned at the middle of the instrumented span (Figure 3.40). However, this two by two formation was moved in the transverse direction to measure the load distribution for different truck positions. The two by two formation was necessary for obtaining the bridge response when subjected to side by side trucks and maximizing the load on Section C (midspan).

For Load Case 1, two of the test trucks were positioned back to back at the shoulder. They were approximately 2 ft away from the parapet. The other trucks were parked right next to these two trucks with an approximate distance of 2 ft in between. Arrangement of the test trucks for Load Case 1 is presented in Figures 3.41 and 3.42. This load case was expected to be the most demanding among all load cases, since the exterior girder was heavily loaded. Furthermore, a similar truck configuration was used for the generation of AASHTO LRFD (2004) GDFs. Hence, the GDF obtained for Load Case 1 can be compared with the GDF calculated with respect to AASHTO (2004).

For Load Cases 2 (Figure 3.43) and 3 (Figure 3.44), the same truck arrangement was moved across the bridge width one lane at each time until the trucks were centered at the middle of the bridge. Load Case 3 created a severe loading for the heavily instrumented girders 6, 7 and 8. Therefore, this load case had a particular importance, and its results were used for the refinement of the FEM.

Load Case 4 was very similar to Load Case 3 in terms of transverse positioning, but the longitudinal location was different than that of Load Case 3. For Load Case 4, the trucks were centered on top of the instrumented diaphragm (Section B, Figure 3.22) in order to create a higher demand on the diaphragms compared to that from Load Cases 1 to 3.

3.6.2. LOAD CASES 5 AND 6

Load Cases 5 and 6 were different than Load Cases 1 to 4 with respect to transverse formation of the test trucks. Trucks were lined up rather than forming a two by two configuration. This enabled simulation of typical superload trucks. For Load Cases 5 and 6, test trucks were centered on Girder 7. Also, the longitudinal alignments of trucks were different. For Load Case 5, all four test trucks were lined up at the second span of the bridge as shown in Figures 3.45 and 3.46. This formation generated a load case which resembled typical superload trucks. Similarly, Load Case 6 was anticipated to produce the maximum negative bending moment with the available test trucks as well as simulating a superload truck with distantly spaced heavy axle groups. Two trucks were positioned back to back at the middle of each span as shown in Figures 3.47 and 3.48.

3.6.3. LOAD CASES 7 TO 10

Load Cases 7 to 10 were quasistatic and dynamic load cases. The objectives of these load cases were to load the bridge dynamically and to estimate the impact factor. For Load Cases 7 and 8, two trucks side by side traversed the bridge in the center and right lanes. However, speeds of the trucks were different for each load case. Trucks were traveling at crawl speed for Load Case 7 and traveling as fast as possible for Load Case 8. Each truck traveled in the same lane as before for both load cases.

Load Cases 9 and 10 were similar to Load Cases 7 and 8 in terms of truck speed, but only one truck was used for Load Cases 9 and 10. The truck used for Load Cases 9 and 10 traversed the bridge at crawling and faster speeds. The test truck was centered over Girder 7 with following the lane marker between center and right lanes. This line was right above Girder 7 (Figure 3.3).

3.7. REFINEMENT OF THE FINITE ELEMENT MODEL

The live-load test of the I-65 Bridge over US-30 was conducted as described above. Results of this load test were then used for refining the FEM and evaluating the performance of the bridge. Abutment fixity, level of continuity, actual stiffness and contribution of the diaphragms were the main issues which were resolved by refinement with respect to measured data. Results of Load Case 3 have been used as a point of reference.

3.7.1. ABUTMENT FIXITY AND CONTINUITY

Abutment fixity could not be calculated directly from the midspan deflections and strains, unlike for the single span I-65 Bridge over Ridge Road. The main reason for this

was the difference with the unsymmetrical boundary conditions at the ends of the span. There were two unknowns: first, fixity of the abutment, and second, the level of continuity. Therefore, each unknown was determined separately. Also, the number of relationships between the known and unknown parameters had to be increased by including the other load cases in the calibration process.

The ratios of the estimated bending moments at the ends to the moment at the midspan were found to be independent of the factors affecting the load distribution. Therefore, the FEM was calibrated to match these ratios. Distribution of strains over the height of Girder 7 at the ends and the middle of the girder are presented in Figures 3.49 to 3.51 for Load Case 3. The locations of the neutral axis on these strain distribution curves varied as much as 6 in with respect to calculated neutral axis location. This might be due to the axial force effects and cracking of the deck at the ends or differences in material properties. If it is assumed that the same neutral axis location should be observed along the span, it may shift up or down with respect to the magnitude of the axial force acting on the cross-section. The slopes of the best fit lines in these figures are related to the magnitude of the bending moments. This can mathematically be expressed with the following equation for a linearly elastic plane section:

$$\kappa = \frac{\varepsilon}{y} \quad (\text{Eqn. 3.1})$$

where κ denotes curvature, and ε is the strain at a distance “ y ” from the neutral axis. Curvature at the ends and the middle of the instrumented span can be determined from Figures 3.49 to 3.51. Furthermore, the following relationship can be used to determine the flexural bending moment (M) at a section,

$$M = EI \kappa \quad (\text{Eqn. 3.2})$$

where E is the elastic modulus and I is the moment of inertia. With this approach, the bending moment at a section can be estimated using the calculated curvature value.

For Girder 7, measurements from all 6 transducers at a typical section were used for the computation of curvature. This was a unique approach, since almost all PC bridge studies in the literature have relied on the bottom flange strains only. For the previous studies, the bottom flange strain and section modulus was used to compute the bending moment. Having several transducers across the girder depth improves redundancy and eliminates problems due to differences between the computed and actual neutral axis locations or the axial force effects.

Calculated curvature values for Girder 7 are presented in Table 3.4. Note that only results from Load Case 3 are presented in this table, since the maximum strains in Girder 7 were due to this load case. In Table 3.4, negative curvature indicates positive moment and positive curvature indicate negative moment. Ratios of curvatures at the ends to the curvature at the midspan were the main parameters used for calibration of the FEM. Also, the elastic modulus and moment of inertia values were assumed to be constant along the girder length for calibration purposes. According to Table 3.4, it can be said that the bending moments at the pier and abutment are approximately 19% and 27% of the bending moment measured at the midspan. These ratios were approximated during the calibration process by trial and error. The rotational fixity at the abutment was about 25% of the fixed support condition, and the continuity between the spans was very limited. Behavior of the I-65 Bridge over US-30 was close to that of two independent spans.

Figure 3.52 shows the side view of a typical composite girder of the FEM of the I-65 Bridge over US-30 between the pier and the abutment. Roller type supports were assigned to the bottom of the girders. Two rollers were assigned to the bottom of the girders over the pier to simulate the bearings, and spring elements were assigned to simulate the rotational restraint due to elastomeric bearings. However, effects of elastomeric bearings were negligible because of obtaining low stiffness coefficients when

AASHTO (2004) provisions were applied. For the abutment, rotational springs were assigned to the centroid of each composite girder.

In addition to 3D FEA runs, a simple beam model was used for the trial and error process during calibration. A rotational spring was assigned to each composite girder to represent the abutment fixity. Overall abutment fixity was achieved by assigning a constraint to all nodes connected to the elements (shells and frame) at the end of a typical composite beam (Figure 3.52). Stiffness of each spring element per composite girder was estimated as 600,000 ft-k/rad. The same spring stiffness was used for both abutments.

For the connection between spans (continuity), a different approach was employed. Rigidity of the connection between the spans was adjusted to give the measured level of continuity. According to the measurements, the bridge was not behaving as continuous. However, the spans were not entirely independent. Since lack of contact at the bottom of the girders was a major reason for the lower than expected level of continuity, rigidity of the connection between the spans was adjusted by modifying the stiffness of the frame elements over the pier. The moments of inertia of the frame elements over the pier (Figure 3.52) were reduced to 1.6% of the moment of inertia of the girders. The slab was assumed to have the same properties over the pier. Possible cracking effects were neglected and the stresses in the slab were not reduced due to cracking to obtain conservative values.

3.7.2. MATERIAL PROPERTIES

The discrepancies between the measured and calculated strains and deflections raised some concerns regarding the material properties used in the analysis. Compressive strength and elastic modulus of concrete typically differ from the specified values (Weiss, 2006). Elastic modulus of concrete can be 20% higher or lower than the specified value (ACI, 2005). This may affect the response of a structure significantly. For example,

measured strains might be 20% higher and deflections could be 20% lower with respect to calculated values if the elastic modulus is 20% higher than the specified value. In an attempt to determine the actual elastic modulus of the I-65 Bridge over US-30, a Fast Fourier Transformation (FFT) analysis was conducted. FFT of the measured strains when the bridge was excited due to regular traffic enabled calculation of the spectral distribution of vibrations. The natural period of vibration inferred from measured data was then compared with the value obtained from the FEM. The eigenvalue analysis was conducted with the FEM which had refined support conditions and level of continuity.

A record which includes only strains at the bottom flanges of girders at the midspan is presented in Figure 3.53. Not all measured events indicated free vibration of the bridge afterwards. However, this record had significant free vibration data, especially for the encircled portion after 5 seconds, which shows harmonic motion of the bridge. All girders go up and down at the same time. Furthermore, magnitude of the strains is almost identical for all bottom flanges. This indicates that mode 1 (Figure 3.54) dominated the free vibration response. FFT analysis also confirmed that. There was only one peak, and it was at 3.5 Hz. Therefore, it can be concluded that the fundamental period of vibration for the I-65 Bridge over US-30 is 0.286 seconds. The FEM without any modifications to material properties gave a fundamental period of vibration of 0.320 seconds. The ratio of the experimentally determined period of vibration to the analytically determined period of vibration can be used to infer the actual stiffness of the girders. The fundamental period of vibration for a single degree of freedom system can be calculated with the following equation:

$$T = 2\pi \sqrt{\frac{m}{k}} \quad (\text{Eqn. 3.3})$$

where, T is the fundamental period of vibration, m is the mass and k is the stiffness of the system. If the bridge structure is considered as a single degree of freedom system for the

first mode, the ratio of the two period values gives the ratio of stiffness properties, as shown below, assuming that calculated mass is close to actual mass:

$$\frac{T_{calc.}}{T_{meas.}} = \frac{2\pi\sqrt{\frac{m}{k_{calc.}}}}{2\pi\sqrt{\frac{m}{k_{meas.}}}} \Rightarrow \frac{T_{calc.}}{T_{meas.}} = \sqrt{\frac{k_{meas.}}{k_{calc.}}} \quad (\text{Eqn 3.4})$$

where $T_{calc.}$ and $T_{meas.}$ are the calculated and measured periods, respectively. Measured and calculated stiffness values are denoted by $k_{meas.}$ and $k_{calc.}$, respectively. Since flexural stiffness is directly proportional to the elastic modulus, and the sectional properties such as the area and moment of inertia are somewhat less variable than the material properties, the following relationship can be achieved for this bridge:

$$\frac{E_{meas.}}{E_{calc.}} = \left(\frac{T_{calc.}}{T_{meas.}} \right)^2 \approx 1.3 \quad (\text{Eqn. 3.5})$$

in which, $E_{meas.}$ stands for the experimental elastic modulus and $E_{calc.}$ denotes the elastic modulus calculated according to ACI (2005) equation. Equation 3.5 indicates that the anticipated elastic modulus value is approximately 1.3 times the computed value, which was 5098 ksi for the girders of the I-65 Bridge over US-30. Having a higher than computed elastic modulus might be due to several factors including variability in concrete strength and elastic modulus. Furthermore, the small strains on the order of 60 microstrains at most may have some effect. For larger strains, the anticipated elastic modulus can be closer to the calculated value, but for small strain response the anticipated modulus might be quite different than the calculated value. Low strain response is not included in elastic modulus measurements and is subject to equipment sensitivity problems (Weiss, 2006). The equation used to approximate elastic modulus of concrete was based on experimental data and the experimental data was obtained according to ASTM C 469. Considering that the ASTM standard specifies a chord

modulus between the 50 microstrain as the lower point and 40% of the compressive strength as the upper point, it should not be unexpected to anticipate higher moduli values for lower strains. Therefore, computed deflections and strains were adjusted accordingly. It was assumed that the same material constant modification can also be applied to the deck.

3.7.3. CONTRIBUTION OF DIAPHRAGMS

The contribution of diaphragms was investigated by attaching transducers to the diaphragms. The highest strains in the diaphragms were obtained for Load Cases 4 and 5. Measured and calculated values are compared in Tables 3.5 and 3.6 for Load Cases 4 and 5, respectively. Note that there were two transducers at each location, one on the north side of the diaphragm and the other at the south side. In Tables 3.5 and 3.6, transducer locations with respect to the nearby girder are presented. Positive stress indicates tension and negative stress indicates compression. The elastic modulus of diaphragms, which were made of 4000 psi concrete, was scaled 1.3 times to be consistent with the aforementioned assumptions regarding the material properties. Signs of the stresses in Tables 3.5 and 3.6 indicate that the diaphragms are subjected to bending and axial forces. Furthermore, the diaphragms follow the deflected shape of the deck.

Although there are some noticeable strains at the diaphragms, measured values are significantly lower than analytically predicted values from the FEM with rigidly connected diaphragms. The transducers at the east and west sides of Girder 7 were the ones highly stressed in the FEA, since Girder 7 was heavily loaded, and its deflection with respect to neighboring girders was causing stretching and bending of the diaphragms. However, the measured tensile stresses at the bottom of the diaphragms were between 5 and 10% of the calculated value. On the other hand, the compressive stresses at the tops of the diaphragm were closer to the analytical values. This suggests that diaphragms are resisting the live load up to a certain extent, but their connection with the

girders is not close to a rigid connection. There is some contact at the top, but the dowel bar connection at the bottom is not capable of transmitting any significant force. It can even be stated that contribution of diaphragms to live load resistance is very limited and it behaves like a cracked beam, which has a reduced stiffness. Therefore, the diaphragms can be neglected in the analysis. Limited contribution of diaphragms to load distribution is believed to be a result of a low strength tensile connection and construction practices. Having only four inserts (two at the top and two at the bottom) to connect the diaphragms to the girders limits the diaphragm contribution.

3.7.4. TORSIONAL RIGIDITY

Discussion regarding the abutment fixity and continuity was independent of load distribution characteristics, since this step of calibration process was focused on Girder 7. However, comparison of the analysis results with the measurements lead to another important modification to the initial FEM, which was neither mentioned in the literature nor predicted. Torsional properties of the bulb-tees used in the FEM had to be modified in order to match the measured load distribution characteristics. The torsional constant (K_T) of a solid shape was initially calculated with the following approximation given by Heins (1975):

$$K_T = \frac{A^4}{40I_r} \quad (\text{Eqn. 3.6})$$

where A is the area of the solid section and I_r is the polar moment of inertia. This approach has also been used by PCI (2004) for the design of curved bridges built with AASHTO Type I-beams or bulb-tees. The torsional constant of a bulb-tee girder was also calculated with Equation 3.6 by PCI (2004).

The value calculated according to Equation 3.6 was assigned as the torsional constant of the frame element used for modeling the girders in the FEM. After thorough refinement efforts, incorporating all variables which could affect the load distribution, it was found that this approach was too conservative for modeling composite girders using the eccentric beam modeling technique. It can also be said that torsional properties of the composite girders were not represented appropriately. The load distribution characteristics were significantly affected by this property. In the literature, this issue has not been discussed before and the difference between predicted and measured response was attributed to diaphragms or support conditions. Instrumenting the diaphragms and ends of the girders enabled determining the effect of each variable on the bridge response independently. Therefore, the torsional constant calculated according to Equation 3.6 was multiplied by 18 and the polar moment of inertia value was used in the FEM for the torsional rigidity of the frame elements representing the bulb-tee girders.

3.8. EVALUATION OF LOAD TEST DATA

The live-load test provided valuable information regarding the bridge behavior. The FEM was adjusted based upon the findings of the load test. Another important aspect of the load test was a reasonable simulation of a superload truck with four dump trucks. Therefore, effects obtained using the combination of the test trucks are expected to be similar to those of superload trucks. Strain and deflection measurements, as well as the analytically calculated values, will be presented in this section. Measured and calculated values are presented in the same figure for evaluation of the FEM's accuracy in representing the actual behavior.

3.8.1. STRESSES CALCULATED FROM STRAINS

All 56 channels of the data acquisition system were recording data for all load cases. However, only the essential measurements are presented here. Recorded strains were converted to stresses using the elastic modulus estimated from the load test. Stresses at the midspan (Section C) are presented in Figures 3.55 to 3.60 for Load Cases 1 to 6. The horizontal axis indicates the girder number and the vertical axis shows the stress in the longitudinal direction at the bottom flange of each girder. The markers in the figures show the average bottom flange stress; this stress was calculated from the measurements at both sides of the bottom flange for girders instrumented with more than one transducer at Section C.

The maximum stress calculated from the measurement was 370 psi, recorded at the bottom flange of Girder 10 due to Load Case 1. Similar load distribution characteristics were observed regardless of transverse truck position for Load Cases 1 to 3 and the maximum stress values were comparable. Figures 3.55 to 3.60 also show that the FEA results are comparable to stresses obtained from measurements. The transducers give consistent results for all cases. Furthermore, the differences between the experimental values at both sides of a girder bottom flange were around 10%, which is negligible for a field study on concrete bridge response. Therefore, the refined FEM is expected to be reliable for evaluation of the effects of superload trucks.

3.8.2. DEFLECTIONS

Deflections of Girders 6 to 10 measured with potentiometers are presented in this section. The potentiometers were zeroed prior to the load test. However, the dynamic load cases (Load Cases 7 to 10) revealed that there was a shift in zero values. A possible reason for this offset might have been movement of the wire loops connected to the angles at the bottom of the girders or thermal effects. Furthermore, vibration of the bridge

could have resulted in shifts. Therefore, zero values obtained from dynamic load cases are used for correction of the recorded deflections. Figures 3.61 to 3.66 show the measured and calculated deflections for Load Cases 1 to 6.

Almost perfect agreement between the measured and calculated deflections has been observed except for Load Cases 1 and 2. Relatively higher error margin compared to the other load cases is believed to be due to shifts in zero values. It should also be noted the maximum static deflection was recorded as 0.19 in, significantly lower than the recommended traditional limit of $L/800$, where L is the span length (AASHTO, 2004). The traditional limit for deflection of I-65 would be 1.8 in. Overall comparison of analytical and measured values confirms that the FEM represents the actual bridge reasonably well. The FEA does not only predict the maximum values accurately, but also captures the actual load distribution.

3.8.3. GIRDER DISTRIBUTION FACTORS

Load Case 1 was similar to the type of loading used for the generation AASHTO (2004) GDF equation. Although this loading position was expected to generate the maximum demand on the exterior girder, the adjacent interior girder was the most heavily loaded member. Therefore, moment GDF computed from the measured strains due to Load Case 1 can be compared with the GDF computed according to AASHTO (2004) equation for an interior girder when more than two or more lanes are loaded. If the contribution of parapets is ignored conservatively because lack of transducers on the parapet, the following equation can be used to compute the GDF for Girder 10:

$$GDF = \frac{\sigma_{\max}}{\sum_{i=1}^{11} \sigma_i} \quad (\text{Eqn. 3.7})$$

where, σ_i is the stress at the bottom flange of the i^{th} girder and $\sigma_{\max}$ is the maximum stress for Load Case 1, which is the bottom flange stress of Girder 10 (Figure 3.55). Consequently, the GDF computed from the measurements using Equation 3.7 is 0.27 when it is assumed that Girders 1, 2 and 3 do not resist any load as observed from the general trend (Figure 3.58). For a simple beam analysis which is based on influence line calculations using a single truck, a GDF value twice the experimental value is needed to be consistent with AASHTO (2004). The AASHTO (2004) GDF equations include multiple presence factors and the maximum moment generated by one truck is multiplied by the two lane GDF in order to obtain the maximum moment generated by two trucks. Therefore, a GDF value of 0.54 should be compared with the AASHTO (2004) GDF value, which was computed as 0.72. Comparison of two values indicate that AASHTO (2004) equation gives 33% higher GDF compared to experimental value for two trucks side by side.

For single lane loading GDF of an interior girder, Load Case 5 can be used as a point of reference. Equation 3.7 estimated the experimental GDF as 0.22 for Load Case 5, which had all four dump trucks lined up on Girder 7. However, the AASHTO (2004) equation for single lane loaded GDF predicts it as 0.49. Therefore, it can be concluded that the AASHTO (2004) equation for one lane loaded is more than 100% higher than the experimentally calculated GDF. For negative moment, it has not been possible to obtain an experimental GDF because of the small strains measured near the pier. Sensitivity of the strain measurements did not enable comparing small ranges of strain measurements, which were approximately between a few and 10 microstrains.

3.8.4. DYNAMIC AMPLIFICATION

Load Cases 8 and 10 were designed to simulate dynamic versions of the Load Cases 7 and 9. However, due to the presence of additional trucks on the bridge while data was recorded, it has not been possible to obtain a clear comparison between the quasi-

static and dynamic cases. Furthermore, the trucks could not reach a significant speed. Due to limited distance available for acceleration, their speed was not more than 50 mph while they were crossing the bridge. For Load Cases 9 and 10, comparison of the maximum strains and deflections at the middle of the second span of the I-65 Bridge over US-30 is presented in Figures 3.67 and 3.68, respectively. Note that only one dump truck was used for these load cases. Load Cases 7 and 8 had two dump trucks running across the bridge side by side, but there were some problems regarding the coordination of trucks. Consequently, Load Cases 7 and 8 ended up being different in terms of truck positions on the bridge, and there was some interaction with the other trucks traversing the bridge at the same time. Therefore, these two load cases are not considered in the dynamic amplification analysis.

Maximum strains recorded during the fast (Load Case 10) and crawl speed (Load Case 9) passage of trucks were almost identical (Figure 3.67). This might be attributed to the magnitude of the strains which were at most 20 microstrains. Considering that an impact factor of 1.1 would produce 22 microstrains, the difference between two cases may not have been noticeable due to the sensitivity of the measurement. Typically, 2 microstrains in a field study involving concrete structures cannot be measured accurately. Therefore, deflection measurements (Figure 3.68) are used as an alternative to strain measurements in an attempt to determine the dynamic amplification. Figure 3.68 indicates that girder deflections due to Load Case 10 are up to 20% higher compared to Load Case 9 deflections. Hence, it can be stated that an impact factor of approximately 1.2 was anticipated for a loaded dump truck. This value was lower than the dynamic amplification constant of 1.3 given in the code (AASHTO, 2004). Nevertheless, semi-trailer trucks traveling faster than the test trucks may result in a higher dynamic amplification. Furthermore, different approach slab conditions or settlement problems can cause higher impact factors for other bridges.

3.9. SUPERLOAD ANALYSIS

Superload analyses of the investigated PC bridges have been conducted in two stages. The first stage included FEA, and the second stage included capacity evaluation. The superload trucks were applied to the FEMs of the I-65 Bridge over US-30 and I-164 Bridge over Weinbach Avenue to calculate the demand due to superloads. For the I-65 Bridge over US-30, the calibrated FEM was analyzed as well as the original FEM. However, for the I-164 Bridge over Weinbach Avenue, the FEM was not calibrated due to lack of experimental data; only the model prepared according to design assumptions was considered. The most critical superload truck locations were determined by an influence line analysis and superload trucks were positioned on bridges accordingly. Furthermore, positive and negative shear and moment envelopes were generated for each bridge.

Capacities of the bridges were evaluated using the commercial software PS-Beam (2004). Girder and deck geometry, reinforcement details and material properties were the main parameters used for evaluation purposes. Flexure and shear capacity of the bridges were calculated with respect to the properties specified in the design drawings. AASHTO LRFR (2003) was used for the capacity analysis. Strength II and Service III limit states were considered in the analysis. Resistance factors of 1 and 0.9 were used for flexure and shear accordingly. The load factor of 1.15 was obtained from Table 6-6 of AASHTO-LRFR (2003). This table's provisions are applicable to special permit trucks. It is required that the permit is requested for single trip and the bridge is closed to regular traffic during the permit vehicle passage. Additionally, restricting the permit vehicle's speed to 5 mph eliminates the dynamic amplification factor according to AASHTO-LRFR (2003). Demand versus capacity evaluation of both bridges are presented in this section.

3.9.1. I-164 BRIDGE OVER WEINBACH AVENUE

Maximum negative and positive moments on I-164 due to superload trucks were calculated using a spreadsheet program as mentioned before. The maximum positive moment was found to be at the middle of the second span. The maximum negative moment and the maximum shear were obtained at the internal supports of the three span bridge. Superload trucks were positioned on the FEM to obtain the bending moments for evaluating distribution of live loads between the girders. Positive moment distributions at the middle of the I-164 Bridge due to superloads and the HL-93 loading are presented in Figure 3.69 for right lane loading position, assuming that a superload vehicle will typically travel at the center of the right lane which is approximately at the middle of the bridge in the transverse direction. Distributions of negative moments over the pier are presented in Figure 3.70 for the right lane loading position. The girder distribution factors for positive and negative moments are presented in Table 3.7. The single lane loading moment GDF calculated according to AASHTO (2004) was 0.46 for an interior girder.

The GDFs were somewhat different for different trucks. For positive moment, the GDFs were between 0.32 and 0.36, whereas they were between 0.38 and 0.40 for negative moment. From the upper bound values for the investigated bridges, it can be stated that positive and negative moment GDFs are 0.36 and 0.40. It is also notable that the AASHTO (2004) equation for GDF, which was 0.46, is conservative compared to the FEA results. Furthermore, 8-wheel axles of the Group D and the Maximum Trucks do not significantly improve the load distribution. Almost all trucks resulted in very similar distributions of moments as presented in Figures 3.69 and 3.70. However, the bending moments generated by Group D and Maximum Trucks were significantly higher than those of other trucks.

Moment and shear capacity of composite girders were computed using PS-Beam along the spans of the I-164 Bridge. The dead load effects acting on the simple spans were subtracted from the calculated moment and shear capacity values. Hence, live load

(LL) capacity of the girders has been determined and compared with the demand. Bending moments and shear generated by the superload trucks have been computed with a spreadsheet program for which the bridge was assumed to be continuous and simply supported at the ends. The maximum demand at each longitudinal location was multiplied with the GDFs. Capacity and demand comparison for superload trucks traveling at the center of the right lane of I-164 Bridge are presented in Figures 3.71, 3.72 and 3.73. The GDFs computed by FEA (Table 3.7) were used for computation of positive and negative moment demand in Figures 3.71 and 3.72, respectively. Shear demand (Figure 3.73) was computed using the AASHTO (2004) equation for calculation of single lane loading shear GDF. The shear GDF has not been determined from FEA, since its evaluation would require rigorous analysis focusing on shear, and experimental data were not available. Therefore, the shear GDF has been taken as 0.63 in the calculations.

The positive moment capacity (Figure 3.71) was not exceeded at the middle of the spans, but the positive moment envelope exceeded the capacity at the first pier location for the Maximum and Group D Superload trucks. This was due to assumed continuity and factors decreasing the level of continuity may prevent this type of demand. Furthermore, the actual positive moment capacity at the pier is expected to be higher than that computed using PS-Beam. However, it should be noted that longer and heavier trucks tend to create some positive moment in the negative moment region.

The negative moment capacity (Figure 3.72) was considerably higher than the envelopes generated by superload trucks. Shear capacity (Figure 3.73) was also higher than the shear forces generated by superload trucks, but the load envelopes were closer to the capacity than for the moment capacity curves. This indicates that shear resistance would control the rating of the I-164 Bridge. It can also be concluded that the effects of Group A, B, C Superloads and the HL-93 loading are close to each other, and that Group D and the Maximum Superload trucks result in significantly higher demands.

Another important consideration regarding the long term effects of superload trucks was the serviceability check. Maximum bottom flange stresses at the middle of the I-164 Bridge were computed for the various superloads and the HL-93 loading. It was conservatively assumed that the I-164 Bridge may not behave like a continuous structure like the I-65 Bridge over US-30. Losses and dead load effects were subtracted from initial prestressing effects to obtain the level of precompression in the bottom flange prior to live load. For the I-164 Bridge, the bottom fibers of the girders at the midspan were under compression and the magnitude of compressive stress was 444 psi after subtracting the effects of losses and dead loads. The maximum allowable tensile stress, when six times square root of the compressive strength was taken as the limit, was 424 psi. Therefore, the total range in tensile stress generated by a superload truck should not exceed 868 psi in order to prevent cracking of the girders. However, AASHTO LRFR (2003) does not have any requirements to control the bottom fiber stress during permit truck passages.

Maximum bottom fiber stresses computed for superloads and HL-93 loading at the middle of the I-164 Bridge are presented in Figure 3.74. Negative values indicate compression and positive values indicate tension in Figure 3.74. The bars representing truck stresses start from -444 psi (compressive stress). The first line drawn at zero stress shows the transition from compression to tension. All superload trucks and HL-93 loading stresses resulted in tension and exceeded the first line. Additionally, the 500-kip Group D Superload and the Maximum Truck exceeded the cracking stress, which was seven and a half times square root of the compressive strength (530 psi), as well as the AASHTO LRFD (2004) limit (424 psi) for design trucks. Although there were no critical issues regarding the flexural and shear capacities of the I-164 Bridge, the serviceability check revealed that maximum allowed girder tensile stresses can be exceeded by superload trucks. The optional service limit state evaluation is further discussed in the damage evaluation section.

3.9.2. I-65 BRIDGE OVER US-30

The capacity of the I-65 Bridge over US-30 was evaluated in a similar way as the I-164 Bridge. Maximum moment and shear values across the spans of the bridge were calculated using a spreadsheet program. The GDFs for positive and negative moments were determined from FEA for each truck. However, GDF for shear was calculated using the AASHTO (2004) equation. Trucks were positioned at the longitudinal locations generating the maximum positive and negative moments to obtain the GDFs to be used in capacity evaluation. The maximum moment envelopes along the spans of the I-65 Bridge over US-30 were multiplied with the GDFs computed from the FEM in order to obtain the live load demand envelope for an interior girder.

Results obtained from the detailed finite element analysis were used for the computation of flexure GDFs, but the moments and shear along the bridge were calculated from a simple beam analysis involving traditional design assumptions. Therefore, the bridge was assumed to be continuous for live load effects, and the abutment fixity was neglected for evaluation purposes. Superload trucks and HL-93 loading were positioned at the center of the center lane, since a superload truck would possibly travel close to the center of the bridge. Distributions of bending moments at the center of the second span due to superload trucks and HL-93 loading positioned at center of the center lane in the calibrated FEM are presented in Figure 3.75. Figure 3.75 shows that most of the bending moment is resisted by four girders closest to the wheel lines. For the center lane loading position, these were Girders 6, 7, 8 and 9. This kind of a distribution pattern was typical for interior girders. Distribution mechanisms were similar for all trucks and the demands due to Group A, B, C Superloads and HL-93 were similar, whereas Group D and the Maximum Superloads resulted in significantly higher demands.

The negative moment distribution at the pier was calculated from the refined FEM assuming limited continuity. The negative moments resisted by the girders were significantly higher than those calculated from the refined FEM when full continuity was

assumed. Distribution of negative moment among the girders is presented in Figure 3.76. Trucks were positioned at the center of the center lane in the FEM to obtain the moment distribution. The distribution in Figure 3.76 is somewhat different than the distribution in Figure 3.75. A higher distribution factor was observed for negative moment. AASHTO (2004) states that continuity can be regained at or close to the strength limit state for structures with partially effective construction joints. Therefore, the GDFs obtained from the continuous FEM were used for all capacity checks. The GDFs for negative and positive moments obtained from the refined FEM, which represents the actual behavior at service load level with respect to field measurements, are presented in Table 3.8. GDFs obtained from the FEM, which was made continuous at the pier and simply supported at the abutment in order to represent the behavior at the ultimate load level, are presented in Table 3.9. Positive moment GDF can be regarded as 0.30 as an upper bound for both cases. On the other hand, negative moment GDFs were higher for both cases and close to 0.40 for the continuous model. Also, continuity resulted in higher GDFs for the negative moment. The moment GDF according to AASHTO (2004) equation was 0.49 for an interior girder. This value is approximately 65% and 25% higher than the computed values for positive and negative moment GDFs, respectively.

The GDFs given in Table 3.9 were multiplied with the negative and positive moment envelopes generated by an influence line analysis. Capacity of a typical interior girder was computed using PS-Beam. Load and resistance factors obtained from AASHTO LRFR (2003) special permit section were used for the capacity evaluation. Effects of impact were neglected, as in the evaluation of the I-164 Bridge. Comparison of the demand and girder capacity along the bridge spans are presented in Figures 3.77 and 3.78 for positive and negative moments, respectively. A similar comparison was also prepared for shear. The shear GDF equation provided in AASHTO (2004) was used for the computation of shear demand. For an interior girder due to single lane loading, the shear GDF was 0.72. The shear forces acting on a typical interior girder are compared with the shear capacity calculated using PS-Beam in Figure 3.79.

Positive moment and shear capacity of the composite girders are well above the demand created by all superload trucks (Figures 3.77 and 3.79). However, the negative moment capacity controls the strength evaluation of the I-65 Bridge over US-30. Negative moment envelope due to the 824-kip Maximum Superload Truck approaches the capacity curve at the two-thirds point of the first span. The sudden change in the negative moment capacity along the development length of the negative moment reinforcement creates a weak zone at this location. PS-Beam follows the provisions of AASHTO (2004) for development. Therefore, the calculated negative moment capacity is expected to be somewhat different than the actual capacity. Demands due to Group A, B, C Trucks and HL-93 loadings decrease at 20 ft away from the pier, but the axle configurations of the Group D and the Maximum Truck lead to a different type of demand. The negative moment envelope due to these trucks with closely spaced axles is similar to that of a uniform loading. Therefore, it can be concluded that the negative moment demand created by Group D or similar superload trucks might be critical at a significant portion of the span, not only at the vicinity of the internal support.

Maximum tensile stresses at the bottom of the girders were also checked to compare the stresses generated by superload trucks with the cracking stress. Compressive stress at the middle of the second span was computed first. Prestressing losses and dead load stresses were then subtracted. After the losses and dead load effects, there was a net compressive stress of 627 psi in the bottom fiber. Therefore, a truck had to generate more than 627 psi tensile stress at the bottom fiber to result in any tension in the girder. The live load stresses in the girders were computed from a FEM in which the two spans of the I-65 Bridge were assumed to be simply supported. Therefore, continuity and abutment fixity were not considered in the serviceability evaluation.

The maximum allowed tensile stress in the bottom fiber of a PC girder for design trucks according to AASHTO (2004), six times square root of compressive strength, was 537 psi for the girders of the I-65 Bridge over US-30. Hence, it can be said that there are two critical stress values for the girders; first, the 627 psi stress level, which is the

threshold for tension, and the second, 1164 psi, which is the summation of precompression after the dead load and the allowable stress level. Maximum stresses in the bottom fiber of the girders in the midspan are presented in Figure 3.80. Stress corresponding to each truck starts from -627 psi compressive strain in the figure. Furthermore, the modulus of rupture, seven and a half times the square root of compressive strength, was represented in Figure 3.80 with a straight line at 671 psi.

Figure 3.80 shows that Group A and B Superload Trucks do not cause any resulting tensile strains in the girders and Group C Superload Truck may cause some negligible tensile strains. However, Group D and the Maximum Superload Trucks have the potential to cause resultant tensile strains above the modulus of rupture. These two trucks may cause cracking in the girders. Since introducing cracks into uncracked girders can impair long term durability of the girders, Group D or similar superload trucks have potential to damage the girders. It should also be noted that lack of continuity may result in higher strains due to simple span behavior. Employment of full continuity action might be possible at ultimate load levels, but the load test results showed that actual behavior might be closer to that of a simple span.

3.10. LONG TERM MONITORING OF THE I-65 BRIDGE OVER US-30

The I-65 Bridge over US-30 was monitored for approximately five months after the load test, between December 2005 and May 2006. Transducers installed on the northbound structure recorded when certain threshold strains were exceeded. The trigger system was based on the bottom flange strains measured at the midspan of Girders 6, 7 and 8. Strains were measured from all channels, but the data acquisition unit recorded only when the bottom flange strain at Girder 6, 7 or 8 exceeded the threshold value. The strain measurement at any moment was compared with the running average of 1000 measurements for the trigger channels. The scanning rate was 100 Hz. Also, 5-second pre-trigger and 5-second post-trigger periods were specified in the data acquisition

program. These resulted in a 10-second record: 5 seconds before the maximum strain instant, and 5 seconds afterwards.

For the first month of monitoring, the trigger threshold value was assigned as 45 microstrains. Having measured strains in excess of 50 microstrains during the load test, this value was selected so as to capture only strains due to heavy vehicles. However, there was only one event in that period of time which generated strains higher than 45 microstrains. Therefore, the trigger value was reduced to 35 microstrains for the following months so as to record more events. Although this threshold value was much lower than what was measured during the load test, several significant events were captured in addition to many less significant events. The strains recorded in the load test were not exceeded during a 5 month monitoring period. Moreover, no superload passages were reported by the permit division of INDOT within the monitoring period. Typical events recorded during the long term monitoring, as well as a comparison of recorded events versus the strains from the load test along with superload analysis and strain statistics will be presented in this section.

3.10.1. CHARACTERISTICS OF THE RECORDED EVENTS

Characteristics of the recorded events are summarized in this section. Load distribution and bridge response due to actual truck traffic are presented. Furthermore, a histogram of the maximum recorded strains is provided.

A single event consisted of 10 seconds of data from all channels of the data acquisition system, as mentioned. All transducers were reading values other than zero even when the bridge was not loaded. This is typical of all strain measurements. Zero values may shift due to slight variations in balancing of the Wheatstone Bridge, lead wire resistance or temperature effects. Therefore, all strain measurements were zeroed periodically from the datalogger. However, shifts in zero values due to thermal effects

occurred a few hours after zeroing. Hence, all measurements were zeroed for each recorded event. This was achieved by subtracting the average of strains at the beginning and end of the event from the entire record for each channel. Strains recorded for 10 seconds for a typical event are presented in Figure 3.81. Note the constant (zero) strain portions of the record in the pretrigger and post-trigger periods. The peak of each event was observed at about 5 seconds.

Recorded events had different characteristics in terms of load distribution and bridge response. One common feature for each event was one or more transducers indicating more than 35 microstrains in the bottom flange of Girder 6, 7 or 8. This implies that a heavy or permit truck was traveling in the right or center lane, or that two trucks side-by-side were crossing the bridge. GDFs of the recorded events were observed to fall between 0.18 and 0.30, with an average of 0.23. Therefore, it can be concluded that there were a significant number of events caused by single trucks. Response of the bridge in terms of abutment fixity, diaphragm connection and level of continuity was almost identical to that observed during the load test. No changes in the bridge response due to seasonal trends were observed.

The distribution of maximum bottom flange strains at Section C is presented in Figure 3.82. The total number of events recorded in five months was 125. The majority of these events (more than 74%) had a maximum strain level between 35 and 40 microstrain, 22% of the events were between 40 and 45 microstrain, and only 5 events had a maximum strain value greater than 45 microstrain; none of them exceeded 50 microstrain. The number of events significantly increased in the spring. More than 80 events were recorded as the construction season started and heavy loads were hauled North. There were only 20 to 30 events recorded from December to February.

The largest strains were recorded for an event in December. The truck was traveling in the right lane and Girder 6 was heavily loaded, as shown in Figure 3.83. The GDF for this event was 0.29. Therefore, only one truck was present on the bridge.

However, there was one significant difference for this event. The primary difference was in the dynamic characteristics of the bridge response. The response of the bottom flange of Girders 7 at the abutment (A7B2), midspan (C7B1) and the negative moment region (D7B2) for 10 seconds are presented in Figure 3.84. The girders were observed to be moving up and down during the event. High tensile strain peaks at the midspan were followed by relatively smaller compressive dips. This indicates that the truck was vibrating and possibly interacting dynamically with the bridge. Note that this was the only event which showed signs of severe dynamic interaction. Although not many, there were some other events after which the bridge was vibrating (Figure 3.53). However, most of the events were like the one shown in Figure 3.81, where there was a large tensile cycle, followed by limited vibration of the bridge. The reason for the highly dynamic response for vehicles using the right lane is believed to be the excessive settlement in the approach slab right before the right lane of the bridge. Settlement of the approach slab was significantly less for the center and left lanes than for the right lane. Visual inspection and comparison of the response of a car traversing the bridge in different lanes verified this difference in settlement.

3.10.2 COMPARISON OF RECORDED EVENTS WITH RESPONSE DUE TO SUPERLOADS

In this section, some of the recorded events will be compared with the load test measurements and superload analysis results. The objective of this assessment was to evaluate the severity of superload passage with respect to the actual truck traffic data. Furthermore, load distribution characteristics due to regular traffic were compared with the load test data to evaluate the validity of experimental GDFs. Superload analysis results were obtained from the calibrated FEM, and the experimentally estimated elastic modulus was used in converting measured strain to stress.

The maximum event recorded in December due to a truck traveling in the right lane is compared with the response due to a 248-kip Group B Superload Truck, HL-93

loading, and the Toll Road Truck in Figure 3.85. Another event, Event 11, which was similar to the event recorded in December in terms of stresses and load distribution, is also included in the figure. Figure 3.85 shows that the load distribution characteristics due to regular truck traffic are almost identical to those of the Group B Superload, HL-93, and the Toll Road Truck. These events and loads also had similar stress magnitudes. Moreover, dynamic interaction between the truck and the bridge, such as that for the event in December, does not affect the load distribution characteristics, and the same GDFs as for the static or quasi-static loads can be used. Dynamic interaction only affects the magnitude of strains. Hence, conducting a static analysis and applying a dynamic amplification factor can replace a dynamic analysis for the investigated bridge.

Bottom flange stresses due to a Group B Superload truck are compared with a recorded event (Event 5) and stresses due to design loads HS-20, HL-93, and the Toll Road Truck in Figure 3.86. Trucks were positioned at the center of the center lane in the FEM. The response of the I-65 Bridge over US-30 FEM to the Group B Superload Truck was almost identical to that of the HL-93 loading. Event 5, which was typical for observed events, had a lower magnitude compared to these two cases, but the distribution of load was very similar. It is also notable that the HS-20 truck generated stresses lower than the recorded event. This indicates that the recorded event was caused by a truck heavier than the HS-20.

The response of the bridge to side by side trucks is presented in Figure 3.87. Girder 7 was resisting 25% of the total load and Girders 6, 7 and 8 were all heavily loaded for recorded Event 34. Hence, this event is a typical example of strains recorded due to trucks traversing the bridge side by side. Load Case 3 (LC-3) of the load test had two trucks positioned side by side in the center and right lanes. The response of the bridge to Load Case 3 was similar to that for two Toll Road Trucks (2*TLRD) positioned in the center and right lanes. Stresses due to two HS-20 trucks and Event 34 were lower compared to the stresses measured during the load test for the two by two formation of the test trucks (Load Case 3).

Figures 3.85 to 3.87 demonstrate that the FEM of the I-65 Bridge over US-30 predicts the general behavior and load distribution accurately. The presented superload analysis can be regarded as a reliable method for predicting the response of the instrumented bridge to superloads. Furthermore, the formation of the dump trucks used for the load test mimics actual semi-trailer type trucks. In terms of GDFs, test trucks, regular traffic and the Group B Superload all lead to similar values. The same statement can also be extrapolated to the other superload trucks, and it can be concluded that variable axle spacing, truck length or more wheel lines do not improve the load distribution considerably. Moreover, comparison of the regular traffic monitoring data with the superload response indicates that regular traffic may cause stresses comparable to that of a 248-kip superload truck. Therefore, the response for the Group B superload trucks can be established as the upper bound for the trucks which traversed the bridge during the 7 months of monitoring.

3.11. DAMAGE EVALUATION FOR PRESTRESSED CONCRETE BRIDGES

Superloads may create two types of damage for concrete structures: immediate damage, which occurs right after or during the passage of the truck and long term damage, which is a result of cumulative damage. Immediate damage can occur, to some extent, if a bridge is traversed by a truck heavier than its elastic capacity. However, long term damage occurs due to a complicated mechanism. Fatigue or long term damage has not been typically a concern for concrete structures, since there are no prestressed concrete bridge failures due to long term effects reported in the literature. However, this should not hinder the efforts to evaluate the possible long term damage mechanisms. Increasing truck weights and a higher number of superload passages can impair safety or performance of bridges in the long term. Short and long term effects of superloads with regards to damage evaluation are presented in this section.

3.11.1. IMMEDIATE DAMAGE

Immediate damage can be described as the consequence of an overload which alters the response of a bridge to regular traffic. Analysis of the investigated bridges and load test of the I-65 Bridge over US-30 revealed possible damage mechanisms, such as flexure or shear failure and possible collapse of the entire bridge, or a portion of the bridge. Analysis of the investigated bridges, which represented a range of parameters, shows that the investigated bridges have adequate capacity to accommodate the demand imposed by the superload trucks included in the database (824-kip maximum load). This can be attributed to the overly conservative analysis techniques used in the design of bridges. Although the design trucks are not as heavy as the superload trucks, in a beam line analysis they may end up generating forces similar to the ones generated by superloads in a detailed and more accurate analysis. For example, using a 50% conservative GDF for the design truck and using the actual GDF determined from a refined analysis for a superload analysis can give comparable bending moments for a girder. Therefore, it can be stated that many bridges, which were designed using conservative GDFs, have the potential to accommodate superload trucks. Previous superload studies also confirm this (Ziehl and Lamana, 2003 and BDI, 1999). On the other hand, it should not be forgotten that a decision regarding the passage of a superload should be made after a rigorous analysis in order to utilize the actual capacity of a bridge efficiently and safely.

Ziehl and Lamana (2003) monitored strains, deflections and acoustic emission of a PC bridge during a two million pound superload passage. Strains up to 300 microstrains were measured in the bottom flanges at the midspan, but there was no permanent strain or deformation. Furthermore, surveying results showed that plunging failure of the piles did not occur. The recorded acoustic emission levels during the superload passage were similar to those for regular traffic. Hence, it was concluded there was no strand failure during the superload passage. There was no information about crack measurements or crack extension during the superload passage.

In a related study, BDI (1999) instrumented two PC bridges with AASHTO Type-IV girders to evaluate the effects of US Army's 226-kip Heavy Equipment Transporter System (HETS) vehicles. Recorded strains were lower than 140 microstrains for both bridges and the bridges had adequate rating for the HETS vehicles. Response of the bridges to regular trucks was not altered due to the HETS vehicle passages. Therefore, it can be concluded that no immediate damage was observed because of the 226-kip vehicle.

Cracking of the girders or the deck are forms of immediate damage on a local scale. Although these cracks may not jeopardize the safety of a bridge, there is potential to impair long term performance. Specifically, Group D and the Maximum Superload Trucks were found to generate stresses which could result in cracking of the girders for both of the analyzed bridges. The reinforcement can still be in the elastic range when cracking takes place, but cracking of concrete should be identified as a damage parameter due to its long term consequences such as corrosion of the reinforcement.

Cracking of the deck in the negative moment region should be treated in a different way than girder cracking. Analysis of the uncracked sections resulted in stresses in excess of cracking stress in the deck over the pier of bridges due to some superload groups, but previous studies suggest that it could be already cracked. Hadidi and Saadeghvaziri (2005) presented a comprehensive literature review on transverse cracking of concrete bridge decks. It was stated that more than 100,000 bridges in the United States develop early age cracking and cracks typically extend through full depth of the deck. Furthermore, there is no requirement regarding cracking of the deck due to live load in AASHTO LRFR (2003). Therefore, it would be overly conservative to check cracking of a section which might be already cracked. Also, a study by Chung et al. (2006) suggests that transverse cracking of the deck did not have a considerable effect on the GDFs. Hence, the general behavior of a bridge is not expected to be affected by transverse deck cracking and it can be regarded as a local defect. However, it can have a

negative effect on the long-term durability due to corrosion of the negative moment reinforcement.

When the I-164 Bridge was assumed to be fully continuous, some sizeable positive moments, up to 100 ft-k per girder, were obtained at the piers. Typically, positive moment reinforcement is provided at the piers of continuous PC bridges to connect the girders to each other and to resist time dependent and thermal effects. Some of the prestressing tendons are extended into the continuity diaphragm as positive moment reinforcement. Positive moment due to live load effects are not considered for the piers, typically. However, for continuous bridges with span lengths similar to those of the I-164 Bridge, the positive moment reinforcement over the piers might be stressed by Group D or similar superload trucks which have closely spaced heavy axles. It might be possible to damage the continuity diaphragms or the positive moment reinforcement at the piers of some short span bridges, when considerable positive live load moments are superimposed with the positive moments due to time dependent effects. This type of damage can eventually lead to a lower level of continuity for service loads.

Based on the preliminary analysis results, another location for possible damage was found to be the diaphragm-girder connection. Differential deflection of the girders could result in considerable axial forces and bending moments in the diaphragms. However, instrumentation of the I-65 Bridge over US-30 revealed that the diaphragms were not subjected to considerable forces. The dowel bars connecting them to the girders were either yielding or being pulled out. It was not possible to obtain the actual stresses in the dowel bars, since they were not strain-gaged. Having a stronger diaphragm-to-girder connection or use of cross-frame type braces can result in a higher demand on the diaphragms. Furthermore, having a rigid diaphragm-to-girder connection may improve the contribution of the diaphragms to the load distribution, but it would also increase the demand on the diaphragms, and this should be considered in the analysis.

3.11.2. LONG-TERM DAMAGE

Long-term effects of the superload trucks on PC bridges are discussed in this section. Also, available studies on the long-term behavior of PC girders subjected to cyclic load and/or overloads are summarized. The demands generated by typical superload trucks are compared with the limitations recommended by ACI and AASHTO to prevent long-term damage. The main parameters to control the damage were stated as the tensile concrete stresses and the tendon stresses for fatigue and serviceability limit states.

Recommendations for the behavior and design of concrete structures subjected to fatigue loading are provided by ACI 215 (1997). ACI committee 215 indicates that no in-service fatigue failures of prestressed concrete members have been reported, but failure of precracked members have been induced in laboratory tests. The committee recommends that the following criterion be used for fatigue design of beams with prestressed reinforcement: “The stress range in prestressed reinforcement that may be imposed on minimum stress levels up to 60 percent of the tensile strength shall not exceed $0.06f_{pu}$ (15.2 ksi typically) based on cracked section analysis if the nominal tensile stress in the precompressed tensile zone exceeds $3\sqrt{f'_c}$ psi under a realistic estimate of service loadings.” In this statement, f_{pu} is the ultimate strength of prestressing steel and f'_c denotes the compressive strength of concrete. The rationale for this approach comes from the fact that fatigue is not an issue unless the girders are cracked.

Since there was no available data on the influence of load history on the fatigue performance of concrete members, the committee did not make any recommendations on this issue. However, for the fatigue strength of plain concrete members, it was stated that effects of different values of maximum stress can be approximately estimated from constant stress fatigue tests using the Miner’s hypothesis. Therefore, superloads or stresses generated by superloads can be included in the stress spectrum and treated like an ordinary truck.

AASHTO (2004) addresses fatigue as a limit state for concrete structures in Section 5.5.3. First of all, AASHTO (2004) emphasizes that fatigue need not be investigated for concrete deck slabs in multigirder applications. Also, fatigue of the reinforcement need not be checked for fully prestressed members designed to have extreme tensile fiber stress due to Service III Limit State within the six times square root of compressive strength (AASHTO, 2004). In the commentary, it is explained that when this limit is followed, the calculated steel stress range is almost always less than the steel fatigue stress range limit. For prestressing tendons, the stress range is limited to 18 ksi for a tendon radius of curvature exceeding 30 ft.

AASHTO LRFR (2003) does not address fatigue as a limit state for concrete structures, although there is an optional serviceability check. Emphasis is given to strength limit states. For legal load rating, Service III limit state is presented as an optional check. This approach is different than the prestressing strand stress range limitation approach given by ACI 215 (1997), since fatigue is not the main concern for concrete structures in AASHTO LRFR (2003). However, in the commentary, it is mentioned that this optional check limits tensile concrete stresses as in past practice, but it is not specifically encouraged. For permit load rating, the stresses in the reinforcing bars and/or prestressing steel nearest the extreme tension fiber were limited to 90% of the yield stress. The intention of limiting the steel stress is to ensure that the cracks that develop during the overload vehicle passage will close afterwards. Another benefit of this limitation is the assurance of reserve ductility in the member. As an alternative to checking the reinforcement tensile stress, AASHTO LRFR (2003) allows limiting the unfactored moments to 75% of the nominal flexural capacity, which is more convenient than calculating stresses in the strands.

Ratios of the maximum tensile stresses in the strands due to superload trucks and HL-93 to the yield stress of the prestressing tendons (f_y), which is $0.9f_{pu}$, and ratios of the unfactored moments to the nominal flexural capacity for the I-164 Bridge and the I-65 Bridge over US-30 are presented in Tables 3.10 and 3.11. Although AASHTO LRFR

(2003) does not have any recommendations on continuity at service load level, both bridges were analyzed as simple spans. This is a conservative approach and AASHTO (2004) allows analyzing bridges as simple spans for limit states other than strength. Furthermore, field data supports this approach. Note that cracked section properties were used in the strand stress calculations for Group D and Maximum Superload trucks for both bridges and uncracked section properties were used for the other trucks and HL-93 loading.

When the strand stress criterion was checked, the Maximum Truck was generating stresses higher than the optional limit for the I-65 Bridge over US-30 (Table 3.10). However, the ratio of the moments to nominal capacity indicated that the response to the Maximum Truck met the alternate requirement for this bridge (Table 3.11). This shows that the alternate limit for serviceability might sometimes be unconservative. For the I-164 Bridge, the Maximum Truck exceeded both limitations, but the other superload trucks generated tendon stresses within the limit. This might be due to lower prestressing forces specified for the tendons of the I-164 Bridge. Based on these optional serviceability checks, it can be concluded that the Group D and Maximum Trucks have the potential to crack the girders. These girders would not crack due to regular traffic and Group A, B and C Superloads.

The possibility of extensive cracking due to heavy superloads may impair the long-term performance of the girders. Studies available on the long term performance of PC girders subjected to fatigue loading and overloads were investigated to understand the long term effects of superloads on PC structures. Some notable studies on the long-term behavior of PC girders subjected to cyclic loading are summarized below.

Hanson et al. (1970) tested several concrete I-beams to evaluate their fatigue resistance. The test setup involved use of two loading points to create a constant moment region between the load points. The specimens were overloaded to cause flexural and inclined cracking prior to repeated loading. Each beam was subjected to 80% of the

ultimate flexural capacity initially to create inclined cracks in both outside “shear spans”, the regions between the support and the load point. The initial overload caused yielding in the steel and resulted in extensive cracking. This response may resemble the behavior of a girder which was subjected to a superload and developed cracks as a result of the overload. Although the initial loading was higher than that of an overload allowed by the optional check of AASHTO LRFR (2003), this was the only available study on overload response.

Hanson et al. (1970) concluded that fatigue life of the tested beams was less than what was calculated with respect to fatigue strength of strands. Furthermore, it was recommended that the bottom fiber stress of the PC beams subjected to large number of stress cycles should be limited to six times the square root of the compressive strength, until more information becomes available. This recommendation is conservative compared to the requirements of AASHTO LRFR (2003). Also, S-N relationships were developed by Hanson et al. (1970) to predict the relationship between fatigue life and stress range. Development of these relationships involved both high cycle and low cycle response. However, there was not any special treatment for overloads which were smaller than the initial loads causing damage. Hence, it can be concluded that Miner’s Rule might be applicable for the assessment of overload effects after the initial damage. Remaining life of the girders can be calculated using the developed S-N relationships.

Zia et al. (2002) investigated the fatigue resistance of PC beams impaired by transverse cracks. Two full size AASHTO PC girders, one Type-III and one Type-V, were tested. Both girders had transverse cracks at their top flanges extending in to their webs due to restraining force against contraction during production. Prior to fatigue testing, each girder was loaded beyond its cracking capacity and flexural cracks were created at the bottom flange. After that, each girder was subjected to one million cycles of service load and 2500 intermittent cycles of an overload which was equivalent to 75% of the ultimate capacity of the composite girder. The overload cycles simulated the

maximum permit trucks which could traverse the bridge according to AASHTO LRFR (2003). Finally, each girder was tested to failure to determine the ultimate load capacity.

Based upon the test results, Zia et al. (2002) concluded that there was no loss of strength or stiffness after one million cycles of service load with a corresponding stress range of 19.5 ksi in the strands and 2500 cycles of overload with stress range of 29.3 ksi in the strands. There was no sign of fatigue or fracture in the strands, although the stress range was higher than the 15.2 ksi limit calculated according to ACI 215 (1997). The ultimate load obtained from the test was higher than the predicted value. Furthermore, ductility of the tested girders was not affected by the fatigue loading and the overload cycles. Finally, it was concluded that the AASHTO LRFD (2004) limit of 18 ksi on stress range for the strands is an appropriate limit. However, progressive increase in cracking and permanent deflection due to each group of 500 cycles of overload was also noted. This implies that some damage or extensive cracking due to overloads should be expected although it may not impair the safety of a bridge. On the other hand, extensive cracking in the girders may have different consequences in the field environment. Exposure to deleterious environmental conditions could possibly alter the fatigue response of the girders. Therefore, results of the tests conducted in the lab environment may not always be conservative.

In a study by Baran et al. (2003), fatigue resistance of PC beams with vertical prerelease cracks was investigated. Vertical cracks near the midspan of the PC girders were observed during the curing process. Cracks extending through the depth of the girders were attributed to restrained shrinkage and thermal effects prior to release of the prestressing strands (Baran et al., 2003). It was concluded that beams with prerelease cracks were likely to have higher strand stresses due to stress concentration. Furthermore, increased stress together with the fretting between the individual wires can cause the prestressing strands to fracture at the crack locations when subjected to repeated loads. Although the cracks did not affect the ultimate capacity of a beam, reduction in the fatigue life was observed.

Bruce et al. (2003) tested bulb-tee girders made of high strength concrete with a compressive strength of 10 ksi. Five million cycles of flexural load was achieved for the girders which had a limitation on the tensile stress of $6\sqrt{f_c}$. However, girders subjected higher tensile stresses (more than the cracking stress) endured only 2.5 million cycles due to fatigue fracture of strands. Similar to Hanson et al.'s study (1970) it was concluded that the bottom fiber stresses should be limited to six times the square root of the compressive strength in order to obtain better fatigue performance.

Hagenberger (2004) studied the effect of strand fatigue on the load rating of PC bridges. Five PC bridges built in the 1950s and 1960s were instrumented and analyzed. Load ratings of the bridges for design trucks were controlled by the concrete tensile stress criterion. It was concluded that repeated cycles causing tensile stresses as low as three times the square root of the compressive strength in the concrete can cause cracking. Furthermore, fatigue related problems increase significantly after cracking. Stress concentration problems due to cracks led to rapid fatigue of prestressing strands. Limiting the tensile stresses in the concrete to six times the square root of the compressive strength was recommended to improve the fatigue life. If the calculated stress range in the strand due to the fatigue truck is limited to 5 ksi, the PC girder is expected to have infinite life according to the test results. For stress ranges exceeding the 5 ksi endurance limit, however, the finite cyclic life can be predicted using the S-N relationships.

Based on the analysis results and an extensive literature survey, the main damage indication parameter is considered to be the bottom fiber stress. Limiting the bottom fiber stress to six times the square root of the compressive strength ensures infinite life, but it may not be economical to adopt the same limit for superloads. Therefore, cracking of the girders due to superloads might be allowed, provided that extensive cracking will not occur and the cracks will close after the passage of superload. Any superload truck cracking the girders has the potential to exceed the optional prestressing strand stress limit. The optional strand stress limitation of $0.9f_{py}$ (the yield stress, f_{py} , is equal to 90% of

the ultimate stress, f_{pu} , for stress-relieved prestressing strands which are also known as LoLax strands) given in AASHTO LRFR (2003) should not be exceeded. A serviceability check is expected to control the rating of most PC bridges for superloads. The number of superload crossings can be limited with respect to the available S-N relationships, but this issue requires further research. However, previous studies suggest that the rare passage of heavy superloads and those superloads generating moments, shears, and stresses within the AASHTO LRFR (2003) limitations are not expected to impair long term performance of PC bridges.

3.12. FINDINGS AND CONCLUSIONS

Analysis of two prestressed concrete bridges, instrumentation of a prestressed concrete bridge, and a comprehensive literature survey led to the following findings and conclusions:

- Performance of the in-house transducers was satisfactory in the field environment. The transducers gave consistent and reliable results during the five-month monitoring period.
- The eccentric beam model may not represent the actual behavior of prestressed concrete bridges with bulb-tee girders because of not including the actual interaction between the top flange and the deck in the FEM.
- Prestressed concrete bridges made continuous for live load may not behave as a fully continuous structure according to the live load test of the case study bridge.
- The GDFs computed using the AASHTO (2004) equation gave conservative results for the investigated bridges. The error was more than 100% for the instrumented bridge when one lane was loaded.
- Diaphragms of the instrumented bridge did not affect the load distribution characteristics. Therefore, the diaphragms were not subjected to considerable live-load stresses.

- Integral abutments led to an end fixity between 20 and 25% for the instrumented bridge.
- The GDFs for the superload trucks were not considerably different the GDFs for regular trucks.
- 248-kip Group B and 366-kip Group C Superload Trucks generated demands comparable to that for the maximum recorded event in the five-month monitoring period. Furthermore, the demands for the Group A and Group B Superload Trucks were comparable to the demand for HL-93 Loading.
- Capacity analyses showed that all superload trucks included in the database generated demands lower than the capacities of the investigated bridges at the critical sections for positive moment, negative moment and shear.
- At the piers of short span continuous bridges, positive moments generated by Group D or similar superload trucks with closely spaced heavy axles can cause sizeable positive moments. Superposition of the positive live load moment with the positive moment due to time dependent and thermal effects may damage the continuity diaphragms or stress the positive moment reinforcement.
- The optional serviceability criterion of AASHTO LRFR (2003) was exceeded only by the demands due to the 824-kip Maximum Superload Truck for the investigated bridges. However, a superload truck with similar axle spacing but weighing about 760 kips would generate a demand lower than the optional serviceability criterion.
- The optional serviceability criterion of AASHTO LRFR (2003), which limits tendon stresses to control cracking, should be accepted as an upper bound to control the long term effects of superload trucks. Limited cracking of the girders might be allowed provided that the recommended limit for tendon stresses is not exceeded.

Table 3.1 Measured and computed strains for 100-kip beam load

Sensor	Microstrains		Difference (%)
	Calculated	Measured	
Transducer 1	-246	-197	-20
Transducer 2	245	207	-15
Transducer 3	193	199	3
Strain Gage 1	258	272	6
Strain Gage 2	221	239	8
Strain Gage 3	-246	-187	-24

Table 3.2 Instrumentation plan summary for the I-65 Bridge over US-30

Number of Transducers												
Section	Section A			Section B		Section C			Section D			Section E
Girder Number	Top Flange	Web	Bottom Flange	West	East	Top Flange	Web	Bottom Flange	Top Flange	Web	Bottom Flange	Bottom Flange
1												
2												
3												
4								1				
5								2			1	
6				4				2		1	2	
7	2	2	2	4	4	2	2	2	2	2	2	2
8					4			2		1	2	
9								2			1	
10								2				
11								1				
SUM	6			16		18			14			2

Table 3.3 Axle weights of the test trucks

	Weight (kips)		
	Axle 1	Axle 2	Axle 3
Truck 1	12.0	15.7	16.2
Truck 2	12.4	15.8	16.4
Truck 3	13.0	16.7	17.0
Truck 4	13.9	16.3	16.8

Table 3.4 Estimated curvature values along Girder 7 for Load Case 3

Long. Location	$\frac{y}{\varepsilon}$	$\kappa \left(\frac{1}{\text{in}} 10^6 \right)$
Pier	5.17	0.19
Midspan	-1.00	-1.00
Abutment	3.64	0.27

Table 3.5 Diaphragm stresses for Load Case 4

Transducer Location	Longitudinal Stress (psi)							
	Top of Diaphragm				Bottom of Diaphragm			
	North Side		South Side		North Side		South Side	
	Calculated	Measured	Calculated	Measured	Calculated	Measured	Calculated	Measured
East of Girder 8	151	9	144	9	194	16	194	54
West of Girder 7	18	7	-42	19	384	18	382	18
East of Girder 7	-55	-11	-60	-40	384	58	404	18
West of Girder 6	171	2	171	12	161	4	160	8

Table 3.6 Diaphragm stresses for Load Case 5

Transducer Location	Longitudinal Stress (psi)							
	Top of Diaphragm				Bottom of Diaphragm			
	North Side		South Side		North Side		South Side	
	Calculated	Measured	Calculated	Measured	Calculated	Measured	Calculated	Measured
East of Girder 8	190	7	212	3	62	12	62	53
West of Girder 7	-57	8	-134	22	405	18	402	16
East of Girder 7	-99	-13	-104	-51	398	21	377	59
West of Girder 6	188	4	188	11	94	5	90	11

Table 3.7 Girder distribution factors for an interior girder of the I-164 Bridge

Truck	GDF	
	Positive Moment	Negative Moment
Group A	0.36	0.39
Group B	0.32	0.38
Group C	0.34	0.40
Group D	0.33	0.38
Max Truck	0.34	0.38
HL-93	0.34	0.39

Table 3.8 Girder distribution factors for an interior girder of the I-65 Bridge over US-30
(from refined FEM)

Truck	GDF	
	Positive Moment	Negative Moment
Group A	0.28	0.36
Group B	0.31	0.31
Group C	0.30	0.30
Group D	0.28	0.34
Max Truck	0.27	0.37
HL-93	0.30	0.29

Table 3.9 Girder distribution factors for an interior girder of the I-65 Bridge over US-30
(from continuous and simply supported FEM)

Truck	GDF	
	Positive Moment	Negative Moment
Group A	0.28	0.42
Group B	0.27	0.36
Group C	0.27	0.35
Group D	0.25	0.39
Max Truck	0.24	0.37
HL-93	0.27	0.33

Table 3.10 Ratios of the strand stresses (f_s) to yield stress (f_y , 243 ksi) for the I-164 Bridge and the I-65 Bridge over US-30

Truck Type	f_s / f_y		Allowed f_s / f_y
	I-164 Bridge	I-65 Bridge	
Group A	0.64	0.72	0.9
Group B	0.64	0.72	
Group C	0.65	0.72	
Group D	0.78	0.82	
Max Truck	0.97	0.93	
HL-93	0.64	0.72	

Table 3.11 Ratios of total moments (M_t) to nominal capacity (M_n) for the I-164 Bridge and the I-65 Bridge over US-30

Truck Type	M_t / M_n		Allowed M_t / M_n
	I-164 Bridge	I-65 Bridge	
Group A	0.48	0.42	0.75
Group B	0.41	0.42	
Group C	0.51	0.45	
Group D	0.69	0.57	
Max Truck	0.87	0.68	
HL-93	0.43	0.41

Figure 3.1 I-65 Bridge over US-30 in Merrillville, Indiana

Figure 3.2 General plan of the Northbound I-65 Bridge over US-30

Figure 3.3 Typical cross-section (looking North) of Northbound structure of the I-65 Bridge over US-30

Figure 3.4 General plan of the I-164 Bridge

Figure 3.5 Cross-section of the I-164 Bridge

Figure 3.6 Cross-section of the finite element model of the I-65 Bridge over US-30

Figure 3.7 Plan view showing the girders and diaphragms of the I-65 Bridge over US-30

Figure 3.8 Plan view of the deck and load patches for the Group C Superload truck

Figure 3.9 Bending moments resisted by the composite girders and parapets of the I-65 Bridge over US-30 for the right lane truck position

Figure 3.10 Bending moments resisted by the composite girders and parapets of the I-65 Bridge over US-30 for the center lane truck position

Figure 3.11 General view of the test setup

Figure 3.12 Vertical crack in the compression region of the beam

Figure 3.13 Load deflection response of the test beam

Figure 3.14 Sensors on the west side of the test beam

Figure 3.15 Sensors on the east side of the test beam

Figure 3.16 A typical foil gage (no insulation)

Figure 3.17 Strain measurement obtained by foil gage G1 at load point

Figure 3.18 Layout of the in-house transducer (all dimensions are in inches)

Figure 3.19 In-house transducer installed to the test beam

Figure 3.20 Full bridge strain gage used for the in-house developed transducer (Vishay Intertechnology, Inc., 2003)

Figure 3.21 Strain measurement obtained by the in-house transducer

Figure 3.22 Instrumented sections of the I-65 Bridge over US-30

Figure 3.23 Transducers on Girder 7 at Section A

Figure 3.24 Transducers installed on the diaphragms at Section B

Figure 3.25 Transducers on Girder 7 at Section C

Figure 3.26 Transducers on Girders 5, 6, 8, 9 and 10 at Section C

Figure 3.27 Transducers on Girders 4 and 11 at Section C

Figure 3.28 Transducers on Girder 7 at Section D

Figure 3.29 Transducers on Girders 6 and 8 at Section D

Figure 3.30 Transducers on Girders 5 and 9 at Section D

Figure 3.31 Transducers on Girder 7 at Section E

Figure 3.32 Potentiometer used for deflection measurement

Figure 3.33 Deflection measurement setup under the I-65 Bridge over US-30

Figure 3.34 In-house transducer installed on the I-65 Bridge over US-30

Figure 3.35 Transducers at Section A (abutment)

Figure 3.36 Transducers at Section B (diaphragm)

Figure 3.37 Transducers at Section C (midspan)

Figure 3.38 Transducers at Section D (and E at negative moment region)

Figure 3.39 Dimensions of a typical test truck

Figure 3.40 Longitudinal position of the test trucks for Load Cases 1 to 3

Figure 3.41 Test truck positions for Load Case 1

Figure 3.42 Load Case 1

Figure 3.43 Test truck positions for Load Case 2

Figure 3.44 Test truck positions for Load Case 3

Figure 3.45 Longitudinal position of the test trucks for Load Case 5

Figure 3.46 Load Case 5

Figure 3.47 Longitudinal position of the test trucks for Load Case 6

Figure 3.48 Load Case 6

Figure 3.49 Strain distribution at Section D (pier) of Girder 7 for Load Case 3

Figure 3.50 Strain distribution at Section C (midspan) of Girder 7 for Load Case 3

Figure 3.51 Strain distribution at Section A (abutment) of Girder 7 for Load Case 3

Figure 3.52 Supports for the eccentric beam model

Figure 3.53 Free vibration record of the I-65 Bridge over US-30

Figure 3.54 Mode 1 of the I-65 Bridge over US-30

Figure 3.55 Bottom flange stresses at the midspan (Section C) for Load Case 1

Figure 3.56 Bottom flange stresses at the midspan (Section C) for Load Case 2

Figure 3.57 Bottom flange stresses at the midspan (Section C) for Load Case 3

Figure 3.58 Bottom flange stresses at the midspan (Section C) for Load Case 4

Figure 3.59 Bottom flange stresses at the midspan (Section C) for Load Case 5

Figure 3.60 Bottom flange stresses at the midspan (Section C) for Load Case 6

Figure 3.61 Girder deflections at the midspan for Load Case 1

Figure 3.62 Girder deflections at the midspan for Load Case 2

Figure 3.63 Girder deflections at the midspan for Load Case 3

Figure 3.64 Girder deflections at the midspan for Load Case 4

Figure 3.65 Girder deflections at the midspan for Load Case 5

Figure 3.66 Girder deflections at the midspan for Load Case 6

Figure 3.67 Maximum strains recorded at the midspan for Load Cases 9 and 10

Figure 3.68 Maximum deflections recorded at the midspan for Load Cases 9 and 10

Figure 3.69 Positive moment distribution at the middle of the I-164 Bridge

Figure 3.70 Negative moment distribution at the pier of the I-164 Bridge

Figure 3.71 Positive moment strength evaluation of the I-164 Bridge

Figure 3.72 Negative moment strength evaluation of the I-164 Bridge

Figure 3.73 Shear strength evaluation of the I-164 Bridge

Figure 3.74 Bottom fiber stresses at the middle of the I-164 Bridge

Figure 3.75 Positive moment distribution at the midspan of the I-65 Bridge over US-30
(from refined FEM)

Figure 3.76 Negative moment distribution at the pier of the I-65 Bridge over US-30
(from refined FEM)

Figure 3.77 Positive moment strength evaluation of the I-65 Bridge over US-30

Figure 3.78 Negative moment strength evaluation of the I-65 Bridge over US-30

Figure 3.79 Shear strength evaluation of the I-65 Bridge over US-30

Figure 3.80 Bottom fiber stresses at the middle of the second span of the I-65 Bridge over US-30 (from simply supported and continuous model)

Figure 3.81 Strains recorded at the bottom flanges of Girders 6, 7, 8, 9 and top flange of Girder 7 at Section C

Figure 3.82 Distribution of maximum bottom flange strains recorded at the midspan for five-month monitoring period

Figure 3.83 Strain distribution at the midspan for the event recorded in December, 2005

Figure 3.84 Strains recorded at the Sections A, C and D of Girder 7 for the event recorded in December, 2005

Figure 3.85 Stress distributions due to trucks in the right lane

Figure 3.86 Stress distributions due to trucks in the center lane

Figure 3.87 Stress distributions due to trucks positioned side by side in the center and right lanes

CHAPTER 4. CONTRIBUTION OF PARAPETS TO LIVE-LOAD RESPONSE FOR SUPERLOAD PASSAGES

The main objective of this chapter is to evaluate the effects of parapets on the live load response of slab-on-girder steel and prestressed concrete (PC) bridges subjected to superload vehicles and the effects of these loads on the parapets. The presence of parapets can result in reduced girder distribution factors (GDF) for critical girders, and this reserve strength can be considered for passage of a superload truck. This reduction, as well as effects of discontinuous parapets and the capacity of parapets, is investigated. Two steel and two PC bridges with significantly different geometric proportions were analyzed to evaluate the sensitivity of the structures to the effects of parapets. The axial forces and bending moments resisted by the parapets were compared with the capacity of the parapets.

Parapets are secondary structural members built at the edges of a bridge deck for resisting vehicle impacts. Similar parapet details have been adopted by many state Departments of Transportation (DOT) throughout the US; this is in contrast to what is done for the other secondary members. Parapets are considered to be secondary members and, per the AASHTO LRFD Bridge Design Code (2004), they are not to be included in the structural analysis for determining load resistance. However, for the evaluation of bridges traversed by special permit trucks, which are often regulated in terms of transverse position and travel speed, the presence of parapets could possibly be quite beneficial.

If the contribution of the parapets is not included in the structural analysis, an extra heavy-weight vehicle may be refused a permit because analysis could possibly show over-stressing of the bridge girders. In reality, however, the bridge may have adequate load capacity. A traditional modeling approach that ignores the parapet contribution can give satisfactory results for wider bridges, but neglecting the contribution of the parapets may result in inaccurate and overly conservative structural models for narrower bridges, such as a two lane bridge with narrow shoulder widths.

Two existing steel and two PC bridges in Indiana were analyzed with the goal of quantifying the influence of parapets on bridge response and determining the forces resisted by the parapets. Properties of the US-52 Bridge, the I-65 Bridge over Ridge Road, the I-164 Bridge, and the I-65 Bridge over US-30 were described in Chapters 2 and 3. The US-52 and the I-164 Bridges have 33-in high parapets. The I-65 Bridges over Ridge Road and US-30 have 46-in and 45-in high parapets, respectively. The 46-in parapet was almost identical to the 45-in parapet. Cross-sections of 33-in and 45-in parapets used by INDOT are presented in Figures 4.1 and 4.2. Concrete with a specified compressive strength of 4 ksi was used for the deck and the parapets of all investigated bridges.

Three-dimensional static analyses of the bridges were conducted using the finite element analysis (FEA) features of the SAP2000 program. Details of the finite element models (FEMs) were presented in Chapters 2 and 3. Material and geometric nonlinearities were not included in the analysis. Therefore, cracking of the concrete sections and yielding of the steel sections were not represented in the FEMs. The FEMs of the bridges included all structural and non-structural members (parapets and cross-frames or diaphragms) which may contribute to the overall stiffness and affect the structural behavior. Discontinuous parapets were simulated by deleting shell elements coinciding with a parapet joint, leaving a 2.54 cm (1 in) gap as shown in Figure 4.3. Rigid link elements were used to provide the proper distance from the centroid of the deck to the top of the girder and to the bottom of the parapet. Roller supports were

assigned to the bottoms of the girders at the piers. Fixity effects due to the integral abutments of the I-65 Bridges were not included in the analysis. For the FEMs of the continuous I-164 Bridge and I-65 Bridge over US-30, it was assumed that the bridges behave as fully continuous to be consistent with the design assumptions at ultimate load level. The US-52 Bridge was also modeled as fully continuous, since the steel beams were continuous. In summary, no refinement based on the instrumentation data was made to the FEMs used in this section. Regardless, in Chapters 2 and 3, it was shown that the FEMs predict the general response of the investigated bridges with a reasonable accuracy.

Performance of the discontinuous parapets (Figure 4.3) along the span was also considered in the analysis. Discontinuous parapets have full depth joints, or gaps, which may be introduced at locations other than the deck joints to reduce transverse parapet cracking (AASHTO, 2004).

The truck loads applied to the investigated bridges were selected from an extensive permit truck database obtained from Indiana DOT and the Wisconsin DOT by Wood (2004). Use of the superload trucks, in addition to the HS-20 Truck, provided a wide variety of load cases which were used to evaluate bridges with and without parapets. Since overload trucks usually do not traverse a bridge side by side, the bridges were analyzed for single truck loadings only. Also, for this portion of the study it was assumed that the trucks would cross at a crawl, so impact factors were not considered.

4.1. BACKGROUND

There are only a few studies which have investigated the effects of concrete appurtenances — like parapets — directly. Billing (1984) explained the stiffening effects of parapets with respect to midspan deflections of two similar bridges. One bridge had discontinuous New Jersey type parapets, whereas the other one did not have parapets.

Measured static deflections of the most heavily loaded girders were about 40% smaller for the bridge with parapets.

Smith and Mikelsteins (1988), meanwhile, investigated load distribution in edge stiffened slab and slab-on-girder bridges. As a result of edge stiffening, it was found that the midspan moments carried by the interior girders decreased. Also, Mabsout et al. (1997) analytically examined the contribution of sidewalks and parapets to the load carrying capacity of steel girder bridges. It was found that when these secondary members act integrally with the bridge deck, they stiffen the edges and attract more load. As a result of this action, a 30% increase in the capacity was obtained for the investigated bridge. Connor and Huo (2006) analytically investigated the effects of parapets for different overhang lengths and skew angles. It was found that the contribution of parapets reduces the maximum stresses by up to 36% for exterior girders and up to 13% for interior girders.

Furthermore, Eamon and Nowak (2002) conducted a comprehensive numerical study on the effects of edge-stiffening elements and diaphragms on bridge resistance and load distribution. It was concluded that typical secondary members can reduce the girder distribution factors (GDFs) by 10 to 40% in the elastic range and by 5 to 20% at ultimate capacity.

The effects of appurtenances on the bridge response can be significant, since a general design feature is to make concrete curbs, parapets, barriers, and dividers structurally continuous with the deck and deck system. Despite this, the current AASHTO LRFD Bridge Design Code (2004) excludes the contribution of an appurtenance at strength limit states as a safety measure and allows the option of considering these members only for serviceability and fatigue limit states.

4.2. EFFECTS OF PARAPETS ON THE LIVE-LOAD RESPONSE OF STEEL BRIDGES

Several output parameters, including bending moments, axial forces, resultant stresses and deflections, were obtained from the three dimensional FEMs of the US-52 Bridge and the I-65 Bridge over Ridge Road. The axial force is not typically critical in the design of bridge beams or girders. However, the same approach may not be true for the parapets, for which the majority of the demand is axial. Furthermore, the centroidal axis of the parapet is located at a considerable eccentricity from the centroid of the entire bridge cross-section. The large eccentricity leads to considerable flexural stiffness and significant participation in the overall resistance of the bridge to loads.

To account for this large eccentricity, an adjustment to the bending moments of the parapets and girders was made. The neutral axis locations of the composite girders and the parapets were used to establish the neutral axis position of the entire bridge cross-section, which consists of all composite girders and parapets. Then, the forces and bending moments corresponding to each member's neutral axis were redefined with respect to the neutral axis of the entire bridge cross-section. The parallel axis theorem was used to calculate the couples, obtained by summing up the bending moment and the product of the axial force and the distance between the neutral axis of the member and the neutral axis of the entire bridge cross-section. For example, the couple resisted by the parapet was calculated as:

$$M_{NA} = M_{NAP} + Pd \quad (\text{Eqn. 4.1})$$

where M_{NA} is the couple resisted by the parapet about the neutral axis of the entire bridge cross-section, M_{NAP} is the bending moment resisted by the parapet about its neutral axis, P is the axial force resisted by the parapet, and d is the distance between the centroidal axis of the parapet and the neutral axis of the entire bridge cross-section, as shown in Figure 4.4. With summation of the resisting couples, equilibrium of the bridge cross-

section is retained. These couples are presented as bending moments in figures showing distribution of bending moments.

4.2.1. EFFECTS OF PARAPETS ON THE US-52 BRIDGE

The couples resisted by the beams and parapets in the first span of the continuous US-52 Bridge will be presented in this section. Two different transverse truck locations were selected for this bridge. The first location was at the center of the right (travel) lane, and the other was at the shoulder. While the trucks would normally cross the bridge in the travel lane, it is possible that they could move to the shoulder during an emergency. When the trucks were positioned in the shoulder, the center of the wheel nearest to the parapet was about 61 cm (2 ft) from the parapet.

A number of different parapet continuity conditions can be found in highway bridges. In this study, three different conditions were evaluated. Case “OP” represents the FEM of the bridge with the ordinary parapet configuration. The OP type parapets are continuous between the piers and discontinuous over the piers. Case “DP” represents the FEM with discontinuous parapets, with joints in the parapet at every 15-20 ft along the bridge length and over the piers. Case “NP” simulates a fictitious bridge without any parapets, guardrails or curbs, but with the same beam, diaphragm, deck and span properties as the original US-52 Bridge. Finally, Case “CP” represents the FEM with continuous parapets along the bridge length.

The load distribution was evaluated at a cross-section 34.75 ft away from the beginning of the first steel span. The maximum bending moments were obtained at this section when the heavy axle groups of the superload trucks were positioned over the first span. For the discontinuous parapet case (DP), this longitudinal location coincided with a parapet gap. Selection of this cross-section enabled investigation of the influence of a stress concentration at the exterior beam due to the discontinuity of the parapet.

The couples resisted by each member (composite beam or parapet) at the selected longitudinal location are presented in Figure 4.5 for the Group C Superload Truck located at the center of the right lane and the shoulder, respectively. The horizontal axes of the graphs indicate the transverse location of the center of each beam or parapet relative to the bridge centerline. The markers in the graphs represent the bending moments at each of the beams and parapets. For example, the parapets are located at $x = -20.75$ ft and $x = 20.75$ ft next to the exterior beams.

First, it can be observed that the bending moments for the shoulder loading position are notably higher than those generated by the travel lane loading position. Moreover, the comparison demonstrates that the travel lane loading position causes a much more uniform distribution of girder bending moments to resist the truck loading than the shoulder lane loading. Second, the couples resisted by the parapets can be as large as the bending moments resisted by the heavily loaded beams for the right lane loading position and even higher than the maximum beam moment for the shoulder loading position.

When considering the effect of parapet type, the discontinuous parapet case (DP) has a load distribution mechanism similar to the ordinary parapet case (OP) for the interior beams. However, the exterior beams carry significantly higher loads in the DP case. For the exterior beams, the bending moment even exceeds the bending moment obtained in the no parapet (NP) case. Hence, the gaps in the discontinuous parapets can be interpreted as notches or stress raisers. From the load distribution point of view, it appears that the load resisted by the parapets for each 20 ft long parapet segment is being transferred to the exterior beams wherever there is a discontinuity.

The influence of parapet size was also studied. A comparison of the couples resisted by the members for the FEMs with 45-in and 33-in continuous parapets between the supports reveals that the stiffer (taller) parapet attracts more load, resulting in higher

parapet forces and lower girder bending moments. For example, when the Group C Superload Truck was positioned in the shoulder, use of the 45-in parapet resulted in a girder distribution factor (GDF) 15% lower than that for the 33-in parapet case. For the right lane loading position, however, the difference was only 2%.

The GDFs were obtained by calculating the ratio of the highest composite beam couple to the summation of couples resisted by all members (beams and parapets) at that section. The GDFs for the OP and NP cases are presented for both of the loading positions in Tables 4.1 and 4.2, with respect to parapet size. The GDFs are always lower for the OP cases, and significantly lower than the NP case for the shoulder loading position, with differences ranging from roughly 28 to 33%, as compared to 5 to 9% for the right lane loading position when the bridge model was analyzed with the original 33-in tall parapets (Table 4.1). If the bridge model is analyzed with the 45-in parapets, the GDFs are 36 to 44% lower for the shoulder loading position and 7 to 12% lower for the right lane loading position compared to the NP case (Table 4.2). Also, the GDF is also somewhat dependent on the truck configuration.

With the contribution of the parapets considered in the analysis (OP Case), approximately equal GDFs can be achieved for all possible transverse loading positions of the US-52 Bridge. The GDFs obtained with respect to the moments resisted by the exterior beams may therefore be less critical in the evaluation or rating of a bridge for flexure if the parapets can be included in the analysis.

4.2.2. EFFECTS OF PARAPETS ON THE I-65 BRIDGE OVER RIDGE ROAD

For the I-65 Bridge over Ridge Road, the live-load strains were monitored on the 46-in parapet using foil gages as described in Chapter 2 (Figures 2.31 and 2.32 show the locations of the strain gages attached to the East side parapet at the midspan). Although there were four gages bonded to the parapet, two of them malfunctioned due to problems

encountered during installation. The 4-in foil gages for concrete surface applications required constant pressure application for six hours at or above 75° F. This would require closure of the left lane on I-65 Southbound. However, due to traffic demands on I-65, constant pressure could only be applied for a limited amount of time. Deviation from the standard installation procedures, as well as the physical challenge in applying constant pressure to a vertical surface, resulted in malfunctioning of some gages. Although accuracy of the measurements obtained from the remaining two gages is expected to be limited, the measurements demonstrated that the parapets are clearly resisting live loads. Figure 4.6 shows the stresses generated in the parapet for Load Case 2 of the load test (test trucks were positioned approximately 2 ft from the parapet). The stresses calculated from measured strains are in reasonable agreement with those predicted by FEA.

In order to evaluate the effects of parapets, bending moments and girder deflections were extracted from the FEA at the midspan of the I-65 Bridge over Ridge Road. One longitudinal and two transverse truck positions were considered in the analysis. For the longitudinal direction, the trucks were positioned such that the heavy axles of the superload trucks were located at midspan of the symmetric, half-span bridge model described in Chapter 2.

In the analyses, the trucks were placed at the center of the bridge in the transverse direction or next to the parapet in the shoulder. With respect to the parapet properties, two cases were analyzed: the first with ordinary continuous parapets (OP), and the second without the parapets (NP).

The OP and NP Cases with the center loading position resulted in almost identical results for the GDFs (Table 4.3). The presence of the parapets does not affect the load distribution characteristics or the deflections of this bridge when the truck is positioned at the center of the bridge. The four girders nearest to the truck were found to resist most of the load. The GDFs are between 21% and 25% for both cases. The wide deck, larger girder spacing and the stiff plate girders are considered to be the primary factors

responsible for this behavior. For the center loading position of the I-65 Bridge, it can be concluded that the presence of the parapets makes no difference in terms of deflections or bending moments resisted by the girders.

Distributions of the bending moments due to the Group C Superload Truck positioned at the midspan of the I-65 Bridge over Ridge Road are presented in Figure 4.7 for the center and shoulder loading positions. The bending moment resisted by the outside three girders is notably less when the parapets are included in the analysis. For all truck groups, the decrease in the GDF is about 22% (Table 4.3) when the parapets are included in the analysis. The contribution of the parapets for the I-65 Bridge is believed to be lower than that for the US-52 Bridge because of the stiffer plate girders of the I-65 Bridge.

4.3. EFFECTS OF PARAPETS ON THE LIVE-LOAD RESPONSE OF PC BRIDGES

Effects of the parapets on the live-load response of typical prestressed concrete bridges were investigated using the same approach presented in the previous section. The I-164 bridge and the I-65 Bridge over US-30 have different characteristics in terms of span and girder properties. Therefore, the influences of parapets on the live-load response of these bridges are expected to be different, due to different girder stiffnesses and span length for the most part.

4.3.1. EFFECTS OF PARAPETS ON THE I-164 BRIDGE

The I-164 Bridge has 33-in high parapets with discontinuities along the span. Therefore, the original parapet configuration of this bridge was similar to the fictitious DP case of the US-52 Bridge. There were three gaps along the parapets of the I-164 Bridge: the first one in the negative moment region of the second span at 4 ft from the

first pier, the second one at the middle of the second span, and the third one in the second span at 4 ft from the second internal pier. Since the original parapet configuration of the I-164 Bridge was discontinuous in the longest span (second span), the first analysis case is called DP and the second one is called NP (no parapet).

The trucks were positioned to generate the maximum positive moment in the FEA. Two transverse locations were considered in the analysis; the first one was at the center of the right lane, and the second one was at the shoulder. Moments resisted by the girders were obtained at the middle of the three span bridge, which corresponds to the midspan of the second span. Furthermore, this longitudinal location included the parapet gap. Therefore, parapets were not resisting any bending moments at this section. Distributions of the bending moments resisted by the PC girders of the I-164 Bridge due to the Group C Superload Truck are presented in Figure 4.8. Since there was a parapet gap at the middle of the bridge, parapets are not included in Figure 4.8. The GDFs obtained for the I-164 Bridge are presented in Table 4.4.

For the right lane loading position, response of the bridge was not affected significantly by the presence of the parapets. However, when the trucks were positioned in the shoulder, the parapet gap amplified the bending moments at the exterior girder. The increase was 7 to 10%. For the I-164 Bridge, it can be said that the discontinuous parapets do not contribute to live-load resistance at the maximum positive moment location, but rather increase the moments resisted by the exterior girders due to the stress concentration effect.

4.3.2. EFFECTS OF PARAPETS ON THE I-65 BRIDGE OVER US-30

The I-65 Bridge over US-30 has continuous 45-in parapets between the supports. The trucks were positioned at the first span of the bridge, and the moments generated at the maximum moments generated at the first span were calculated. In the transverse

direction, the trucks were positioned at the center of the bridge and at the shoulder. Only two cases were considered in the analysis with respect to parapet configuration. The I-65 Bridge over US-30 was analyzed with the ordinary 45-in parapets (OP) and without the parapets (NP). The bending moment distributions at the first span for the Group C Superload Truck are presented in Figure 4.9. The GDFs obtained for both loading positions from the FEMs with and without the parapets are presented in Table 4.5.

Effects of the parapets on the live-load response of the I-65 Bridge over US-30 were similar to those on the I-65 Bridge over Ridge Road. When the superload trucks were positioned at the center of the bridge, the parapets decreased the GDFs by 2%. For the shoulder loading position, however the parapets led to a 20% reduction in the GDFs. Furthermore, the parapet developed a bending moment corresponding to 30% of the moment resisted by the exterior girder.

Analysis results obtained from all four bridges indicate that lower GDFs can be obtained when the trucks are positioned close to the center of the bridge. Therefore, the superload trucks should be positioned close to the center of bridge, since this loading position leads to a better load distribution and lower demands on the girders. The parapets will likely have a limited contribution for bridges with more than two lanes for the center loading position. However, for bridge rating, the capacities of all girders are typically considered. The exterior girders, which have higher GDFs, control the rating of a bridge. Including the parapets in the analysis results in significantly lower GDFs for the exterior girders. Therefore, with the contribution of parapets, higher ratings can be obtained regardless of bridge width, girder stiffness or span length. This effective reserve strength provided by the parapets may be beneficial when considering permit limits for a controlled superload truck passage.

4.4. LOAD DEMAND VERSUS CAPACITY OF THE PARAPETS

In this section, the structural capacity of the parapets and the shear strength of the parapet-deck interface are compared with the maximum load resisted by the parapets. Both the bending moments and the axial forces resisted by the parapets are considered. The superload trucks may generate considerable forces in the parapets. Therefore, investigation of the capacity of the parapets was needed.

4.4.1. LOAD DEMAND ON THE PARAPETS

The bending moments presented here are different than the couples discussed in the previous section, which were calculated with respect to the neutral axis of the entire cross-section. In this section, the bending moment values are obtained directly from SAP2000, and they are relative to the neutral axes of the parapets. Therefore, the eccentricity of the parapets relative to the bridge centroid is not taken into account.

The maximum compressive axial forces and the bending moments resisted by the heavily loaded parapets of the US-52 Bridge, the I-65 Bridge over Ridge Road and the I-65 Bridge over US-30 are presented in Tables 4.6, 4.7 and 4.8, respectively. The maximum values presented in Tables 4.6 to 4.8 were obtained when the trucks were positioned at the shoulder. For the US-52 Bridge, the right lane loading position resulted in axial forces and moments less than half of those for the shoulder loading position. The center loading position for the I-65 Bridges yields negligible parapet forces and moments compared to those for the shoulder loading position. For the I-164 Bridge only with and without parapet cases were investigated and this bridge has discontinuous parapets originally. Furthermore, the evaluated section was at a parapet gap. Therefore, the demand on the parapets of the I-164 Bridge was not evaluated.

4.4.2. CAPACITY OF THE PARAPETS

The parapets of the investigated bridges, with cross-sectional dimensions as given in INDOT's current specifications, were evaluated. The 33-in tall parapet adopted by INDOT has eight #5 Grade 60 longitudinal reinforcing bars, with four bars on each side of the parapet (Figure 4.1). The combined axial force and bending moment capacity of the parapets was estimated using the reinforced concrete column analysis and design software PCA Column (2003). Behavior of the parapets is considered to be similar to that of a column with a very short length because of the bracing provided by the deck.

Strength reduction factors and slenderness effects were not considered for the capacity calculations. The capacity, considering axial-moment interaction, of the 33-in parapet and the maximum demands (unfactored) due to the shoulder loading with superload and HS-20 trucks are presented in Figure 4.10. The load points are labeled from one to three with respect to the truck types. Point 1 is the demand due to the Maximum Superload Truck, Point 2 is the demand due to the Group C Superload Truck, and Point 3 is the demand due to the HS-20 Truck. The other two superload trucks examined herein produced load demands slightly higher than those for the HS-20 Truck. The axial and flexural capacities of the 33-in parapet are 860 kips and 110 ft-k, respectively, for a load eccentricity identical to that for the 824 kip truck. The 45-in parapet had a higher capacity. These capacity values are above the maximum demand caused by the investigated superload trucks, indicating that the parapet has not reached the strength limit state. However, considering the effects of impact and the load and resistance factors, the demand caused by the Maximum Superload Truck or a superload truck heavier than the Maximum Superload truck might be critical for the 33-in parapet.

It should be noted that the load and moment demands shown in Figure 4.10 are for the shoulder loading of the US-52 Bridge. Significantly lower demand values were obtained for a truck located at the right lane.

4.4.3. THE INTERFACE SHEAR TRANSFER-SHEAR FRICTION CAPACITY OF THE PARAPETS

The findings of this study suggest that the parapets can function like girders in resisting the loads. However, this can be true only if the connections to the deck are adequate. The shear transfer capacity of the interface between the deck and the parapet, each cast at different times, was calculated with respect to relevant AASHTO LRFD specifications (AASHTO 2004) which address the interaction between the girders and the deck. Two #5 dowel bars spaced at 8 in were used for the parapets of the US-52 and I-164 Bridges. Similarly, two #7 dowel bars spaced at 8 in were used for the parapets of the I-65 Bridges. For the US-52 Bridge, the shear resistance was calculated for a length of 35 ft from the location of maximum axial force to the beginning of the span. Similarly, the shear resistance was calculated for a section from the midspan to the beginning of the bridge, a length of 59 ft, for the I-65 Bridge over Ridge Road. A total interface shear capacity of 1,750 kips was calculated for the US-52 Bridge and 4,800 kips was obtained for the I-65 Bridge over Ridge Road. Both of these values are significantly greater than the maximum calculated axial force induced by the heaviest superload truck, which was less than 850 kips (Tables 4.6 to 4.8).

It is notable that the 33-in parapet, which is the smallest parapet used by INDOT, has adequate capacity to resist the load demands due to most of the superload trucks. The 45-in parapet has a much higher capacity than the 33-in parapet. Therefore, it can be stated that the parapets of both bridges have sufficient strength to resist the axial forces and bending moments generated by heavy superload trucks, as well as adequate shear transfer capacity at the interface with the deck.

4.5. EFFECTS OF PARAPETS IN THE NEGATIVE MOMENT REGION

Parapet discontinuities can result in increased girder moments and bottom flange stresses in the positive moment region. However, in the negative moment region of a bridge, parapet discontinuities can lead to higher tensile stresses in the deck. Effects of the parapets on the continuous US-52 Bridge and the I-65 Bridge over US-30 were investigated. These two bridges had distinct characteristics in terms of girder material and stiffness properties as well as the parapet height.

The FEMs of the US-52 Bridge and the I-65 Bridge over US-30 with the ordinary parapets had continuous parapets along each span with a one inch gap between the parapets over the pier (Figure 4.3). The maximum tensile deck stresses in the longitudinal direction obtained over the first internal support of the US-52 Bridge due to the superload and HS-20 trucks positioned in the right lane and the shoulder are presented in Table 4.9 for the following parapet cases: ordinary 33-in parapet (OP), continuous 33-in parapet (CP), discontinuous 33-in parapet (DP) and no parapet (NP). The CP case has continuous parapets throughout the entire bridge length. The maximum tensile stresses obtained in the deck of the I-65 Bridge over US-30 for the ordinary 45-in parapet (OP) and no parapet (NP) cases are presented in Table 4.10.

If the trucks are located on the shoulder, for all cases except the CP case, the highest longitudinal deck stresses are obtained. Since material non-linearity and cracking of the concrete deck were not included in the model, stresses higher than the modulus of rupture (approximately 475 psi according to ACI, 2005) were obtained. Regardless, the linear elastic analysis reveals that higher stresses can be expected due to the resulting stress concentrations from parapet discontinuities. For the ordinary parapet and discontinuous parapet cases the deck stresses over the pier can be six times higher than those for the no parapet case when the trucks are located at the shoulder, and about three times higher when the trucks are located at the center of the right lane. Clearly, the parapet gap can function as a stress raiser over the piers. This suggests that continuous

parapets may be preferred, since they can prevent possible stress concentrations and extend the service life of the deck.

More important than the magnitude of stresses obtained from a linear analysis, the ratio of the deck stresses should be considered in the evaluation of the effects of parapet gaps in the negative moment region. The deck might be already cracked, but the extent of cracking might become widespread due to the notch effect of the discontinuity.

The maximum tensile parapet stresses in the negative moment region due to bending and axial effects for the right lane and shoulder loading positions of the US-52 Bridge are presented in Table 4.11. Stresses were found to range from 0.2 to 1.8 ksi for the US-52 Bridge analyzed with ordinary parapets (OP) when the superload trucks were positioned in the shoulder. For the continuous parapet case (CP), the stresses were approximately two times higher for the same loading position. On the other hand, the average reductions in the maximum tensile parapet stresses were 50 and 70% for OP and CP cases, respectively, compared to shoulder loading when the trucks were positioned at the center of the right lane. These results suggest that the negative bending moment demand on the continuous parapets can be damaging, and certainly will cause cracking or extend the existing cracks, when the loads are positioned in the shoulder. However, repair or replacement of the parapets may be easier than comparable maintenance operations for the deck.

4.6. SUMMARY

Parapet behavior and performance was studied for four vastly different bridges. The geometries of these bridges provide a wide range of parameter behavior with respect to bridge width, girder spacing and girder depth for common continuous and single span slab on steel and PC girder bridges with no skew. The following observations and conclusions can be made:

- Parapets can significantly contribute to the stiffness of a bridge and result in an increased load rating for a bridge. An increased parapet stiffness attracts more load to the parapet and results in lower demands for the girders.
- Depending on the girder and parapet properties, the girder distribution factors can decrease up to 30% compared to the no parapet case when the parapets are included in the analysis. The decreases in the girder distribution factors are highest for exterior girders and shoulder loading position. Since exterior girders typically control the capacity of a bridge, a lower demand for the exterior girders improves the rating of a bridge.
- The required shear resistance at the parapet-deck interface is easily obtained by a regular spacing of dowel bars for typical parapet details.
- In the positive moment region, crushing of concrete or yielding of the longitudinal or transverse reinforcement in the parapet is not probable for loading by most typical superload trucks. However, stresses higher than the cracking stress in the parapets is likely in the negative moment regions.
- Discontinuities in parapets can result in significant stress concentrations. In the positive moment region, the bending moments carried by the exterior girders and their bottom flange stresses are amplified due to a parapet notch effect. In the negative moment region, the effects of discontinuities can be more dramatic. Continuous parapets can eliminate possible stress concentration problems and decrease cracking of the deck in the negative moment region.

Table 4.1 Comparison of the GDFs for the US-52 Bridge analyzed with 33-in parapets

Loading Position	Truck Type	GDF (%)		$\Delta\text{GDF} / \text{GDF}_{\text{NP}} (\%)$
		OP	NP	$(\text{GDF}_{\text{OP}} - \text{GDF}_{\text{NP}}) / \text{GDF}_{\text{NP}}$
Right Lane	Maximum Truck	17	18	-9
	Group C	18	19	-8
	Group B	17	19	-8
	Group A	21	22	-5
	HS-20	20	21	-6
Shoulder	Maximum Truck	19	28	-32
	Group C	20	29	-33
	Group B	20	30	-33
	Group A	21	30	-28
	HS-20	21	29	-29

Table 4.2 Comparison of the GDFs for the US-52 Bridge analyzed with 45-in parapets

Loading Position	Truck Type	GDF (%)		$\Delta\text{GDF} / \text{GDF}_{\text{NP}} (\%)$
		OP	NP	$(\text{GDF}_{\text{OP}} - \text{GDF}_{\text{NP}}) / \text{GDF}_{\text{NP}}$
Right Lane	Maximum Truck	16	18	-12
	Group C	17	19	-10
	Group B	17	19	-10
	Group A	20	22	-7
	HS-20	19	21	-8
Shoulder	Maximum Truck	16	28	-42
	Group C	17	29	-43
	Group B	17	30	-44
	Group A	19	30	-36
	HS-20	19	29	-37

Table 4.3 Comparison of the GDFs for the I-65 Bridge over Ridge Road analyzed with 46-in parapets

Loading Position	Truck Type	GDF (%)		$\Delta\text{GDF} / \text{GDF}_{\text{NP}} (\%)$
		OP	NP	$(\text{GDF}_{\text{OP}} - \text{GDF}_{\text{NP}}) / \text{GDF}_{\text{NP}}$
Center	Maximum Truck	21	21	0
	Group C	22	22	0
	Group B	23	23	0
	Group A	25	25	0
	HS-20	24	24	0
Shoulder	Maximum Truck	35	45	-22
	Group C	38	48	-22
	Group B	37	48	-23
	Group A	39	50	-22
	HS-20	38	49	-22

Table 4.4 Comparison of the GDFs for the I-164 Bridge analyzed with 33-in parapets

Loading Position	Truck Type	GDF (%)		$\Delta\text{GDF} / \text{GDF}_{\text{NP}} (\%)$
		DP	NP	$(\text{GDF}_{\text{NP}} - \text{GDF}_{\text{DP}}) / \text{GDF}_{\text{NP}}$
Right Lane	Maximum Truck	33	34	+1
	Group C	33	33	+1
	Group B	32	32	+1
	Group A	35	36	+1
	HS-20	33	33	+1
Shoulder	Maximum Truck	36	39	+8
	Group C	44	40	+8
	Group B	45	41	+10
	Group A	43	40	+7
	HS-20	44	40	+9

Table 4.5 Comparison of the GDFs for the I-65 Bridge over US-30 analyzed with 45-in parapets

Loading Position	Truck Type	GDF (%)		$\Delta\text{GDF} / \text{GDF}_{\text{NP}} (\%)$
		OP	NP	$(\text{GDF}_{\text{OP}} - \text{GDF}_{\text{NP}}) / \text{GDF}_{\text{NP}}$
Center	Maximum Truck	33	33	-2
	Group C	35	35	-2
	Group B	35	36	-2
	Group A	36	37	-2
	HS-20	37	38	-2
Shoulder	Maximum Truck	39	48	-20
	Group C	43	54	-20
	Group B	43	54	-20
	Group A	44	54	-20
	HS-20	44	55	-20

Table 4.6 The maximum axial forces and bending moments resisted by the 33-in parapets of the US-52 Bridge

Loading Position	Truck Type	Axial Force (kip)	Bending Moment (ft-k)
Right Lane	Maximum Truck	355	28
	Group C	105	8
	Group B	80	6
	Group A	67	5
	HS-20	47	3
Shoulder	Maximum Truck	768	98
	Group C	247	33
	Group B	190	25
	Group A	173	22
	HS-20	118	15

Table 4.7 The maximum axial forces and bending moments resisted by the 46-in parapets of the I-65 Bridge over Ridge Road

Loading Position	Truck Type	Axial Force (kip)	Bending Moment (ft-k)
Center	Maximum Truck	-20	-7
	Group C	-6	-2
	Group B	-4	-1
	Group A	-4	-1
	HS-20	-3	-1
Shoulder	Maximum Truck	843	131
	Group C	267	43
	Group B	178	29
	Group A	162	25
	HS-20	144	23

Table 4.8 The maximum axial forces and bending moments resisted by the 45-in parapets of the I-65 Bridge over US-30

Loading Position	Truck Type	Axial Force (kip)	Bending Moment (ft-k)
Center	Maximum Truck	3	-2
	Group C	0	0
	Group B	0	0
	Group A	1	0
	HS-20	4	-6
Shoulder	Maximum Truck	526	-21
	Group C	175	-4
	Group B	128	-4
	Group A	135	-3
	HS-20	76	-2

Table 4.9 The maximum tensile deck stresses over the first pier of the US-52 Bridge

Loading Position	Truck Type	σ_x (ksi)			
		OP	CP	DP	NP
Right Lane	Maximum Truck	2.1	0.8	3.0	0.9
	Group C	1.3	0.4	1.3	0.4
	Group B	0.9	0.3	0.9	0.3
	Group A	0.5	0.3	0.5	0.3
	HS-20	0.3	0.1	0.4	0.1
Shoulder	Maximum Truck	7.5	0.6	7.7	1.2
	Group C	4.3	0.3	4.4	0.7
	Group B	3.1	0.2	3.2	0.5
	Group A	1.9	0.2	2.0	0.3
	HS-20	1.0	0.1	1.0	0.2

Table 4.10 The maximum tensile deck stresses over the pier of the I-65 Bridge over US-30

Loading Position	Truck Type	σ_x (ksi)	
		OP	NP
Center	Maximum Truck	1.8	1.8
	Group C	0.8	0.8
	Group B	0.6	0.6
	Group A	0.6	0.6
	HS-20	0.2	0.2
Shoulder	Maximum Truck	3.8	2.1
	Group C	2.1	1.2
	Group B	1.5	0.9
	Group A	0.9	0.6
	HS-20	0.5	0.2

Table 4.11 The maximum tensile parapet stresses in the negative moment region of the US-52 Bridge

Loading Position	Truck Type	σ_x (ksi)		
		OP	CP	DP
Right Lane	Maximum Truck	1.0	1.0	0.8
	Group C	0.3	0.5	0.2
	Group B	0.2	0.3	0.2
	Group A	0.2	0.2	0.1
	HS-20	0.1	0.1	0.1
Shoulder	Maximum Truck	1.8	2.9	1.6
	Group C	0.8	1.6	0.7
	Group B	0.5	1.2	0.5
	Group A	0.3	0.8	0.4
	HS-20	0.2	0.4	0.2

Figure 4.1 Cross-section of the 33-in parapet used by INDOT (INDOT, 2006)

Figure 4.2 Cross-section of the 45-in parapet used by INDOT (INDOT, 2006)

Figure 4.3 Parapet discontinuity (gap) in the FEM

Figure 4.4 Strain distribution of the bridge cross-section, and axial force and bending moment resisted by the parapet

Figure 4.5 Bending moment distributions across the cross-section of the US-52 Bridge for the Group C Superload Truck for ordinary parapet (OP), discontinuous parapet (DP) and no parapet (NP) cases

Figure 4.6 Stresses measured in the parapet of the I-65 Bridge over Ridge Road for Load Case 2 of the load test

Figure 4.7 Bending moment distributions across the cross-section of the I-65 Bridge over Ridge Road for the Group C Superload Truck for ordinary parapet (OP) and no parapet (NP) cases

Figure 4.8 Bending moment distributions across the cross-section of the I-164 Bridge for the Group C Superload Truck for discontinuous parapet (DP) and no parapet (NP) cases

Figure 4.9 Bending moment distributions across the cross-section of the I-65 Bridge over US-30 for the Group C Superload Truck for original parapet (OP) and no parapet (NP) cases

Figure 4.10 P-M interaction diagram for the 33-in parapet of the US-52 Bridge

CHAPTER 5. A SIMPLE METHOD TO PREDICT THE 3-D LIVE LOAD RESPONSE OF SLAB-ON-GIRDER BRIDGES

In the current bridge design methodology, beam line analysis and girder distribution factors are often used to compute the maximum live load demands on the girders. However, this method does not predict the response of all girders under live load. Furthermore, it has been found to be conservative compared to 3-D analysis results and load tests of bridges. With the proposed simple method, it is possible to closely approximate the 3-D live load response of bridges with accuracy comparable to that from a detailed finite element analysis. For a given truck position, the bending moment resisted by each girder and individual girder deflections can be computed easily. The live load demand imposed on the secondary members, such as diaphragms or cross-frames, can also be predicted using the results of the proposed method. This analysis method can be used for any truck configuration. However, given its accuracy and flexibility in terms of transverse positioning of trucks and parapet effects, this method is expected to improve the rating of bridges for superloads, and therefore is important for the analysis of bridges subjected to superloads.

Bridge design and rating often involves use of a simple beam line analysis for the structural analysis. A typical bridge is idealized as a one-dimensional beam. Then, the live load in the form of a prescribed truck configuration is applied to the beam model. Maximum bending moment and shear values are obtained by positioning the truck at the most critical locations. After this step, the design bending moment and shear values are obtained by multiplying the girder distribution factors (GDF) times the maximum moment or shear value. The GDF represents the fraction of the total moment or shear that

is distributed to one beam or girder. This method can only be utilized for the calculation of maximum demand on the girders. Strength of the traditional method comes from its simplicity with respect to a detailed finite element analysis. The GDF equations (AASHTO, 2004), which were derived after comprehensive studies (Zokaie, 2000), are found to be conservative for many bridges (Sotelino et al., 2004). Furthermore, the complicated GDF equations require computation of several parameters.

The beam-line method is sufficient for bending moment and shear design, but the forces developed due to the interaction of the girders with the bracing members cannot be predicted with this method. The beam-line approach does not provide any information about the deflection of girders; an advanced structural analysis method like 3-D finite element analysis (FEA) must be used by researchers to investigate distortion induced problems. A three dimensional analysis can better describe the structural behavior, but it requires significant preparation and computational effort. A simple, but accurate, method is needed to fill the gap between these two approaches. With the proposed method, it is possible to obtain the amount of load carried by each girder and the deflected shape of the transverse cross-section at the location of interest. From the deflected shape, the interaction of diaphragms or cross-frames with the girders, and the resulting stresses, can be inferred.

A simplified analysis method is explained in the following sections. The results obtained by the simplified analysis method are compared with the ones obtained from 3D finite element analyses. In addition to the four Indiana bridges described in Chapters 2 and 3, several bridges investigated by other researchers were studied for the purpose of validating the proposed method.

5.1. DESCRIPTION OF THE PROPOSED METHOD

The underlying principles and derivation of the proposed method are presented in this section. A bridge structure consists of several composite or non-composite girders. All of these girders resist bending moments due to live load. The girders which are close to the truck or the wheel lines carry more load than the other girders, and hence deflect more. The distribution mechanism of live load is primarily a function of the girder spacing and stiffness of the girders. This relationship can be inferred from the GDF equations in which the girder spacing and the parameters related to the stiffness of the girders have greater influence (AASHTO, 2004).

For the proposed simplified method, the bridge cross-section is idealized as a system of parallel rotational springs which represent the girders. Each spring element is connected to the adjacent spring elements with flexible torsion bars as shown in Figure 5.1. The torsion bars behave like the deck of a bridge connecting girders to each other. Springs are attached to pin supports. The truck load is applied to the system in the form of torsion at the wheel line locations, with the applied force resisted by the rotational springs. These springs are one-dimensional and resist only rotation, corresponding to bending of the girders.

The stiffness of a rotational spring is proportional to the flexural stiffness of the girder it represents. Springs resist the moments applied at the wheel lines of a truck in the same way that girders resist the bending moment generated by the truck. Since the main concern is distribution of load and calculation of how much load will be carried by each girder, only relative flexural stiffness values are required in the calculations. Further simplification of the procedure can be obtained for a typical bridge with all girders the same size; each interior girder can be represented with a rotational spring which has unit stiffness (one). Similarly, the exterior girders can be represented by spring elements whose stiffness is equal to the ratio of the moment of inertia of the exterior girder to the moment of inertia of the interior girder. For composite girders, the stiffness of an exterior

spring will typically be less than one, since the effective slab width, including an overhang, of an exterior girder is less than that of an interior girder. Interior girders with irregular stiffness or at an irregular spacing (i.e. slab width) can also be modeled using relative moment of inertia ratios.

Having determined the stiffness of the rotational spring elements, determining the stiffness of the torsion bars is the next step. The stiffness of the torsion bars play an important role in the distribution of bending moments between the girders. If a very rigid bar is placed between the rotational springs, load would be distributed equally between the springs (girders). On the other hand, if very flexible bars with relatively low stiffness values are used between the springs, the total live load will be carried by the two or three girders (springs in the model) closest to the wheel lines. The appropriate torsion bar stiffness is the value which ensures proper load distribution. It should be between the aforementioned upper and lower bounds.

Since torsion bar stiffness is the major parameter affecting the load distribution, it should be a function of the girder spacing and the girder stiffness. From the three-dimensional analyses of two steel slab-on-girder bridges in Indiana (US-52 and I-65 Bridges), a formula was obtained for the calculation of torsion bar stiffness. These two bridges have significantly different widths, girder depths and spacing as shown in Chapter 2. They are believed to represent a range for many bridges in Indiana. Therefore, the torsion bar stiffness equation determined with respect to these two bridges is expected to be valid for a range of bridges. Stiffness of the torsion bars is determined by Equation 5.1:

$$K_{TM} = \frac{2.8}{\left(\frac{S}{72}\right)^2} \quad (\text{Eqn. 5.1})$$

where S is the girder spacing in inches and K_{TM} is the stiffness of the torsion bar to be used in the simple structural model for moment distribution. Equation 5.1 gives the K_{TM} value in terms of the interior rotational spring stiffness, which was unity. Note that the units used for stiffness expressions in the model do not make a difference provided that the units are consistent. The constant “2.8” in Equation 5.1 was obtained by calibration of the model with respect to the analyzed bridges. This constant can be modified to account for span length, bracing type and spacing, support conditions and other factors which may affect the load distribution characteristics but are beyond the scope of this study.

Placement of the rotational springs at the girder locations enables proper positioning for analysis. Use of real dimensions for girder spacing allows for transverse truck placement without scaling of any dimensions. After the model is complete, the truck load should be divided into two and applied to the desired transverse location. If two or more trucks are present on the bridge, and they are to be positioned side by side to create the most critical loading, and moments due to each truck should be applied at the truck locations in the same manner as for the single truck case.

For direct evaluation of the girder distribution factors, a bending moment of one half (in any unit system) is applied at each wheel line location to the structural model consisting of rotational springs and torsion bars as shown in Figure 5.2. Then, the rotational spring reactions are obtained from the structural analysis. Each spring reaction equals the GDF of that girder.

From the simple model, one can obtain the girder moments. This is achieved by multiplication of the calculated GDFs with the total bending moment at the longitudinal location of interest. The total moment can be calculated with a simple beam line analysis of axle loads on a one-dimensional beam (Figure 5.3). The cross-section of this one-dimensional beam represents all longitudinal members of the bridge including girders, deck and parapets. Therefore, its structural properties, such as the moment of inertia and the shear area, should be equal to the sum of the properties of all longitudinal members.

Also, the span lengths must be equal to the original span lengths of the bridge, with appropriate support conditions. The bending moment and average deflection obtained from the beam line analysis is distributed to individual girders through the distribution coefficients. The effect of shear deformations may be negligible for many cases, but neglecting them can lead to an underestimation of the deflection calculations.

Vertical displacements of the girders can be obtained in a manner similar to the calculation of moments for girders. However, the three dimensional finite element analyses of the investigated bridges indicate that the distribution of displacements is somewhat different than the distribution of bending moments across the girders. Therefore, another torsion bar coefficient should be utilized for determining the distribution of displacements. Modifying the constant term “2.8” in Equation 5.1 to “3.8” gave the best results and resulted in the following equation for distribution of displacements:

$$K_{TD} = \frac{3.8}{\left(\frac{S}{72}\right)^2} \quad (\text{Eqn. 5.2})$$

In Equation 5.2, S is the girder spacing in inches and K_{TD} is the stiffness of torsion bars to be used in the simple structural model for girder displacement distribution. The K_{TD} value is in terms of the interior rotational spring stiffness, as was K_{TM} (Equation 5.1). The only differences between procedures for obtaining moment distribution and deflections is the use of different torsion bar stiffness properties and the use of spring rotations, rather than the spring reactions, for distribution coefficients. The underlying principle for obtaining individual girder displacements as a function of the total displacement stems from the observation that the sum of the girder displacements was approximately constant and independent of transverse truck location for the US-52 and I-65 Bridges.

For the calculation of individual member deflections, the deflection value obtained from the beam line analysis is first multiplied by the sum of spring coefficients used in the spring analogy model. Then, this total deflection value is distributed to the girders by the spring rotations obtained from the spring analogy model. The beam line analysis to be used in conjunction with spring analogy (SA) includes all longitudinal members of the bridge. Therefore, it gives an average deflection value, as if a uniform load had been applied across the width of the bridge deck. The deflection distribution coefficients from the spring analogy model assure proper transverse distribution of the total deflection among the girders.

Combination of a traditional beam line analysis with the proposed spring analogy method results in simple calculations for bending moment and displacement of each girder with an accuracy comparable to that of a 3-D analysis. Girder moments and displacements obtained by this method for the investigated Indiana bridges are compared with the ones obtained by three dimensional finite element analyses in the following sections. Sample calculations show the advantages and capabilities of the proposed method in addition to illustrating the procedure. Incorporating the effects of support conditions and the secondary members such as cross frames on the girder bending moment and displacement distributions requires further investigation and will not be addressed in this study. However, the parapets, which act like flexural members and affect the load distribution, as discussed in Chapter 4, can be included in the spring analogy model without any difficulty, as will be shown.

5.2. SAMPLE CALCULATIONS AND COMPARISON WITH THE DETAILED FEA RESULTS

Girder moments and displacements were calculated for the two steel and two prestressed concrete (PC) bridges presented in Chapters 2 and 3. The procedure will be explained step by step to demonstrate the proposed method with the two typical,

composite steel girder bridges. For the PC bridges described in Chapter 3, only the comparison of results will be presented.

The first bridge, on US-52, is a five-span continuous structure with rolled shapes. The other bridge, on I-65 over Ridge Road, is a one-span integral abutment bridge. Plate girders are used to support the composite reinforced concrete deck. Properties of both bridges were presented in Chapter 2. For comparison purposes, all bridges were modeled following the design assumptions. Therefore, abutment fixity, secondary members such as cross-frames and parapets, and deck cracking were not included in the analysis. Only the HS-20 Truck was used in the sample calculations, since the GDF due to the HS-20 Truck was higher or comparable to the GDF for most of the superload trucks.

5.2.1. CALCULATIONS FOR THE US-52 BRIDGE

This five span continuous structure has eight beams. The interior beams are spaced at 6 ft and the exterior beams are 5 ft apart from the nearest interior beam. The truck loads were only applied to the first steel span and the calculations were made accordingly in this example. Composite and non-composite moment of inertias of the beams were calculated with respect to the dimensions given in the pertinent AISC code (1963) and the effective concrete flange width. Non-composite and composite moments of inertia were used as specified in the framing plan along the length of the bridge (Figure 5.3). Transformed moment of inertia calculations were repeated for the exterior beams. Then, the sum of the transformed moments of inertia of all beams was calculated. Calculated moment of inertia values were assigned to the corresponding regions of the beam line model. The transformed moments of inertia were calculated with respect to the elastic modulus of steel. Therefore, elastic modulus of steel was used in the analysis. The simple beam model (Figure 5.3) was analyzed with SAP2000 (2001).

In the simple beam model, roller supports were used for the rocker bearings and a pin support was used for the fixed support at the first interior pier location. The pin support used in the beam line model did not simulate the original support condition of the bridge, since the supports were assigned to the neutral axes of the beams in the beam line model. Therefore, the rotational constraint action of the fixed support was not considered in the beam line analysis. The same support condition, without any rotational restraints, was also used in the three dimensional analysis.

The HS-20 Truck with axle weights of 32 kips, 32 kips and 8 kips was applied to the first span of the beam line model. The beam line model and the applied truck load are presented in Figure 5.3. In Figure 5.3, dots are the joints at which composite and noncomposite sections are connected to each other or the joint loads and boundary conditions are assigned.

The maximum bending moment was obtained at the location of middle axle which was located at 46.5 ft from the first exterior support. The calculations throughout this example are for this section only, since it was the most critical longitudinal location. Other sections along the bridge could be analyzed similarly if needed. The total bending moment and the average vertical deflection at this section were 1,136 ft-kip and 0.31 in, respectively. The next step is to laterally distribute these two parameters to the girders along the cross-section.

A simple structural model as shown in Figure 5.1 was constructed for determining the distribution factors. Properties of the structural elements (rotational springs and torsion bars) and the transverse location of the truck were first established. The truck load in terms of moment was applied at the selected wheel line locations. For this example, the transverse location of the truck was selected as the middle of the right lane and the spacing between the centers of wheel lines was taken as 72 in as specified by AASHTO (2004) for the HS-20 Truck. The conceptual model is presented in Figure 5.2. Notice that the eight rotational springs in Figure 5.2 represent the eight beams and the interior spring

spacing is 6 ft. The moments applied to the torsion bar at the wheel line locations were each half of a unit moment such as 0.5 in-k or 0.5 ft-k. The spring reactions obtained from the structural analysis of the model were used directly as the girder distribution coefficients. The distribution coefficients were multiplied by the total bending moment of 1136 ft-k to obtain the beam moments.

The rotational stiffness values of the spring elements used in the model were calculated with respect to the beams' moments of inertia. Since the selected longitudinal section was located at 46.5 ft, the composite moment of inertias were used in the spring analogy calculations. The composite moment of inertia for the interior beam, which has an effective concrete flange width of 6 ft, was $21,645 \text{ in}^4$. The exterior beam had a composite moment of inertia of $18,780 \text{ in}^4$. The internal rotational spring stiffness was unity. The exterior spring stiffness was the ratio of the composite moment of inertia of the exterior beam to that of the interior beam. Consequently, stiffness for the exterior rotational springs was 0.87.

The next step was to determine the stiffness values of the torsion bars for the model according to Equation 5.1 for moment distribution and Equation 5.2 for deflection distribution. For a beam spacing of 6 ft, K_{TM} and K_{TD} were 2.8 and 3.8, respectively. Recall that the units of moments and stiffness coefficients do not make any difference in the calculations as long as they are consistent. Structural analysis of the model shown in Figure 5.2 was conducted with SAP2000. Ordinary beam elements with six degrees of freedom and linear spring elements available in SAP2000 were used. Torsion bar stiffness (GJ/L) was established by assigning G and J values which gave the required values (obtained from Equation 5.1 or 5.2). The length L in the stiffness equation (GJ/L) was equal to the interior beam spacing (S), which was 6 ft. The shear modulus (G) was assigned as 1 kip/ft^2 and the torsional constant (J) was determined to be 6 ft^4 for K_{TM} . Note that the values of G and J were chosen arbitrarily such that their product gives the required K_{TM} value when divided by the length (L).

The resulting spring reactions and rotations translate into bending moment and displacement distribution factors and are presented in Table 5.1. Note that there is a noticeable difference between the distribution coefficients for moment and deflection. The distribution coefficient for the maximum bending moment is 0.23, whereas for the maximum deflection, the distribution coefficient is 0.21.

The bending moment and deflection corresponding to each beam were calculated by the multiplication of each of these distribution coefficients with the total bending moment and deflection values obtained from the beam line analysis. The deflection obtained from the beam line analysis was an average displacement value. Therefore, for the calculation of beam deflections, the deflection obtained from the beam line analysis was multiplied first by the sum of spring constants used in the model (7.91 for the US-52 Bridge) to obtain the total deflection and then by the distribution coefficient for each beam. The spring analogy (SA) results for the bending moments and deflections are presented in Tables 5.2 and 5.3.

For comparison, a finite element analysis of the US-52 Bridge was conducted following the general principles described in Chapter 2. The detailed FEM included all structural elements of the bridge. However, the secondary elements were not included in the model. At the noncomposite regions of the bridge, the deck elements were deleted in order to avoid inadvertent transfer of forces through membrane action. Also, the supports in the FEM were modeled like the supports of the beam line model. The pin support over the first pier was modeled as a combination of the roller support and a longitudinal spring with negligible stiffness. This support pattern provided stability in the longitudinal direction without generating any rotational restraint at the first interior support location. The other supports were modeled as ordinary roller supports.

The truck load was applied to the center of the travel lane in the FEM. This transverse location matched the transverse location of the applied bending moments in the spring analogy model (Figure 5.2). Also, the longitudinal location of the truck in the

FEM was the same as the longitudinal truck location of the beam line model as discussed previously. FE results for bending moments at the middle of the first span (46.5 ft) and deflections of the girders at the same location are presented in Tables 5.2 and 5.3.

The bending moments and displacements, presented in Tables 5.2 and 5.3, are also compared in Figures 5.4 and 5.5. In addition to prediction of the maximum GDF (0.23) or maximum deflection (0.52 in) comparable to those of the FEA, distribution of the bending moments and deflections were computed with a reasonable accuracy using spring analogy method. Deflections and bending moments of the critical girders were computed accurately. Error was within 1% for both cases. Deviation from the FEA results is observed for the girders which were not heavily loaded. Therefore, this example proves that the spring analogy method can predict the response of US-52 Bridge effectively. Given its simplicity, the spring analogy method has an acceptable level of accuracy for bridges similar to the US-52 Bridge in terms of girder spacing and span properties. Considering that welded W-shape diaphragms might be present on similar bridges, it is expected that the spring analogy method can enable computation of relative deflections for estimation of the stresses in the web-gap region, as discussed in Chapter 2.

5.2.2. CALCULATIONS FOR THE I-65 BRIDGE OVER RIDGE ROAD

The I-65 Bridge over Ridge Road has 10 plate girders spaced at 11.2 ft which are composite with the reinforced concrete deck. In the structural analysis, effects of the integral end abutment, the cross-frames and the parapets were not taken into account. The bridge was modeled as a simple span. This approach is conservative, but consistent with the beam line analysis and the design assumptions (AASHTO, 2004).

The beam line analysis of the I-65 Bridge over Ridge Road was conducted in a similar manner as for the US-52 Bridge. The sum of the moments of inertia of all girders and the sum of their shear areas were used in the beam line analysis. Beam line analysis

of the I-65 Bridge over Ridge Road gave a maximum bending moment of 2393 ft-kip and an average deflection of 0.14 in at the midspan.

A structural model like the one shown in Figure 5.2 was developed for a girder spacing of 11.2 ft. The ten plate girders were represented by ten springs. Bending moments due to the HS-20 truck were applied to the center of the model with a wheel line spacing of 72 in. The stiffness for each interior spring was 1, and the exterior spring stiffness was 0.93 (ratio of the exterior girder's moment of inertia, 140,682 in⁴, to the interior girder's moment of inertia, 150,592 in⁴). The stiffness values of the torsion bars were calculated from Equations 5.1 and 5.2 for bending moment and displacement distributions. The K_{TM} value was 0.81 and K_{TD} value was 1.10. The bending moment and deflection distribution coefficients computed by the spring analogy method for the midspan of the bridge are presented in Table 5.4.

The total bending moment, 2,393 ft-k, obtained from the beam line analysis was multiplied with the distribution coefficients presented in Table 5.4 to obtain the bending moments resisted by each girder. For the girder displacements, the average displacement obtained from the beam line analysis was multiplied by the sum of the spring coefficients (9.86) and then this value was multiplied by the displacement distribution coefficients (Table 5.4). Bending moments and girder deflections calculated with the spring analogy method are presented in Tables 5.5 and 5.6.

The FEM of the I-65 Bridge was described in Chapter 2. The HS-20 Truck was positioned at the center of the FEM in both the longitudinal and transverse directions. The axle spacing and weights were the same as used in beam line analysis. FEA results for the bending moments and deflections are presented in Tables 5.5 and 5.6. With respect to the results presented in Table 5.5, the spring analogy method overestimates the maximum bending moment resisted by the most heavily loaded girders (Girders 5 and 6) by about 14%. For the maximum deflections obtained at Girders 5 and 6 (Table 5.6), the spring analogy method results are 4% higher than the FEA results. The relatively higher

error observed for the I-65 Bridge is believed to be a result of its longer span length relative to the US-52 Bridge. However, the error is within acceptable limits for a simple analysis technique and it is conservative. Bending moment and displacement distributions of the girders are compared in Figures 5.6 and 5.7.

Even though the spring analogy method slightly over-predicts the bending moments and displacements for the girders of the I-65 Bridge over Ridge Road, it captures the general trend obtained from FEA, as shown in Figures 5.6 and 5.7, and approximates 3-D FEA results reasonably well.

5.2.3. CALCULATIONS FOR THE I-164 BRIDGE

The FEM of the I-164 Bridge was not refined in terms of material properties and actual torsional stiffness of the girders because of lack of experimental data to which to compare the results. The only modification to the original FEM described in Chapter 3 was the removal of the parapets. The HS-20 Truck was positioned at the center of the right lane. Analysis of the spring analogy model including all seven girders resulted in the distribution factors presented in Table 5.7. Girder moments and deflections calculated from the FEA and spring analogy method are presented in Tables 5.8 and 5.9, accordingly. Furthermore, the bending moments and deflections obtained by the spring analogy method are compared with the FEA results in Figures 5.8 and 5.9.

Comparison of the spring analogy results with the FEA results indicate that spring analogy method gives somewhat unconservative results for the I-164 Bridge. This is believed to be a result of the short span length and continuity as well as some modeling issues due to use of the eccentric beam modeling technique. In Chapter 3, it was shown that the FEM of the I-65 Bridge over US-30 was overestimating the GDFs compared to the experimental data. Therefore, a similar condition may also exist for the I-164 Bridge. However, lack of experimental data for this bridge raises some concerns about

applicability of the method to short span bridges. This issue will be further discussed in the following sections.

5.2.4. CALCULATIONS FOR THE I-65 BRIDGE OVER US-30

The I-65 Bridge over US-30 was analyzed and instrumented as discussed in Chapter 3. Some of the refinements to the FEM were not included in the FEM used for comparison purposes in order to be consistent with the design assumptions. Therefore, abutment fixity, contribution of parapets and experimentally anticipated level of continuity were not considered in the analysis. However, torsional rigidity of the bulb-tee girders were adjusted as described in Chapter 3 in order to overcome shortcomings of the eccentric beam modeling technique. Loading was similar to the other analyzed bridges; the HS-20 Truck was positioned at the center of the bridge in the transverse direction for both the FE and spring analogy models. Eleven bulb-tee girders of the bridge spaced at 9 ft were included in the spring analogy model, and the distribution coefficients presented in Table 5.10 were calculated. Girder moments and deflections calculated using these distribution coefficients and results from the FEA are presented in Tables 5.11 and 5.12, respectively.

The spring analogy method overestimates the maximum bending moment resisted by Girder 6 by about 11%, as shown in Figure 5.10 and Table 5.11. The same conclusion applies to the deflections presented in Figure 5.11 and Table 5.12. However, the spring analogy model captures the general behavior fairly well except for overestimation of the maximum moment and the maximum deflection. Nevertheless, the error margin of spring analogy (11%) is significantly lower than that of the AASHTO (2004) GDF equation (36%) for this bridge and spring analogy mimics the FEA.

5.3. APPLICATION OF THE PROPOSED METHOD TO OTHER STUDIES

Results from bridges investigated by other researchers were used to further validate the spring analogy method. In this section, GDFs and the distribution of the bending moments presented in the literature are compared with the GDFs obtained with the spring analogy method. Testing of the proposed analysis technique on a more diverse database of bridges was done to increase confidence in its accuracy.

The main input parameters needed for the spring analogy method are the girder spacing and the truck location. The stiffness of each interior spring element is taken as unity, since the interior girders generally have equal moment of inertia values. The only parameter which requires more information about the bridge is the ratio of the stiffness of the exterior girder to the interior girder's stiffness. Properties of the girders and deck for most of the bridges were not presented in the publications forming the benchmark database. Therefore, exterior to interior girder stiffness ratios were estimated. Most of the bridges have a comparable overhang width, and the exterior to interior girder stiffness ratio is typically less than one. Therefore, the stiffness of the exterior spring element was selected as 0.9 based on experience from the analysis of bridges presented in Chapters 2 and 3. This value is believed to be acceptable when properties of a bridge were not available. The difference between the actual value and the assumed value is expected to be around 5% for most bridges based on the properties of the analyzed steel and PC bridges. Spacing of girders and transverse truck positions for benchmark cases were obtained from two comprehensive studies on GDFs. It should be noted that no more information was required to set up the spring analogy model for each bridge. The first study was conducted by Eom and Nowak (2001), and the second one was conducted by Sotelino et al. (2004).

5.3.1. EOM AND NOWAK'S (2001) STUDY

The objective of Eom and Nowak's study (2001) was to determine the GDFs for steel girder bridges through an experimental program and detailed 3-D FEA. Eom and Nowak (2001) reported that 17 single span steel girder bridges were instrumented and three dimensional FEA of these bridges were conducted using the commercial software ABAQUS. Selected bridges were constructed between 1922 and 1986. Five bridges had nonzero skew angles. The number of girders varied from 6 to 12. The span length, number of girders, girder spacing, skew angle and date of construction of the investigated bridges are presented in Table 5.13.

The bridges were loaded by two 11-axle Michigan trucks for the field tests and the FEMs. Although the weight of the test trucks were not mentioned explicitly, it was stated that 11-axle trucks can weigh up to 164 kips in Michigan. Therefore, these trucks can be classified as superloads with respect to Indiana permit regulations. Test trucks were positioned side by side at the middle, transversely, of the tested bridges. The structural models for the spring analogy method were loaded by two trucks positioned side by side at the same longitudinal location. Therefore, this benchmark study also provided a verification of the proposed method for multiple lane loading, which generally governs design or rating of bridges. The distance between the wheel lines of two adjacent trucks, which was not provided, was determined to be 48 in, based on the FEM loading figure provided in the study (Eom and Nowak, 2001). Also, spacing of the wheel lines was estimated to be 72 in for each truck.

The boundary conditions also affect the load distribution characteristics of a bridge. Although the bridges are designed as simply supported, traditionally, the authors investigated the effects of the various support conditions on the load distribution. Some older bridges exhibited more constraint at the supports for rotations and displacements due to corrosion of bearings (Eom and Nowak, 2001). In the numerical part of the study,

this effect was taken into account by assigning pins and/or rollers to the bottom of girders, and attaching longitudinal springs in the 3-D FEM.

Since the effects of support conditions were not considered in the derivation of Equation 5.1, the results obtained by spring analogy are compared with the results of the FEM which had support conditions similar to roller supports. The FEM which was used for comparison had rollers at one end and pins at the other end at the bottom of the girders. Although the pin at the bottom of a girder in the three dimensional model introduces a considerable amount of fixity at one end and affects the GDFs somewhat, it was regarded as a simple support by Eom and Nowak (2001). The maximum GDFs obtained from the “hinge (pin) and roller support” FEMs are compared with those obtained from the spring analogy method in Table 5.14. Results for some of the bridges were not presented by Eom and Nowak (2001). Therefore, GDFs for bridges 3, 8 and 9 were not available (N/A). Error of the spring analogy GDFs with respect to the FEA GDFs is also presented in Table 5.14. A negative error indicates that an unconservative result is obtained by the spring analogy method, whereas a positive error indicates a conservative result.

The average ratio of the spring analogy method GDFs to FEA GDFs was 1.06, with a corresponding standard deviation of 0.10. This indicates that, for the 14 bridges included in the database, the spring analogy method overestimates GDFs by 6% on average and approximates the FEA results within a 10% error margin. The average of ratios of AASHTO (2004) GDFs to FEA GDFs was 1.50 and the standard deviation of ratios was 0.11. Therefore, it can be concluded that spring analogy method matches the detailed FEA results reasonably well for this database. Furthermore, its accuracy is significantly higher compared to the AASHTO (2004) GDF equations. Scatter in the spring analogy results with respect to FEA results was comparable to that of AASHTO (2004) equation.

Bridges 15 and 17 had relatively higher errors, which were larger than 20%. However, the GDFs obtained by the spring analogy method were conservative for most cases. The exceptional error observed for these two bridges are estimated to be related to some factors not considered in the derivation of Equation 5.1. These factors may include effects of secondary members in addition to the longer span lengths of Bridges 15 (126 ft) and 17 (140 ft). These factors may have a higher influence on the load distribution characteristics of Bridges 15 and 17 compared to the other bridges. Ratios of the GDFs obtained by the spring analogy method to the GDFs from FEA are plotted with respect to span length of the analyzed bridges in Figure 5.12.

Figure 5.12 shows that the spring analogy method can over-predict the maximum GDFs for longer spans and may slightly under-predict for shorter spans. However, the error was limited to -6% for the short span bridges of this database. There is a noticeable trend between the GDF ratios and the span lengths. For future work, incorporating the span length into the spring analogy method as another parameter can overcome this observed error trend. Nevertheless, in order to keep the method as simple as possible, and considering that the level of error is within an acceptable limit, the spring analogy method was not modified.

Eom and Nowak (2001) also provided the GDF profile along the cross-section for some of the bridges. The GDF distribution for Bridge 14 is presented in Figure 5.13 for both the 3-D FEA and the spring analogy method. According to Figure 5.13, the spring analogy method captures the bending moment distribution across the section in addition to matching the maximum GDF of 0.37 for Bridge 14.

GDFs obtained experimentally by Eom and Nowak (2001) were not significantly different than the analytically calculated ones, and therefore were not compared directly with those calculated by spring analogy.

5.3.2. SOTELINO ET AL.'S (2004) STUDY

Sotelino et al. (2004) analyzed more than 50 steel bridges and 18 prestressed concrete bridges in Indiana with the objective of developing a simplified girder distribution factor (GDF) equation, which gives comparable results with the code equation (AASHTO, 1998). The analyzed bridges constituted a database representing typical range of properties for steel and PC girder bridges found in Indiana. A wide range of bridges with respect to span length was included in the database, as were multi-span, continuous bridges. The number of girders, their spacing and the skew angle were the other variables considered. Properties of the analyzed steel bridges are presented in Tables 5.15, 5.16 and 5.17. Table 5.15 has the properties of the bridges with no skew, Table 5.16 contains bridges with skew angles of 14 to 45 degrees and Table 5.17 includes the bridges which were outside of initial range considered for the simplified GDF equation proposed by Sotelino et al. (2004). Properties and results for the PC bridges will be presented after the discussion on the steel bridges.

FEA software ABAQUS was used for 3-D analysis of the bridges in this study. Rolled shape beams or plate girders were modeled with beam elements; the decks were modeled by shell elements. This modeling technique is called the eccentric beam model, since the eccentricity and the composite action between the deck and the girders are enforced by rigid link elements. Secondary members like parapets, cross-frames or diaphragms were not included in the FEMs. Pins were assigned to the bottoms of the girders at one end and rollers at the other end to simulate the support conditions. Roller supports were used at the interior supports of continuous bridges.

The investigated bridges were loaded by two HS-20 trucks placed side by side. The trucks were positioned close to the side of bridge. Typically, the first truck was placed at 2 ft from the parapet, which was assumed to be 1.5 ft wide at the base. Therefore, the center of first wheel line was 3.5 feet from the edge of the bridge. Wheel lines of the trucks were spaced 6 ft apart. There was a net distance of 4 ft between the

wheel lines of the two adjacent trucks. The GDFs obtained by three dimensional finite element analyses of Sotelino et al. (2004) and the spring analogy method are presented Tables 5.18, 5.19 and 5.20 for all steel bridges.

The average error of the spring analogy method with respect to the FEA was calculated as +1% for steel bridges based on the data presented in Tables 5.18, 5.19 and 5.20. The number of girders, girder spacing, and skew angle did not show any correlation to the error observed for this database. However, a relationship between the error and span length, similar to the one noted in Eom and Nowak's study (2001), was observed. Ratios of GDFs obtained with the spring analogy method to the GDFs obtained by FEA, with respect to span length, are presented in Figure 5.14. For continuous bridges, the length of the longest span was used.

Figure 5.14 shows that spring analogy method can predict the GDFs with $\pm 10\%$ accuracy for most bridges. The standard deviation of the GDF ratios was 0.08 for spring analogy results and was similar to the standard deviation of GDF ratios for the AASHTO (2004) equation, which was 0.09. However, the average of the GDF ratios was 1.01 for the spring analogy results, whereas average ratio was 1.10 for the AASHTO equation (2004). Therefore, the spring analogy method is believed to have an acceptable level of accuracy with respect to this comparison for steel bridges. For short span bridges, the possibility of under-predicting the GDF is higher.

Prestressed concrete bridges included in Sotelino et al.'s database (2004) represented typical PC bridges in Indiana in terms of skew, girder properties, spacing and span arrangements. The analysis of the PC bridges was conducted in a manner similar to the steel bridges. The investigated bridges had AASHTO Type II, Type III and Type IV girders. Properties of the 18 investigated bridges are presented in Table 5.21. Ratios of the GDFs obtained with spring analogy to the GDFs obtained by FEA are plotted with respect to span length (Figure 5.15). Also, the GDFs calculated by the spring analogy

method and FEA as well as the percentage error of the spring analogy method with respect to FEA are presented in Table 5.22.

The average of the GDF ratios was 1.00 and the average of the error values with respect to FEA results was 0% for the PC bridges. The standard deviation of the GDF ratios was 0.07 for the spring analogy method. On the other hand, the average of the GDF ratios was 1.15 and the standard deviation of the GDF ratios was 0.07 for the AASHTO (2004) equation. Although both methods had the same standard deviation, the AASHTO equation (2004) was overly conservative for most of the analyzed PC bridges. A notable trend between the GDF ratio and the span length, similar to that of steel bridges was observed for PC bridges. Regardless, analysis of the PC bridge database with the spring analogy method further validates its applicability to PC bridges in addition to steel bridges.

5.4. INCLUDING PARAPETS IN THE SPRING ANALOGY MODEL

In Chapter 4, it was shown that parapets can contribute to stiffness of a bridge and modify the live load response. However, a detailed literature survey revealed that a method for computing and incorporating the contribution of parapets is not currently available. For example, deflections or the GDFs might decrease due to the participation of parapets. Nevertheless, if the decrease in demand due to the effects of parapets cannot be calculated with a practical method, the contribution of parapets may not be taken into account. The spring analogy method can be used to compute the effects of parapets in a simple way.

The spring analogy model includes elements contributing to the flexural stiffness of a bridge by representing them with rotational springs. The parapets, which were found to be functioning like girders, can be modeled as girders. The springs representing the parapets should be positioned with respect to the transverse location of the parapets. The

stiffness of these springs is calculated in a similar fashion as the stiffness of exterior girders. Since the parapets are located above the deck as shown in Figure 4.4, their eccentricity with the neutral axis of the entire bridge creates a higher moment of inertia and stiffness.

First, the neutral axis location of the entire bridge cross-section with the girders and the parapets should be calculated. Then, the moment of inertia of the parapets with respect to the calculated neutral axis should be computed with Equation 5.3.

$$I_{NA} = I_{NAP} + Ad^2 \quad (\text{Eqn. 5.3})$$

where I_{NAP} and I_{NA} are the moments of inertia of the parapet with respect to its own neutral axis and the neutral axis of the entire bridge cross-section, respectively. A is the area of parapet and d is the distance between the neutral axis of the entire bridge cross-section and the neutral axis of the parapet. The calculated moment of inertia value (I_{NA}) should be divided by the modular ratio (ratio of the elastic modulus of steel to the elastic modulus of concrete) in order to obtain the appropriate transformed moment of inertia for the parapet for use in the calculations.

The moment of inertia, I_{NA} , calculated for the parapet should be divided by the moment of inertia of the interior girder, and this value should be used for the stiffness of a spring representing a parapet. For most of the bridges, the neutral axis location of the entire bridge cross-section and neutral axis location of an interior girder are not considerably different. Therefore, this eccentricity is negligible for girders typically. In order to simplify the calculations, the neutral axis location of the entire bridge can be taken as the neutral axis location of the interior girder. However, this may introduce some error, depending on the stiffness of girders and parapets.

When the parapets are included in the spring analogy model, calculation of the deflections requires a special approach due to the limitations of the model. The exterior

girder and the parapet, which are usually very close to each other usually, should have similar deflection values. However, the spring analogy model may predict different deflection values for the exterior girder and the parapet, since the proximity and overlapping of these members is not taken into account. In order to avoid different deflections, the exterior girder and the parapet should be constrained to move together. Although these two members may not have exactly the same deflections, assigning a constraint leads to more consistent deflection values for these members.

For the US-52 Bridge, two springs were added to the edges of the structural model shown in Figure 5.2. These springs were located at the location of the parapets in the transverse direction. The cross-sectional properties of the torsion bars connecting the parapets and the exterior girders were the same as the properties of torsion bars between the girders. The only difference was the shorter spacing. The same approach was also applied to the spring analogy model of the I-65 Bridge over US-30 in order to include the parapets. The stiffness of the torsion bar connecting the spring representing the parapet to the spring representing the exterior beam was found to be independent of the distance between the parapet and the exterior beam during development of the spring analogy method. For the torsion bar connected to the spring representing the parapet, having a stiffness value equal to that of the torsion bar between the interior beams gave the best agreement between the spring analogy method and FEA results.

The bending moments and deflections are presented in Figures 5.16 and 5.17 for the US-52 Bridge and in Figures 5.18 and 5.19 for the I-65 Bridge over US-30. The figures include results obtained by the spring analogy method and the FEA. The axle weights and truck locations were exactly the same as those presented in the sample calculations section without the parapets.

System analysis with parapets demonstrates that the proposed method is also effective in the calculation of GDFs and the deflection profile for bridges with longitudinal members which may have irregular stiffness properties. Incorporating the

parapets in the analysis does not affect accuracy. The difference between the GDFs computed using the spring analogy method and FEA were at most 6% and 14% relative to the FEA results for the US-52 and I-65 Bridges, respectively. Therefore, the spring analogy method can be used for computing the contribution of parapets.

5.5. ANALYSIS FOR SHOULDER LOADING POSITION

Positioning the trucks close to the edge of a bridge generates a critical load case for the exterior girder and the adjacent interior girder for most bridges. If there are two trucks positioned side by side, then the first interior girder may become the most critical member, as was the case for most of the bridges in Sotelino et al.'s (2004) study. Therefore, it might be necessary to position a truck on the shoulder of a bridge and analyze accordingly. Furthermore, the exterior girder may control the rating of a bridge almost all times. This is mostly because of the lever rule (AASHTO, 2004) used for the determination of GDFs for the exterior girders. The lever rule predicts higher GDFs compared to FEA results, and it can be considered as the bottle-neck in the rating of a bridge. Hence, improving the techniques to determine the exterior girder distribution factors can significantly increase the load rating capacity of bridges.

Effectiveness of the spring analogy method for the shoulder loading position was tested by analyzing the US-52, I-65 over Ridge Road, I-164, and I-65 over US-30 Bridges for the shoulder loading position. The HS-20 Truck was positioned on the shoulder, 2 ft from the parapet. Everything else, including the longitudinal truck position, was kept the same. US-52 and I-65 over Ridge Road bridges were analyzed with parapets in addition to the no parapet case, which was consistent with the analysis assumptions of AASHTO (2004). For the PC bridges only the no parapet case was considered.

Bending moment and deflection distributions are presented in Figures 5.20 to 5.23 for the US-52 Bridge. Analysis of the US-52 Bridge for the shoulder loading position

shows that the spring analogy method can under-predict the bending moments resisted by Beam 7, which was the most heavily loaded beam. The error relative to the FEA was 6% and 10% for the cases with and without the parapet, respectively. The bending moment resisted by the parapet, which resists more load compared to the most heavily loaded beam, is predicted accurately. For the deflections, the spring analogy method matched the general trend, but underestimated the maximum deflections for both cases.

Bending moments resisted by the girders and parapets of I-65 Bridge over Ridge Road are presented in Figures 5.24 and 5.25. Deflections of the members are presented in Figures 5.26 and 5.27 for shoulder loading position. For trucks positioned on the shoulder of the I-65 Bridge, the exterior girder was the most critical member. The spring analogy method predicted bending moments 4% and 14% lower than the FEA for the cases with and without parapet, respectively, for Girder 10. The bending moment resisted by the parapet was calculated accurately. Maximum deflection estimations were better than those for the US-52 Bridge. However, the spring analogy method also underestimated the maximum deflection for I-65 Bridge despite the fact that it matched the general trend reasonably well.

The distribution of the bending moments resisted by the girders of the I-164 Bridge due to the shoulder loading position is presented in Figure 5.28. Similar to the previously presented right lane loading position, the spring analogy method gives a lower bending moment compared to FEA for the exterior girder. The difference between the GDFs obtained by FEA and spring analogy was about 24% for Girder 7.

For the I-65 Bridge over US-30, the spring analogy method gave satisfactory results when the HS-20 Truck was positioned on the shoulder. As shown in Figure 5.29, the spring analogy method matched the general trend of bending moments resisted by the girders, and the demand on exterior girder was 10% higher than that predicted by the FEA.

5.6. OVERVIEW AND ADJUSTMENT FOR SPAN LENGTH

The spring analogy method was developed as an alternative to the traditional bridge analysis methods using GDF equations. It uses the girder spacing as the main variable and combines two simple analyses for obtaining bending moments and deflections corresponding to each girder or parapet. The main advantage of the proposed method was shown to be its simplicity, along with results that were comparable to those from a detailed 3-D finite element analyses.

Comparison of calculated GDFs for two typical steel and two typical prestressed concrete bridges demonstrated that the spring analogy method can predict the response of all girders in addition to predicting the maximum GDF with an acceptable accuracy for most of the slab-on-girder bridges. Also, the contribution of the parapets can be easily included in the analysis with this method. Use of the spring analogy model for permitting superload trucks can give accurate results, in addition to determining higher and more realistic load limits than a regular beam line analysis. GDFs calculated with the AASHTO (2004) equation, FEA and the spring analogy method for the interior girders of the investigated bridges are summarized in Table 5.23. Except for the I-164 Bridge, the spring analogy method approximated the FEA GDFs reasonably well and was conservative. Furthermore, the AASHTO (2004) equation resulted in approximately 30% higher GDFs than the FEA except in the case of the I-164 Bridge.

When a single truck load was positioned on the shoulders of the investigated bridges, the spring analogy method predicted lower bending moments and deflections compared to the FEA for some bridges, as summarized in Table 5.24. The magnitude of error for the maximum GDF was more than 25% for the I-164 Bridge. For the I-65 Bridge over Ridge Road, the spring analogy method gave a 4% lower GDF compared to FEA GDF. Spring analogy analysis of the other two bridges, US-52 and I-65 over US-30, gave comparable results with the FEA. Meanwhile, the GDFs calculated with the lever rule were overly conservative for both I-65 Bridges; the lever rule was unconservative for

the US-52 Bridge. Results presented in Tables 5.19 and 5.20 suggest that for short span bridges such as the I-164 Bridge, the spring analogy method has the potential to predict unrealistically lower GDFs, and that a factor of safety should be incorporated into the spring analogy method with respect to span length. Furthermore, an amplification factor should be applied to exterior girder distribution factors calculated by the spring analogy method.

The extensive databases of bridges analyzed by other researchers were used for further validation of the method. Application of the proposed method to the bridges analyzed by Eom and Nowak (2001) and Sotelino et al. (2004) shows the applicability of the spring analogy method to a broad range of bridges. The method is not limited to a certain number of girders or girder spacing. For most of the bridges analyzed by the aforementioned researchers, the spring analogy method predicts the maximum GDFs within 10% with respect to the 3-D finite element analyses.

Both databases showed that spring analogy can under-predict the GDFs for short span bridges. In order to keep the method as simple as possible, the length parameter was not included in the analysis. Including the span length as another parameter in the spring analogy method requires further development and is expected to be the subject of future research. However, the narrow band of error indicates that this potential problem can be handled by applying amplification factors to account for uncertainties in the method. The following amplification factors are recommended for the calculation of girder distribution factors and deflections:

- If the span length of a bridge is less than 60 ft, multiply the maximum GDF, maximum deflection, and largest relative deflection values by 1.3.
- If the span length is between 60 ft and 110 ft, multiply the maximum GDF, maximum deflection and largest relative deflection values by 1.1.

- If the span length is more than 110 ft, use the calculated GDFs and deflections without any modifications for the interior girders. However, for the exterior girders, the calculated GDF value should be multiplied by 1.1.

This approach brings in three different levels of scaling factors with respect to span length and reduces previously unconservative results obtained for short span bridges. The average error for bridges with span lengths less than 60 ft is 24% after scaling. For bridges with span lengths between 60 and 110 ft and span lengths longer than 110 ft, the average error is 11 and 10%, respectively. The error of the spring analogy method is conservative for all ranges. Girder distribution factor ratios, after scaling, versus span length for all of the analyzed bridges are presented in Figure 5.30. The average value is 1.14 and the standard deviation is 0.11 for the GDF ratios of all bridges after scaling. This indicates that the average error of the spring analogy method is 14% for all bridges included in the database. On the other hand, the average error is 18% for the AASHTO (2004) GDF equation. Furthermore, the standard deviation of the AASHTO (2004) equation is 0.17. Therefore, it can be concluded that the scaled spring analogy method is less conservative and its scatter margin is lower than the AASHTO (2004) GDF equation.

Note that the exterior GDF would give the highest distribution factor for most of the bridges and the margin of error for the spring analogy method was lower for the exterior girder than for the interior girder for the I-164 Bridge, which had the largest margin of error among all detailed analyses. Figure 5.30 shows that there is only one bridge for which the scaled spring analogy method gives unconservative results. The number of this bridge is 23440, and its properties were presented in Table 5.15. It did not have any skew or unusual girder arrangement in terms of spacing. Therefore, the reason for the unconservative result for Bridge 23440 analyzed by Sotelino et al. (2004) is not known. However, this result is believed to be an exception and might be related with the 3-D FEA of this structure. The spring analogy method results were more conservative for the bridges analyzed by Eom and Nowak (2001) compared to the bridges analyzed by

Sotelino et al. (2004). This is expected to be due to presence of secondary members in the FEMs of the bridges analyzed by Eom and Nowak (2001).

Table 5.25 shows the maximum GDFs for the analyzed bridges after application of the scaling factors. The spring analogy method results approximate the FEA results better than the AASHTO (2004) GDF equation or the lever rule. The error relative to the FEA results was ranging from 2 to 20% for the spring analogy method, whereas the error was between 6 and 56% for the GDFs calculated with respect to AASHTO (2004). Furthermore, the AASHTO (2004) approach was unconservative for the US-52 and the I-164 Bridges when these bridges were loaded by a single permit truck. Moreover, AASHTO (2004) does not provide any simple method to include the parapets in the analysis.

The I-164 Bridge was also analyzed with two trucks positioned side by side for investigating the variation in the error margin of the spring analogy method with respect to the number of trucks. It was found that the error was 16% when two trucks were positioned on the bridge compared to an error margin of 25% when there was only one truck on the bridge. This shows that the margin of error may not be the same for some bridges when more than one truck is present. Therefore, use of different scale factors for single truck and two trucks cases might be needed. However, validity of the scale factors for single truck loading was shown in Table 5.25. It is believed that the recommended scale factors might be conservative for two truck loading, but lack of data did not enable development of different scale factors.

Bending moments resisted by the parapets and contribution of the parapets to live load resistance were also calculated with reasonable accuracy using the spring analogy method. Therefore, the spring analogy method can be employed easily to compute the maximum demand imposed on the parapets and to calculate the contribution of parapets. This is especially important for bridges like the US-52 Bridge, for which the parapet may be the most heavily loaded member when a truck is positioned in the shoulder.

The spring analogy method can be developed further to give more accurate results for more bridges regardless of the span length and loading position. It has already been shown that the method can predict the bridge response accurately for most of the bridges and loading conditions. However, like all other simplified analysis methods, the spring analogy method may not be exact for all conditions. Given its low margin of error, the spring analogy method is believed to fill the gap between three-dimensional analysis and the beam line analysis with girder distribution factors. Accurate calculation of GDFs and the computation of differential girder deflections with the spring analogy method is expected to improve the evaluation of bridges for superloads. Meanwhile, in Chapter 2, differential girder deflection was shown to be the driving force for out of plane distortion of girder web and high secondary member stresses. Therefore, control of differential deflection in steel bridges can reduce damage due to superloads. For negative moment and shear GDFs, use of the AASHTO (2004) equations is recommended, since the spring analogy method was developed for positive moment only.

Table 5.1 Spring analogy distribution coefficients for the US-52 Bridge

Spring No.	Distribution Coefficient	
	Moment	Displacement
1	0.04	0.06
2	0.06	0.07
3	0.10	0.10
4	0.17	0.16
5	0.23	0.21
6	0.20	0.19
7	0.12	0.13
8	0.08	0.11

Table 5.2 Calculated beam moments for the US-52 Bridge

Beam No.	M (ft-k)	
	FEA	SA
1	14	47
2	62	68
3	125	109
4	200	190
5	258	262
6	229	224
7	154	140
8	93	97

Table 5.3 Calculated beam deflections for the US-52 Bridge

Beam No.	D (in)	
	FEA	SA
1	0.03	0.14
2	0.14	0.17
3	0.29	0.25
4	0.44	0.39
5	0.52	0.52
6	0.48	0.46
7	0.35	0.31
8	0.23	0.26

Table 5.4 Spring analogy distribution coefficients for the I-65 Bridge over Ridge Road

Spring No.	Distribution Coefficient	
	Moment	Displacement
1	0.01	0.01
2	0.01	0.02
3	0.04	0.05
4	0.11	0.12
5	0.33	0.30
6	0.33	0.30
7	0.11	0.12
8	0.04	0.05
9	0.01	0.02
10	0.01	0.01

Table 5.5 Calculated girder moments for the I-65 Bridge over Ridge Road

Girder No.	M (ft-k)	
	FEA	SA
1	-27	15
2	32	34
3	146	94
4	357	271
5	687	782
6	684	782
7	360	271
8	153	94
9	35	34
10	-33	15

Table 5.6 Calculated girder deflections for the I-65 Bridge over Ridge Road

Girder No.	D (in)	
	FEA	SA
1	-0.02	0.02
2	0.01	0.03
3	0.09	0.07
4	0.22	0.16
5	0.39	0.41
6	0.39	0.41
7	0.22	0.16
8	0.09	0.07
9	0.02	0.03
10	-0.03	0.02

Table 5.7 Spring analogy distribution coefficients for the I-164 Bridge

Spring No.	Distribution Coefficient	
	Moment	Displacement
1	0.05	0.07
2	0.07	0.09
3	0.13	0.13
4	0.24	0.22
5	0.26	0.24
6	0.15	0.15
7	0.10	0.12

Table 5.8 Calculated girder moments for the I-164 Bridge

Girder No.	M (ft-k)	
	FEA	SA
1	-4	19
2	10	26
3	53	49
4	120	90
5	127	97
6	62	56
7	6	37

Table 5.9 Calculated girder deflections for the I-164

Girder No.	D (in)	
	FEA	SA
1	0.00	-0.02
2	-0.01	-0.02
3	-0.03	-0.03
4	-0.07	-0.05
5	-0.08	-0.06
6	-0.04	-0.04
7	-0.01	-0.03

Table 5.10 Spring analogy distribution coefficients for the I-65 Bridge over US-30

Spring No.	Distribution Coefficient	
	Moment	Displacement
1	0.01	0.01
2	0.02	0.02
3	0.04	0.04
4	0.08	0.08
5	0.20	0.18
6	0.33	0.30
7	0.19	0.19
8	0.08	0.09
9	0.03	0.04
10	0.01	0.02
11	0.01	0.01

Table 5.11 Calculated girder moments for the I-65 Bridge over US-30

Girder No.	M (ft-k)	
	FEA	SA
1	28	13
2	37	24
3	66	54
4	133	128
5	285	305
6	457	506
7	262	282
8	123	119
9	60	50
10	36	22
11	30	12

Table 5.12 Calculated girder deflections for the I-65 Bridge over US-30

Girder No.	D (in)	
	FEA	SA
1	-0.01	-0.01
2	-0.02	-0.01
3	-0.03	-0.02
4	-0.06	-0.05
5	-0.10	-0.10
6	-0.13	-0.17
7	-0.10	-0.11
8	-0.06	-0.05
9	-0.03	-0.03
10	-0.02	-0.01
11	-0.01	-0.01

Table 5.13 Properties of the bridges analyzed by Eom and Nowak (2001)

Bridge No.	Span Length (ft)	Number of Girders	Girder Spacing (ft)	Construction Date	Skew Angle (Degrees)
1	45	10	4.3	1935	30
2	55	11	4.7	1932	0
3	32	12	4.5	1922	10
4	45	9	4.8	1939	20
5	38	10	4.7	1929	0
6	67	7	4.7	1933	0
7	62	6	6.2	1965	11
8	75	9	4.0	1928	0
9	98	5	9.3	1970	0
10	70	11	4.5	1936	0
11	87	10	4.5	1932	0
12	50	9	5.2	1947	0
13	35	10	4.6	1948	15
14	55	8	5.9	1953	10
15	126	7	7.3	1972	0
16	35	9	5.2	1949	0
17	140	6	9.4	1986	0

Table 5.14 Comparison of GDFs obtained by three dimensional FEA (Eom and Nowak, 2001) and spring analogy

Bridge No.	Skew Angle (Degrees)	Maximum GDF		Error (%)
		3D FEA by Eom and Nowak (2001)	Spring Analogy	
1	30	0.30	0.29	-3
2	0	0.31	0.31	0
3	10	N/A	0.29	N/A
4	20	0.31	0.32	3
5	0	0.28	0.30	7
6	0	0.31	0.35	13
7	11	0.43	0.42	-2
8	0	N/A	0.28	N/A
9	0	N/A	0.60	N/A
10	0	0.26	0.29	12
11	0	0.27	0.30	11
12	0	0.33	0.34	3
13	15	0.32	0.30	-6
14	10	0.37	0.37	0
15	0	0.37	0.46	24
16	0	0.36	0.34	-6
17	0	0.43	0.54	26
			Average	6

Table 5.15 Properties of steel bridges with no skew (Sotelino et al., 2004)

Bridge No.	Number of Spans	Span Lengths (ft)	Number of Girders	Girder Spacing (ft)	Skew Angle (Degrees)
5870	1	48	8	5.4	0
17860	3	42/56/42	6	6.5	0
27340	1	59	8	5.5	0
33010	4	50/60/60/50	6	5.3	0
210	4	36/65/65/36	11	5.3	0
35090	2	65/65	6	5.3	0
960	2	67/67	8	5.8	0
12350	3	60/75/60	8	5.4	0
25000	3	62/78/62	7	5.3	0
23490	3	55/85/55	8	6.2	0
21260	4	77/93/93/77	7	5.2	0
75030	2	94/94	8	7.3	0
38160	2	98/98	6	5.3	0
13057	3	82/103/82	5	8.5	0
14320	3	83/104/83	5	6.0	0
6330	1	110	5	9.0	0
34960	2	112/112	7	5.7	0
23408	1	122	9	6.6	0

Table 5.16 Properties of steel bridges with skew (Sotelino et al., 2004)

Bridge No.	Number of Spans	Span Lengths (ft)	Number of Girders	Girder Spacing (ft)	Skew Angle (Degrees)
33162	3	37/55/46	7	9.2	4
35890	2	88/88	5	8.0	14
29660	1	60	11	4.3	15
11658	2	120/120	6	7.7	15
75310	3	35/61/35	5	6.8	16
11275	4	39/57/57/39	6	5.8	17
37270	2	110/110	6	6.3	17
18314	3	47/65/54	6	7.8	18
33043	4	42/93/70/48	5	6.1	18
40120	2	121	9	7.3	20
32836	2	102/102	8	8.5	22
34330	3	48/97/34	6	6.7	27
17030	1	46	11	4.3	30
16080	3	54/65/54	7	7.4	30
27350	1	72	7	6.1	30
35760	2	98/98	6	6.3	31
18370	1	85	5	10.0	36
14620	4	45/56/56/45	9	4.2	42
17980	4	55/106/106/55	7	8.0	44
13330	1	60	8	5.8	45
1620	3	60/78/60	7	7.0	45
43860	2	85/85	5	6.5	45
70430	3	68/97/68	6	8.0	45

Table 5.17 Properties of steel bridges out of typical range (Sotelino et al., 2004)

Bridge No.	Number of Spans	Span Lengths (ft)	Number of Girders	Girder Spacing (ft)	Skew Angle (Degrees)
22560	3	30	7	5.3	40
28930	3	30	10	4.5	30
22680	1	140	7	8.8	16
40420	2	140	4	8.7	24
10212	1	144.5	8	7.0	26
22625	4	150	4	12.2	0
50460	2	150	5	8.3	0
26360	3	170	5	10.5	0
76160	3	26.5/122/26.5	7	7.5	2
5503	3	194	7	7.3	11

Table 5.18 GDFs for steel bridges with no skew (analyzed by Sotelino et al., 2004)

Bridge No.	Maximum GDF		Error (%)
	3D FEA by Sotelino et al. (2004)	Spring Analogy	
5870	0.44	0.41	-8
17860	0.53	0.50	-7
27340	0.45	0.44	-3
33010	0.36	0.41	13
210	0.44	0.40	-10
35090	0.46	0.41	-10
960	0.45	0.43	-6
12350	0.45	0.42	-8
25000	0.44	0.41	-8
23490	0.48	0.41	-15
21260	0.43	0.41	-4
75030	0.54	0.56	2
38160	0.41	0.41	0
13057	0.60	0.62	4
14320	0.48	0.47	-3
6330	0.65	0.65	0
34960	0.42	0.43	2
23408	0.47	0.50	6
		Average	-3

Table 5.19 GDFs for steel bridges with skew (analyzed by Sotelino et al., 2004)

Bridge No.	Skew Angle (Degrees)	Maximum GDF		Error (%)
		3D FEA by Sotelino et al. (2004)	Spring Analogy	
33162	4	0.79	0.66	-17
35890	14	0.59	0.60	2
29660	15	0.36	0.33	-6
11658	15	0.56	0.58	3
75310	16	0.56	0.54	-4
11275	17	0.46	0.44	-4
37270	17	0.51	0.47	-7
18314	18	0.61	0.58	-5
33043	18	0.50	0.48	-4
40120	20	0.51	0.56	9
32836	22	0.58	0.62	7
34330	27	0.49	0.50	1
17030	30	0.34	0.33	-1
16080	30	0.55	0.56	3
27350	30	0.46	0.46	0
35760	31	0.46	0.47	2
18370	36	0.66	0.70	6
14620	42	0.31	0.34	11
17980	44	0.53	0.59	11
13330	45	0.42	0.44	5
1620	45	0.43	0.54	24
43860	45	0.48	0.50	4
70430	45	0.55	0.59	7
			Average	2

Table 5.20 GDFs for steel bridges out of typical range (analyzed by Sotelino et al., 2004)

Bridge No.	Skew Angle (Degrees)	Maximum GDF		Error (%)
		3D FEA by Sotelino et al. (2004)	Spring Analogy	
22560	40	0.40	0.41	3
28930	30	0.42	0.34	-19
22680	16	0.61	0.63	4
40420	24	0.61	0.65	7
10212	26	0.47	0.53	12
22625	0	0.71	0.78	9
50460	0	0.56	0.61	9
26360	0	0.69	0.73	7
76160	2	0.55	0.57	3
5503	11	0.51	0.55	9
			Average	4

Table 5.21 Properties of prestressed concrete bridges (Sotelino et al., 2004)

Bridge No.	Number of Spans	Span Lengths (ft)	Number of Girders	Girder Spacing (ft)	Skew Angle (Degrees)
18317	1	70	7	6.5	0
530	1	54	6	7.3	0
270	3	55/65/55	8	6.0	0
31100	3	150	5	7.5	35
1690	1	78	6	8.5	35
22850	1	79	7	7.0	15
1342	1	87	7	6.3	0
2230	3	74/74/74	6	6.0	45
12720	1	65	7	7.0	15
610	1	49	7	7.0	30
33020	1	48	7	7.0	0
18253	1	55	7	6.5	15
19670	3	43/60/43	6	6.3	0
24380	1	67	5	7.4	0
10400	3	71/71/71	6	6.5	16
11110	3	76/76/76	7	7.0	20
23700	1	82	7	7.0	0
590	1	115	11	6.8	20

Table 5.22 GDFs for prestressed concrete bridges (analyzed by Sotelino et al., 2004)

Bridge No.	Skew Angle (Degrees)	Maximum GDF		Error (%)
		3D FEA by Sotelino et al. (2004)	Spring Analogy	
18317	0	0.44	0.50	12
530	0	0.59	0.55	-6
270	0	0.50	0.46	-9
31100	35	0.54	0.58	6
1690	35	0.61	0.62	2
22850	15	0.54	0.54	0
1342	0	0.49	0.49	0
2230	45	0.41	0.46	12
12720	15	0.56	0.54	-4
610	30	0.54	0.54	-1
33020	0	0.57	0.54	-6
18253	15	0.47	0.50	5
19670	0	0.53	0.49	-9
24380	0	0.58	0.57	-2
10400	16	0.51	0.50	-4
11110	20	0.55	0.54	-3
23700	0	0.56	0.54	-4
590	20	0.47	0.52	12
			Average	0

Table 5.23 GDFs for the interior girders of the investigated bridges for no parapet cases

Bridge	GDF		
	AASHTO Eqn.	FEA	Spring Analogy
US-52	0.29	0.23	0.23
I-65 over Ridge Road	0.44	0.29	0.33
I-164	0.38	0.34	0.24
I-65 over US-30	0.41	0.30	0.33

Table 5.24 GDFs for the exterior girders of the investigated bridges for no parapet cases

Bridge	GDF		
	Lever Rule	FEA	Spring Analogy
US-52	0.30	0.32	0.32
I-65 over Ridge Road	0.70	0.52	0.50
I-164	0.39	0.41	0.31
I-65 over US-30	0.67	0.43	0.47

Table 5.25 Maximum GDFs for the investigated bridges for no parapet cases

Bridge	Maximum GDF		
	AASHTO Lever Rule or Eqn.	FEA	Scaled Spring Analogy
US-52	0.30	0.32	0.35
I-65 over Ridge Road	0.70	0.52	0.55
I-164	0.39	0.41	0.40
I-65 over US-30	0.67	0.43	0.52

Figure 5.1 Simple structural model for calculation of GDFs

Figure 5.2 Structural model for determining distribution factors

Figure 5.3 Simple beam model for the US-52 Bridge in SAP2000

Figure 5.4 Distribution of bending moments for the US-52 Bridge due to the HS-20 Truck positioned at the center of the right lane

Figure 5.5 Distribution of beam displacements for the US-52 Bridge due to the HS-20 Truck positioned at the center of the right lane

Figure 5.6 Distribution of bending moments for the I-65 Bridge over Ridge Road due to the HS-20 Truck positioned at the center of the bridge

Figure 5.7 Distribution of girder displacements for the I-65 Bridge over Ridge Road due to the HS-20 Truck positioned at the center of the bridge

Figure 5.8 Distribution of bending moments for the I-164 Bridge due to the HS-20 Truck positioned at the center of the right lane

Figure 5.9 Distribution of girder displacements for the I-164 Bridge due to the HS-20 Truck positioned at the center of the right lane

Figure 5.10 Distribution of bending moments for the I-65 Bridge over US-30 due to the HS-20 Truck positioned at the center of the bridge

Figure 5.11 Distribution of girder displacements for the I-65 Bridge over US-30 due to the HS-20 Truck positioned at the center of the bridge

Figure 5.12 Comparison of GDFs with respect to span lengths of bridges analyzed by Eom and Nowak (2001)

Figure 5.13 Comparison of GDFs obtained by 3D FEA and spring analogy for Bridge 14 of Eom and Nowak (2001)

Figure 5.14 Comparison of GDFs with respect to span lengths of steel bridges analyzed by Sotelino et al. (2004)

Figure 5.15 Comparison of GDFs with respect to span lengths of PC bridges analyzed by Sotelino et al. (2004)

Figure 5.16 Bending moment distribution for the US-52 Bridge due to the HS-20 Truck positioned at the center of the right lane

Figure 5.17 Deflection distribution for the US-52 Bridge due to the HS-20 Truck positioned at the center of the right lane

Figure 5.18 Bending moment distribution for the I-65 Bridge over Ridge Road due to the HS-20 Truck positioned at the center of the bridge

Figure 5.19 Deflection distribution for the I-65 Bridge over Ridge Road due to the HS-20 Truck positioned at the center of the bridge

Figure 5.20 Bending moment distribution for the US-52 Bridge shoulder loading position when the parapets are not included in the analysis

Figure 5.21 Bending moment distribution for the US-52 Bridge shoulder loading position when the parapets are included in the analysis

Figure 5.22 Deflection distribution for the US-52 Bridge shoulder loading position when the parapets are not included in the analysis

Figure 5.23 Deflection distribution for the US-52 Bridge shoulder loading position when the parapets are included in the analysis

Figure 5.24 Bending moment distribution for the I-65 Bridge over Ridge Road shoulder loading position when the parapets are not included in the analysis

Figure 5.25 Bending moment distribution for the I-65 Bridge over Ridge Road shoulder loading position when the parapets are included in the analysis

Figure 5.26 Deflection distribution for the I-65 Bridge over Ridge Road shoulder loading position when the parapets are not included in the analysis

Figure 5.27 Deflection distribution for the I-65 Bridge over Ridge Road shoulder loading position when the parapets are included in the analysis

Figure 5.28 Bending moment distribution for the I-164 Bridge shoulder loading position

Figure 5.29 Bending moment distribution for the I-65 over US-30 Bridge shoulder loading position

Figure 5.30 Ratios of scaled spring analogy GDFs to FEA GDFs

CHAPTER 6. SUMMARY AND CONCLUSIONS

Effects of superloads on bridge superstructures were studied. Two steel and two prestressed concrete (PC) bridges were investigated to evaluate the effects of superloads on typical slab-on-girder bridges. These representative bridges were selected from an extensive database of bridges in Indiana. In the evaluated bridges, rolled shapes, plate girders, AASHTO type girders and bulb-tee girders were used to support a reinforced concrete slab. There were both continuous and simple span bridges in the inventory. Furthermore, two of the bridges had integral end abutments. Therefore, the representative bridges covered a range of bridge parameters in terms of material, girder and span properties. None of the investigated bridges, however, were skewed.

The investigated bridges were analyzed using the SAP2000 and ANSYS programs. Nonlinear analyses involving geometric and material nonlinearities related with structural steel components were conducted in addition to detailed linear analyses and beam line analyses. The capacity of each bridge was calculated with respect to the provisions of AASHTO LRFR (2003). The analytical part of the study also revealed possible problem locations in the bridges should they be subjected to superloads.

Furthermore, one steel girder bridge and one PC girder bridge were instrumented. The instrumentation plan was based on the results of the preliminary finite element analysis. A strain transducer was developed to measure strains on concrete surfaces. The comprehensive instrumentation program and the long term monitoring of the bridges provided valuable information about the behavior and live load response of slab-on-girder bridges. The finite element models were then refined with respect to the field

measurements. Also, fatigue resistances of the details found on the investigated bridges were evaluated. An S-N relationship was developed for a welded stiffener top flange connection. Moreover, measurements were used in the fatigue life calculations of a welded stiffener plate top flange connection.

Effects of secondary members on the live-load resistance were also checked using both analytical tools and actual bridge response. The contribution of the parapets to live load resistance was quantified, as was the live load demand on the parapets.

A simple method was developed for more accurate evaluation of bridges, as compared to the current beam line approach. Results obtained with the proposed simple analysis method were compared with the detailed analysis results.

The findings and conclusions from the evaluation of investigated bridges and the tools developed to assess live-load response of bridges are summarized below.

6.1. SUMMARY OF FINDINGS

Analysis and capacity evaluation of all four bridges indicated that, except for the 824-kip Maximum Truck included in the database, all superload trucks included in the study can safely traverse the investigated bridges provided that position and speed of the permit truck is controlled and presence of any other traffic on the bridge is avoided. The Maximum Truck or heavier superload truck configurations can cause significant distress in some bridges because of the notably high demand.

Both of the investigated PC bridges were found to have adequate capacity to accommodate the Maximum Truck or heavier superloads. On the other hand, the Maximum Truck generated a demand slightly higher than the capacity in the secondary

members for both of the investigated steel bridges. The rating factor for the Maximum Truck was 0.9 for both steel bridges.

Serviceability evaluation of the investigated prestressed concrete bridges revealed that the Maximum Truck may create a demand slightly higher than the optional serviceability provisions of the AASHTO LRFR (2003). Furthermore, the 500-kip Group D Superload Truck and the Maximum Truck can cause cracking in the girders of both prestressed concrete bridges, which respond elastically to HL-93 Loading. However, the steel bridges satisfied all optional serviceability criteria recommended by AASHTO (2003). For the prestressed concrete bridges, the serviceability criterion was based on the prestressing tendon stresses, and the objective of this limit was to avoid excessive cracking due to permit loads. If the optional serviceability criterion was enforced, a 740-kip Group D type superload truck would be the heaviest truck which could obtain a permit for the analyzed PC bridges.

Although the serviceability criteria to limit excessive deflections of steel bridges were met, it was found that some of the secondary members or beam webs in bridges with diaphragm-beam connections might be damaged. Cross-frames in plate girder bridges and diaphragms and their connections in rolled-shape bridges were subjected to the highest stresses in the bridge. Failure of a diaphragm, development of fatigue cracks in the beam web, damage to cross-frame angles, and failure of the welded stiffener plate top flange connections were found to be possible damage mechanisms for steel bridges. However, no damage or change in the response of the main members is expected, since these secondary members are not typically considered in the live load resistance.

The strain transducer developed for monitoring surface strains on concrete members proved that the custom built sensor can measure strains with an acceptable accuracy and provided consistent results. Furthermore, its rugged yet simple structure was shown to be durable enough for field applications.

Instrumentation of the two span PC bridge revealed several notable observations about the actual behavior. First of all, it was found that the PC bridge, although made continuous for live load, was not responding to live load as a fully continuous structure. The actual behavior was closer to that of two independent spans, rather than continuous behaviour. The integral abutment provided 20-25% fixity on a scale which has simple (0% fixity) and fixed (100% fixed) supports as the upper and lower bounds. The general behavior was observed to be consistent with the aforementioned statements with respect to strain and deflection measurements as well as the free vibration characteristics. The diaphragms of the instrumented bridge did not participate notably in the live load resistance. Therefore, the load distribution was not affected by the presence of diaphragms. Consequently, the reinforced concrete diaphragms are not expected to experience any appreciable damage during superload passages as predicted for the steel bridges.

Load tests of the instrumented steel and PC bridges generated strains comparable to those for Group B and C Superload Trucks. Furthermore, the recorded events generated similar strains. This implies that the response of the instrumented bridges on I-65 for a Group B or C Superload Truck may not be more significant than that for the highest magnitude events recorded during long-term monitoring.

Analysis of the investigated bridges showed that the parapets of steel and PC bridges and cross-frames of the steel bridges contribute to live-load resistance. Including the parapets in the analysis can result in a 30% decrease in the girder distribution factor for flexure compared to the case without the parapets. The contribution of the parapet to load resistance was more significant when the trucks were positioned close to the parapet. The decrease in the distribution factor for the exterior girder was also more than those for the interior girders. The demand generated on the parapets for the superload trucks was lower than the capacity of the parapets and their connection with the deck. Also, discontinuities in the parapets along the bridge length were found to be stress raisers. In the positive moment region, parapet gaps amplified the bending moment resisted by the

exterior girder up to 1.7 times compared to the continuous parapet case, and up to 1.2 times compared to the no parapet case. In the negative moment region, the parapet gaps increased the tensile stresses calculated in the deck more than 10 times compared to the continuous parapet case. This suggests that parapet gaps will increase the likelihood of deck cracking when heavy loads are positioned near the parapet in the negative moment region of the bridge.

The spring analogy method was developed for computing the three dimensional live-load response of slab-on-girder bridges. The results were found to approximate the detailed finite element analysis results within a 10% error margin for the bridges included in the existing databases. Furthermore, the spring analogy method mimicked the moment and deflection distributions obtained from the detailed analysis of the investigated steel and prestressed concrete bridges.

Deflections of the girders and the relative deflection of the girders can be used to evaluate the stresses in the web gap using an approach similar to that proposed by Jajich and Schultz (2003). Although accuracy of the equation developed by Jajich and Schultz (2003) to compute the stresses in the web gap due to interaction with the secondary members was limited, the equation can still be used to approximate the distortional stresses. Furthermore, use of the relative girder deflections calculated with the spring analogy method to compute the out-of-plane stresses provides a practical check to control the distortional stresses in the web.

Fatigue analysis of the welded stiffener plate to top flange connection of a case study bridge indicated that this detail can be classified as Category E for typical weld and stiffener plate geometries. Fracture of the weld may lead to increased stresses in the web gap region and could possibly lead to out-of-plane cracking of the web.

Based upon work reported in the literature, overload cycles due to heavy permit trucks are found to increase the fatigue life if the overload cycles occur only infrequently.

Having more than 1,000 cycles between the overloads was recommended by Abtahi et al. (1976) in order to have beneficial overload effects.

6.2. CONCLUSIONS AND RECOMMENDATIONS

Based on the findings of this study, the following conclusions and recommendations are provided:

- The effects of Superload Groups A (201-kip), B (248-kip) and C (366-kip) on typical slab-on-girder bridges are comparable to large magnitude events recorded during long-term monitoring. Based upon the frequency of occurrence of these loads, they were not found to be detrimental to the bridges evaluated in the study.
- The 824-kip Maximum Truck and the 500-kip Group D Superload Truck generate moments and shear forces significantly higher than those of the other superloads included in the database. Closely spaced axles of Group D Superload Trucks result in a demand envelope similar to that for uniform loading. This generates a demand envelope different than that of a typical truck. Therefore, negative moments generated by these trucks cannot be obtained as multiples of the HL-93 loading. Superload trucks that spread the load to widely spaced axle groups generate a lower demand since all axles are generally not present on the bridge at the same time. This is in contrast to the closely spaced axles of a Group D Superload truck.
- The investigated steel and PC bridges have adequate capacity to accommodate all superload trucks included in the database except for the 824-kip Maximum Truck, which resulted in a demand exceeding either the strength or serviceability limit state. Superload trucks with an axle configuration similar to that of the Maximum Truck may exceed the live-load capacity of typical slab-on-girder bridges.
- The optional serviceability limit state recommended by AASHTO LRFR (2003) should be enforced to improve long term performance of bridges. Overloads

exceeding the serviceability limits can lead to extensive cracking of the PC girders.

- Controlled passage of superload trucks requires accurate analysis to utilize the actual capacity of a bridge effectively. The AASHTO (2004) equations to compute the GDFs were found to be conservative. Use of a detailed analysis can improve the rating of a bridge, since it provides a more accurate load distribution.
- The contribution of the parapets to live-load resistance should be included in the analysis to obtain a realistic demand estimate.
- The proposed spring analogy method can be used to predict the three dimensional live-load response of a bridge more accurately than the use of the GDF equations (2004). Furthermore, the spring analogy method is considerably simpler than a detailed finite element analysis in terms of the preprocessing and postprocessing effort required. Also, the parapets can be included easily in the spring analogy model to fully represent the bridge cross-section.
- Secondary members, such as cross-frames and diaphragms, as well as the web gap in steel bridges, are possible damage locations. Limited yielding was observed in the web gap of an analyzed steel bridge with a diaphragm-web connection when subjected to a heavy superload, such as the 824-kip Maximum Truck.
- Monitoring of the instrumented prestressed concrete bridge indicated that the diaphragms did not experience very considerable shear forces due to traffic live loading. Therefore, no damage due to superloads is expected in the diaphragms.
- Overload cycles from permit trucks are not expected to have a negative effect on the fatigue life of slab-on-girder bridges. Furthermore, load interaction effects can be beneficial for steel components. Prestressed concrete girders were insensitive to load interaction effects, according to available literature.
- The behavior of the instrumented prestressed concrete bridge made continuous for live load was found to be close to that of two independent spans for the service load level. Therefore, for serviceability limit states, as a lower bound, continuous prestressed concrete bridges can be analyzed as independent spans.

- Integral abutments resulted in considerable end fixity for both of the instrumented bridges. This beneficial effect can be utilized if its load response is verified through field measurements.

6.3. IMPLEMENTATION RECOMMENDATIONS

A detailed study of slab-on-girder and slab-on-beam type bridges was undertaken to understand the influence that superloads have on these common types of bridge structures. The study included both analytical and experimental components to evaluate a number of proto-typical bridge structures. Based upon the findings in this study, the following implementation recommendations are provided.

First, care should be taken when evaluating superloads with a gross vehicle weight greater than 500-kips. Group A through C superload truck configurations (200-kips to 324-kips) were not found to cause serious distress in the studied bridges, but larger superload groups may be problematic.

Second, the types of bridge structure distress that were identified through the superload structural analysis include tension zone cracking in prestressed girders and damage to secondary members (cross frames and diaphragms) or in the web gap of diaphragm-to-web connection regions of steel members. These critical areas should be evaluated carefully if a number of large superloads have crossed a bridge structure.

Third, the structure should be assessed carefully for all superloads with a gross vehicle weight exceeding 500-kips. Experimental monitoring and/or detailed structural analysis were found to provide an accurate depiction of the structural response of a bridge to a superload truck. Since an increased number of superload trucks are being used throughout the state, it is important to limit the superload trucks to configurations that do not cause distress. Gross vehicle weights greater than 500-kips can be successfully used if the axle width and spacing are adjusted to minimize the impact of the loads on the structure.

6.4. FUTURE RESEARCH NEEDS

The current study revealed possible damage mechanisms and weak points of typical slab-on-girder bridges when traversed by various types (classes) of superload trucks. The study included an experimental and analytical evaluation of typical steel and concrete slab-on-girder bridges to evaluate the behavior and capacity analysis of these common bridge systems when subjected to various superload patterns. However, other types of bridge systems, such as box-girder bridges, trusses and culverts, were not included in the scope of this investigation. These structures should also be studied to evaluate the bridge response under the action of a various classes of superload trucks to determine possible problem locations in these bridges.

End fixity in integral abutment bridges can have a considerable effect on the moment demand. Verification of the integral abutment fixity is needed in order to utilize this reserve strength. Also, the shear resistance of plate girders connected to integral abutments should be investigated. The behavior of an end panel is expected to be different when it is embedded in a concrete abutment. However, the current approach does not address this issue. Rating of the investigated plate girder bridge was controlled by the shear strength of the end panel, which could actually be higher than the calculated value.

The magnitude of the live-load for which continuous PC bridges behave as fully continuous should be investigated further. Evaluation of these bridges as simple spans might be considered based on the observations of this study. However, in-field evaluation of more continuous prestressed concrete bridges is needed to draw a general conclusion about the actual behavior these bridges.

The spring analogy method should be verified further, and the span length might be incorporated as another parameter. A simple spreadsheet program to obtain the

response of slab-on-girder bridges can be developed and utilized by bridge engineers as a practical tool to obtain results comparable to those from a detailed three dimensional analysis.

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APPENDICES

APPENDIX A

Some as-built drawings of the investigated steel bridges are presented in this section.

Figure A.1 Framing plan of the first steel span of the US-52 Bridge

Figure A.2 Properties of the plate girder used for the I-65 Bridge over Ridge Road (Wood, 2004)

APPENDIX B

Some as-built drawings of the investigated prestressed concrete bridges are presented in this section.

Figure B.1 Prestressed concrete beam details and prestressing pattern for the first and third span beams of the I-164 Bridge

Figure B.2 Prestressed concrete beam details of the second span of the I-164 Bridge

Figure B.3 Prestressing pattern for the second span beams of the I-164 Bridge

Figure B.4 Stirrup detail for the beams of the I-164 Bridge

Figure B.5 Beam details for the first span (Span A) of the I-65 Bridge over US-30

Figure B.6 Prestressing pattern for the girders of the first span (Span A) of the I-65 Bridge over US-30

Figure B.7 Beam details for the second span (Span B) of the I-65 Bridge over US-30

Figure B.8 Prestressing pattern for the girders of the second span (Span B) of the I-65 Bridge over US-30