University Transportation Research Center - Region 2

Final Report

Effects of Overweight Vehicles on NYSDOT Infrastructure

Performing Organization: The City College of New York (CUNY)

September 2015

Sponsor:
New York State Department of Transportation

University Transportation Research Center - Region 2

The Region 2 University Transportation Research Center (UTRC) is one of ten original University Transportation Centers established in 1987 by the U.S. Congress. These Centers were established with the recognition that transportation plays a key role in the nation's economy and the quality of life of its citizens. University faculty members provide a critical link in resolving our national and regional transportation problems while training the professionals who address our transportation systems and their customers on a daily basis.

The UTRC was established in order to support research, education and the transfer of technology in the field of transportation. The theme of the Center is "Planning and Managing Regional Transportation Systems in a Changing World." Presently, under the direction of Dr. Camille Kamga, the UTRC represents USDOT Region II, including New York, New Jersey, Puerto Rico and the U.S. Virgin Islands. Functioning as a consortium of twelve major Universities throughout the region, UTRC is located at the CUNY Institute for Transportation Systems at The City College of New York, the lead institution of the consortium. The Center, through its consortium, an Agency-Industry Council and its Director and Staff, supports research, education, and technology transfer under its theme. UTRC's three main goals are:

Research

The research program objectives are (1) to develop a theme based transportation research program that is responsive to the needs of regional transportation organizations and stakeholders, and (2) to conduct that program in cooperation with the partners. The program includes both studies that are identified with research partners of projects targeted to the theme, and targeted, short-term projects. The program develops competitive proposals, which are evaluated to insure the mostresponsive UTRC team conducts the work. The research program is responsive to the UTRC theme: "Planning and Managing Regional Transportation Systems in a Changing World." The complex transportation system of transit and infrastructure, and the rapidly changing environment impacts the nation's largest city and metropolitan area. The New York/New Jersey Metropolitan has over 19 million people, 600,000 businesses and 9 million workers. The Region's intermodal and multimodal systems must serve all customers and stakeholders within the region and globally.Under the current grant, the new research projects and the ongoing research projects concentrate the program efforts on the categories of Transportation Systems Performance and Information Infrastructure to provide needed services to the New Jersey Department of Transportation, New York City Department of Transportation, New York Metropolitan Transportation Council , New York State Department of Transportation, and the New York State Energy and Research Development Authorityand others, all while enhancing the center's theme.

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Principal Investigator(s):

Dr. Michel Ghosn
Department of Civil Engineering
The City College of New York
The City University of New York
160 Convent Ave.
New York, NY 10031
Email: ghosn@ce.ccny.cuny.edu

Dr. Neville Parker
Department of Civil Engineering
The City College of New York
The City University of New York
160 Convent Ave.
New York, NY 10031
Email: parker@utrc2.org

Co author(s):
-Graziano Fiorillo
-Volodymyr Gayovyy
-Tenzin Getso
-Sallem Ahmed
-The City College of New York

Performing Organization:
The City College of New York

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To request a hard copy of our final reports, please send us an email at utrc@utrc2.org

Mailing Address:

University Transportation Reserch Center
The City College of New York
Marshak Hall, Suite 910
160 Convent Avenue
New York, NY 10031
Tel: 212-650-8051
Fax: 212-650-8374
Web: www.utrc2.org

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Bahman Moghimi*: Research Assistant;*
Ph.D. Student, Transportation Program

Wei Hao*: Research Fellow*

Andriy Blagay: *Graphic Intern*

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16. Abstract

This report develops a methodology for estimating the effects of different categories of overweight trucks on NYSDOT pavements and bridges. A data mining algorithm is used to categorize truck data collected at several Weigh-In-Motion stations around the state of New York based on the trucks' adherence to the state's legal weight limits. The data indicate that about 11% of the trucks traveling on New York highways may be carrying divisible load permits, 1% may be carrying special hauling permits, while about 6% may be illegally overweight

The analysis shows that these overweight trucks are increasing the risk to failure of bridges by causing stresses above those specified in design specifications and by reducing bridge service (fatigue) lives through repetitive overloading. A monetization of the safety margin utilization due to the combined overstress and cyclic fatigue shows that trucks carrying divisible load permits may be responsible for $50M per year in NYS bridge infrastructure cost, trucks with special hauling permits may be responsible for $2M/yr in additional cost while illegally overweight trucks may be responsible for $43M per year for a total of $95M/yr.

The cost allocation study performed on the NYS pavement network shows that the cost to NYS pavements due to overweight trucks is about $145M/yr divided into $78M/yr for divisible load permits, $7M/yr for special hauling permits and $60M/yr for illegally overweight trucks.

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Effects of Overweight Vehicles on NYSDOT Infrastructure

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Effects of Overweight Vehicles on NYSDOT Infrastructure

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Effects of Overweight Vehicles on NYSDOT's Infrastructure

Executive Summary

The objective of this project is to develop models for assessing the cost of overweight vehicles to New York State's highway pavements and bridges. The models are applied to assess the cost of overweight trucks that have been issued permits by the New York State Department of Transportation (NYSDOT) to carry both divisible and non-divisible loads, as well as illegally overweight non-permit trucks that violate the legal limits and trucks that violate their permit limits.

In a first phase, this study develops a data mining procedure to estimate the numbers, types and legal status of heavy vehicles traveling on New York State (NYS) highways. Truck data for the year 2011 are assembled from 21 Weigh-In-Motion (WIM) stations to identify overweight trucks. These are defined as trucks that exceed the 80,000 lb federally imposed Gross Vehicle Weight (GVW) limit and NYSDOT criteria which include the Federal Bridge Formula (FBF) for trucks above 71,000 lbs. The data mining algorithm is used to compare the overweight trucks extracted from the WIM files to trucks that have been issued permits as recorded in the NYSDOT permit databases. The analysis of the data shows that the numbers of overweight trucks vary depending on location and vehicle types and classes. On the average, it is estimated that about 18% of the total truck population is overweight. A further breakdown into categories shows that about 11% of the trucks may have been issued divisible load permits, 1% may have been issued special hauling permits, while about 6% may be illegally overweight.

Truck damage to highway infrastructure is a complex phenomenon that is highly dependent on each structure's location, geometric configuration, and material type as well as traffic composition, individual truck configuration and distribution of gross weights to axles. Therefore, the cost of this damage is usually estimated by comparing the effect of current truck traffic to bridge and pavement design and maintenance criteria using procedures consistent with Federal Highway Administration's (FHWA) cost allocation methods.

The analysis of the effect of overweight trucks on New York state network of bridges is implemented on a representative sample of fifty five bridges. The bridges analyzed include concrete slab bridges, concrete T-beam bridges, simple span and continuous steel girder bridges, prestressed concrete I-girder bridges, and prestressed adjacent multi-box girder bridges. These bridges have configurations, section and material properties that are representative of about 14,500 simple span bridges and 495 continuous bridges. The analysis excludes special bridge types such as culverts, trusses, arches, movable, suspension and long span bridges which are less prone to damage by individual overweight trucks. The selected fifty five sample bridges are considered to be representative of 93% of the bridges in the New York State Department of Transportation database that are prone to damage by individual overweight trucks.

The analysis accounts for the cost associated with increasing the bridge design live load from the minimum design load specified by the Association of American State Highway and Transportation Officials standard specifications AASHTO (2002) to a new load level that envelopes the effects of the overweight trucks. This cost is identified as "bridge overstress cost". The HS-20 design live load is used as baseline because most existing NYS bridges were designed for HS-20 loads and also because the HS-20 criteria were the basis for the development of the mandated FBF legal weight limits. In addition to overstress, the cost analysis accounts for fatigue life reduction of bridge members and decks.

Because of the inherent presence of reserve strength capacity in bridge design standards, most overweight trucks do not lead to bridge collapses but they increase the risk of failure by reducing the built-in safety margin. For this reason, the cost allocation approach followed in this study is identified as "Safety Margin Utilization (SMU) cost". The total SMU cost for New York state bridges is estimated to be $95M per year due to all overweight vehicles. The effect of divisible load permit vehicles is about $50M per year, while the cost of the special hauling permits is $2M. The contribution to the safety margin utilization cost due to illegally overweight vehicles is $43M.

The basic concept for allocating the cost of overweight trucks to NYS pavements consists of designing pavement sections using the Equivalent Single Axle Load (ESAL) method and determining the number of maintenance cycles using the Mechanical-Empirical Pavement Design Guide (MEPDG) method. The cost of the designs and number of maintenance interventions under current truck stream including overweight trucks is compared to the costs when only trucks that are not overweight are allowed to travel. The cost allocation study performed on the entire NYS pavement network shows that the overall cost to NYS pavements due to overweight trucks is on the order of $145 M/yr divided into $78 M/yr for divisible load permit trucks, $7 M/yr for special hauling trucks and $60M/yr for illegally overweight trucks.

Chapter 1

Introduction

1.1 Background

The U.S. highway system is continuously experiencing heavier and more frequent commercial truck traffic. Simultaneous to the increase in heavy vehicle miles travelled, the issuance of permits for overweight trucks has witnessed continuous growth. For example, an increase by more than 13% has been reported over the 10-year period between 1997 and 2007 (FHWA, 2007). The census bureau has estimated that in New York State, the Vehicle Miles Traveled (VMT) by trucks weighing over 26,000 lb increased by 15.7% from 1997 to 2002. These trucks' VMT was estimated to be on the order of 1,767 million miles in 2002. The permit trucks' VMT was estimated to be 110 million miles while the VMT of trucks that violated the weight limits was estimated at 5.4 million miles. Thus, according to the census bureau, overweight trucks would be responsible for about 6% of the total miles travelled on New York highways. However, because of the limitations of the weight enforcement process, it has been suggested that the actual percentage of the total mileage of overweight vehicles is much larger than the survey numbers indicate and could be in the range of 15% (Strauss and Semmes, 2006).

There seems to be evidence to suggest that the high volume of overweight permit and illegal trucks is disproportionately increasing the damage to the U.S. and New York State highway infrastructure systems and reducing the service lives of bridges and pavements. Damage to bridges typically occurs in the deck as well as structural elements including floor beams and girders, diaphragms, joints and bearings (Fu et al, 2003). Bridge damage is due to two factors: a) loads that exceed member capacities and b) fatigue from cyclic loading. Bridge costs associated with overweight trucks result from the accelerated maintenance, rehabilitation or replacement required to keep bridge structures within acceptable safety margins. Overweight trucks also cause a significant and disproportionate amount of damage to pavements. Damage to pavements is primarily caused by repetitive crossing of heavy vehicles. Although legal truck traffic accounts for a large percentage of damage to highway pavements, the damage caused by an overweight permit or illegal truck is usually much greater than the expected damage from any legal truck.

It is important that the New York State Department of Transportation (NYSDOT) be able to estimate the impact and cost implications of the increasing numbers of overweight vehicles in order to maintain the safety of the highway system and develop effective infrastructure management and rehabilitation strategies. In particular, developing a mechanism to evaluate the damage and cost impact of overweight trucks will help NYSDOT personnel manage their permit issuance practices, establish permit policies and procedures, and devise weight enforcement strategies.

1.2 Research Objectives

The objective of this project is to develop models for assessing the cost of overweight vehicles to New York State's highway pavements and bridges. The models must be applicable for assessing the cost of NYSDOT permit vehicles for both divisible and non-divisible loads including superloads as well as non-permit trucks violating the legal limits or trucks violating their permit limits. The effects of heavy vehicles can be classified into two categories: 1) effects related to exceeding the strength limits of bridge members, and 2) effects related to accumulated fatigue damage due to repetitive loading on bridge members and pavements.

Achieving the objectives of the study requires the following tasks and subtasks:

- Review the technical literature to investigate available methods for evaluating the effects of heavy vehicles and propose a method suitable for analyzing New York State highways.
- Estimate the numbers and types as well as the permit and legal status of heavy vehicles travelling over the New York State (NYS) highway network.
- Evaluate the effects of heavy vehicles on NYS bridges; which would require the following sub-tasks:
    - Assemble information on the most common types of NYS bridges
    - Assemble cost models to monetize the value of NYS bridges
    - Assess the effects of overstressing NYS bridges
    - Assess the effects of repetitive loads on NYS bridges
- Evaluate the effects of heavy vehicles on NYS pavements; which would require the following sub-tasks:
    - Assemble information on the most common types of NYS pavements
    - Assemble cost models to monetize the value of NYS pavements
    - Assess the effects of repetitive loads on NYS pavements
- Write programs to perform the evaluation of the effects for specific corridors and for the NYS highway network.

This introductory Chapter summarizes the findings of the literature review (also presented in Task 1.a Report of this study) and outlines the framework that is followed in the rest of the Report to achieve the objectives of the study.

1.3 Review of Previous Research

1.3.1 Overview

In recent years, several states have expressed their concern over the large number of overweight vehicles traveling over their road networks. These overweight vehicles are believed to be reducing the service lives of pavements and bridges and causing an increase in the costs associated with maintaining, upgrading and replacing the highway infrastructure system. This concern has motivated several states to sponsor independent research studies on the subject. These studies included: a) the analysis of overweight truck traffic, b) the analysis of the effect of heavy trucks on bridges, and c) the analysis of the effects of heavy trucks on pavements.

A review of the available literature indicates that states that have sponsored research projects to estimate the numbers of overweight trucks include Indiana (Reisert and Bowman, 2006), Arizona (Straus and Semmens, 2006), Texas (Lundy and McCullough, 1987), and Idaho (Carter & Burgess Inc., 2004). The states of Wisconsin (Owusu-Ababio and Schmitt R., 2005), and Texas (Jeongho et al., 2007) sponsored studies on evaluating pavement damage caused by overweight trucks while others such as Arizona, California (California PATH, 2008) and Kansas (Bai et al, 2009) also performed damage cost analyses. States that have sponsored research projects on overweight trucks and performed cost analyses of their effects on both bridges and pavements include Indiana (Sinha et al, 2005), Virginia (Virginia Transportation Research Council, 2008), Louisiana (Roberts et al, 2005), Texas (Conway, 2009) and Ohio (Campbell et al, 2009).

According to these and other related studies, the main reasons for bridge and pavement damage and service life reduction are cyclic fatigue (Bowman, 1997; or Dicleli and Bruneau, 1995), and overstress (Transportation Research Board, 1990; also Ghosn et al, 1995; Ghosn, 2000). Fatigue can be defined as the process of accumulating damage which can be caused by low levels but high numbers of stresses cycles. Overstress is a damage caused by a single application of a heavy load that exceeds the live load design limits. These two processes affect both bridge structures and pavements. Mechanistic as well as empirical models have been developed over the years to study the relationships between the frequency and intensity of the applied loads, the pavement and bridge materials and topologies, and the resulting structural damage. It has been noted that the large variety of bridge topologies and pavement types in addition to environmental effects as well as the random nature of the applied loads and that of the material properties have added to the difficulty of developing accurate models to measure and quantify the resulting damage and for obtaining accurate estimates of the costs associated with this damage. Therefore, studies on the effect of heavy truck traffic on bridges and pavements can only provide rough estimates of the damage and associated costs that can serve for comparative analyses purposes and help guide decision making processes rather than give exact estimates of cost for budgetary considerations.

1.3.2 Truck Overweight Data Collection and Classification

The weights of trucks traveling over the U.S. highway network are regulated using what is known as the Federal Bridge Formula (FBF) (FHWA, 2006). The Bridge Formula calculates the maximum allowable gross weight that can legally be applied on a bridge by any group of two or more consecutive axles of a vehicle or combination of vehicles. The Bridge Formula reflects the fact that loads concentrated over a short distance are generally more damaging to bridges than loads spread over longer distances. The FBF provides for additional gross weight as the wheel base lengthens and the number of axles increases (FHWA, 2006). The Federal Bridge Formula is:

$$W = 500 \times \left[\frac{L \cdot N}{N-1} + 12 \cdot N + 36 \right] \tag{1.1}$$

Where: W = the maximum weight in pounds that can be carried on a group of two or more axles to the nearest 500 pounds.

L = the distance in feet between the outer axles of any two or more consecutive axles.

N = the number of axles being considered.

In addition to satisfying Eq. (1.1), the Federal law imposes an 80,000 lb limit on the Gross Vehicle Weight (GVW). New York State truck weight regulations use a combination of the FBF and pre 1975 limits. According to NYSDOT regulations, a vehicle is considered overweight if it does not conform to the weight and axle spacing limits stipulated in Title III, Article 10, Section 385 of the New York State Vehicle and Traffic Law (NYS Government Documents 2005-2006 found on NYSDOT website). According to this document, legal-weight vehicles are generally defined as vehicles with three or more axles weighing a total of 34,000 lbs plus 1000 lb per foot measured from the first to the last axle. If the vehicle's gross weight is less than 71,000 lbs, the higher value from the above stated limit or the limit imposed by the FHWA Bridge Formula B (FBF) will govern. Two consecutive sets of axles closer than 96 inches apart are defined as a tandem and may carry a gross load of 36,000 lbs. For any vehicle or combination of vehicles having a total gross weight of 71,000 lbs or greater, Formula B shall apply to determine the maximum gross weight which is permitted. The total weight of legal vehicles should be less than 80,000 lbs and the maximum axle weight cannot exceed 22,400 lbs. (See section 385 of The New York State Vehicle and Traffic Law found on the NYSDOT website for additional restrictions).

Data collected by various agencies and research groups have demonstrated that a large number of trucks traveling over the U.S. highway network exceed the legal limits. While some of these trucks may be traveling illegally, many trucks that exceed the legal weight limits may have been allowed to travel following the issuance of a permit. New York State permits are classified into two groups:

- "Divisible load" which is any vehicle or combination of vehicles transporting cargo of legal dimensions that can be separated into units of legal weight without affecting the physical integrity of the load. Examples of divisible loads include: aggregate (sand, top soil, gravel, stone), logs, scrap metal, fuel, trash/refuse/garbage, among others;
- "Special Hauling" is a non-divisible load or any load or vehicle exceeding applicable length or weight limits which, if separated into smaller loads or vehicles, would:
  - Destroy the value of the load or vehicle, i.e. make it unusable for its intended purpose (i.e. manufactured/modular home, transformer, bulldozer, or a self-contained, self-propelled construction vehicle such as a crane, well-drilling rig, backhoe, pay loader, steel and concrete girders, etc.); or
  - Require more than 8 work-hours to dismantle using appropriate equipment. The applicant has the burden of proof as to the number of work-hours required to dismantle the load.
  - Compromise the intended use of the vehicle, i.e. make it unable to perform the function for which it was intended.

Special Hauling vehicles with loads having gross weights that are 200,000 lbs or greater are classified as Superloads and are subject to special requirements.

In order to estimate the cost of damage to New York highways and bridges from overweight trucks, it is necessary to collect data on the numbers, types and weights of the trucks crossing the State's highways and bridges. In recent years, the primary tools used to assemble this information have consisted of various types of Weigh-In-Motion systems (WIM). WIM data have shown large variations in the percentage of overweight trucks and illegal trucks between states and different sites within a state. In some cases, less than 1% of the trucks were found to be overweight while in others up to 30% of trucks were found to be overweight (Taylor et al, 2000). These variations are related to the truck weight limits and the legal exclusions that are allowed in different jurisdictions, the level of overweight permits that are granted, the levels of truck weight enforcement, the visibility of truck weight enforcement activity and the existence of bypass options. While a properly calibrated traditional WIM system can provide information on whether a particular truck meets the pertinent legal limits, it cannot provide information on whether the truck has been issued a permit to carry overloads. Research is currently ongoing to develop improved WIM systems known as Virtual Weigh Stations (VWS) (Cambridge Systematic Inc., 2009 a, 2009 b). VWS combine information from WIM systems and images from cameras to enable the identification of a moving truck through an Automated Vehicle Identification (AVI) algorithm. Such systems can verify the compliance of individual trucks with legal weight limits and their adherence to the overweight permits that they may have been issued. However, while one such system has been installed on one site in New York to collect some data on a trial basis, VWS have not been deployed on a large scale basis to enable the collection of necessary information for this project. The approach proposed in this study to collect information on the percentages of illegal and permitted overweight trucks on New York state highways consists of using data mining algorithms to study the vast truck database that is available from over twenty WIM sites in New York State and compare the WIM database to the permit database that NYSDOT maintains. The goal is to obtain statistical estimates of the likelihood that an overweight truck is illegal or may have been issued a permit. The data mining algorithm can then be supplemented by a Bayesian statistical updating approach to improve the statistical estimates by utilizing the limited information on the percentages of permitted and illegal overweight trucks in the traffic stream collected at the sole operating VWS system in New York. Chapter 2 of this Report summarizes the approach and the findings from the data mining analysis (which is also detailed in the Task 1.b Report – Part II).

1.3.3 Effect of Truck Weights on Highway Bridges

Bridge Overstress Criteria

Highway bridges in the United States are designed to satisfy the guidelines and specifications of the American Association of State Highway and Transportation Officials (AASHTO) (AASHTO, 2002, 2014). The AASHTO specifications require that that bridge members support the worst loading combinations the structure is expected to carry during its service life. For typical bridge configurations in the state of New York, the controlling load consists of the combination of dead load (the effect of the weight of the bridge itself) and the live load (the effect of the heavy vehicles using the bridge). The AASHTO specifications provide a set of design live load models that are believed to envelope the effects of the trucks currently using the highway system. Also, the live load models combined with the live load factors account for the possibility of having several trucks simultaneously on the bridge, the probability of exceeding the legal truck weight limits, the manner in which the effect of the applied load is distributed to individual bridge components and critical bridge sections, and the dynamic effects resulting from trucks operating at high speeds (Ghosn and Moses 1986; Nowak et al, 1993; Ghosn et al, 2011).

Most bridges in the United States were designed to accommodate either an H-15 or HS-20 loading (James et al, 1985). The recently adopted AASHTO Load and Resistance Factor Design (LRFD) specifications use a nominal design live load consisting of the HS-20 truck in combination with the distributed lane load of 640 lb/ft (AASHTO, 2014). The methods used by AASHTO to calculate bridge stresses caused by a given loading are generally conservative and actual measured stresses are generally lower than those calculated analytically. The conservative models are adopted to account for variations in material properties, inaccuracy of analysis methods, deterioration over time, and future changes in truck traffic density and weights including the numbers of overweight trucks (Ghosn, Moses and Gobieski, 1986). Accounting for the above listed uncertainties is reflected through safety factors or load and resistance factors utilized when checking the safety of bridge members as well as implicit conservative biases in the nominal values used in the design/safety checking equations. The explicit and implicit safety factors are selected so that there is only a very small probability that an applied load will exceed the load carrying capacity of bridge members within the bridge's design life (Ghosn and Moses, 1986; Nowak, 1993; Ghosn et al.; 2011).

The determination of the effects of heavy trucks on a given bridge or a bridge network may be expressed in terms of the level of overstress caused by the load, which is directly related to the reduction in the safety margin that bridges should maintain and indirectly to the risk imposed by heavy trucks (Ghosn et al, 1995; Ghosn, 2000). For the analysis of bridge overstress, most studies use as a baseline the effects of the HS-20 trucks such that bridges that may produce load effects higher than those of the HS-20 under the effect of different overweight scenarios would be considered damaged. This is justified for the purposes of this study because the vast majority of New York State bridges were designed for HS-20 criteria and because the FBF was calibrated to satisfy the HS-20 bridge design criteria.

Bridge Fatigue

In addition to the overstress analysis, an important factor that should be considered when evaluating the effect of truck weights on bridges is the fatigue life. Fatigue life is reduced due to the repetitive loading of bridges even when the load effects are lower than the overstressing limits. Each truck crossing produces one or more stress cycles in bridge components, which use up a portion of the components' fatigue lives. The occurrence of a fatigue failure is signaled by cracks developing at points of high stress concentration. These cracks may quickly propagate leading to the fracture of the component and in some cases to the collapse of the bridge (Fisher, 1997).

The analysis of fatigue has generally focused on only steel bridges, although some studies indicated that commonly used prestressed concrete spans, and concrete decks are similarly susceptible (Altan et al, 2003). The governing damage law for steel components has a third-power relationship between stress and damage, so that a doubling of stress causes an eight-fold increase in damage (Moses et al, 1987). Similar observations have also been made for other bridge components.

The fatigue resistance of a bridge component depends on the loading level represented by the stress range or the difference between the maximum stress and the minimum stress observed from a truck crossing. The fatigue life also depends on the frequency of loading. The relationship between stress ranges and loading cycles is often shown using an S-N curve similar to the one presented in Figure 1.1 (Davis, 2007).

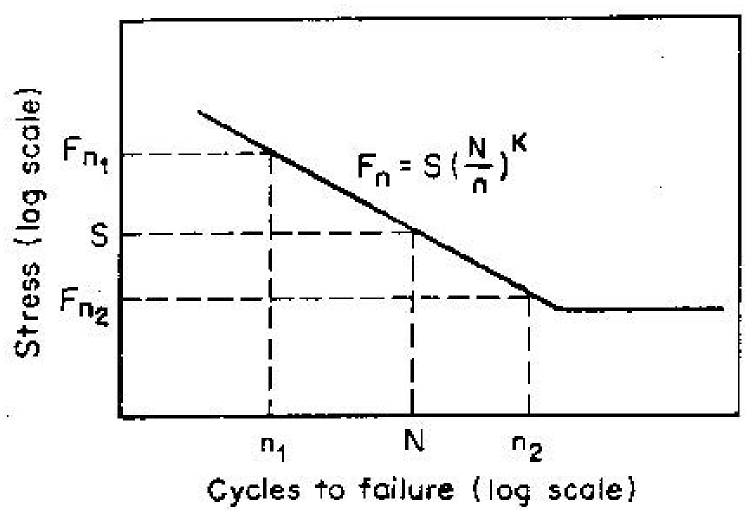

Figure 1.1 - Stress vs. Number of Cycles to Failure (S-N) Diagram

The scale of the S-N plot is log-log to show the number of stress cycles that would lead to failure for a given stress range. The horizontal line represents the stress level for infinite life. At this level, the material can theoretically endure an infinite number of cycles without developing a fatigue crack although some studies have indicated that the infinite fatigue life criterion would dissipate if a single cycle exceeds the stress limit (Fisher, 1997). S-N curves vary by material type and also by the geometry of the element (AASHTO, 2014).

The effective stress range for a loading history is defined as the stress that causes an equivalent amount of fatigue damage as the damage accumulated by stress cycles of various amplitudes. The equation that relates the effective stress range to stresses from steel bridge component loading history is known as Miner's Law and is given as (Davis, 2007):

$$S_r = \left(\sum f_i S_{ri}^3\right)^{1/3} \tag{1.2}$$

Where: f_i = fraction of cycles producing stress ranges of level *i*,

S_{ri} = stress range magnitude of interval *i*,

S_r = effective stress range.

The exponent 3 (and its inverse 1/3) corresponds to the slope of the S-N curve on the log-log scale.

For a loading history consisting of the same stress range, failure occurs when the number of applied cycles n_i at that stress range, i, is equal to the number of cycles causing failure for that stress cycle which is obtained from the S-N curve as N_i. If the damage for each stress cycle, i, is defined as $D_i=n_i/N_i$, Miner's rule assumes that failure takes place when the accumulated fatigue damage from all stress cycles reaches unity. For loading histories that involve different stress ranges, the accumulated damage is D= $\sum D_i$ and failure takes place when:

$$D=\sum D_i = \sum \frac{n_i}{N_i} = \frac{n_1}{N_1} + \frac{n_2}{N_2} + \frac{n_3}{N_3} + ... = 1 \quad (1.3)$$

where: n_i = number of stress cycles at stress range S_{ri},

N_i = number of stress cycles to produce failure at S_{ri}.

The current design provisions for highway bridges as set in the AASHTO (2014) Load and Resistance Factor Design (LRFD) Specifications and the AASHTO (1986) guide were developed by Moses et al (1987) using Miner's rule for fatigue damage. The design provisions use a factored "fatigue design truck" with axle spacing and factored axle weights calibrated to produce the same fatigue damage as that obtained from truck spectra observed on typical bridge configurations. The stress range obtained from the analysis of the bridge component under the effect of the "fatigue truck" along with the expected number of truck crossings within the bridge's service life are used to estimate the cumulative fatigue damage and verify that it remains below D=1 with an acceptable level of probability accounting for the uncertainties in estimating the fatigue life and the stress ranges.

Although not commonly considered, NCHRP 495 (Fu et al, 2003) used a similar approach for estimating the fatigue damage of concrete decks. In this case, the recommended approach is based on the fatigue damage model developed by Perdikaris et al (1993). Deck fatigue damage is obtained based on the axle weights frequency and magnitude and the ultimate shearing capacity of the deck. Equation (1.2) can then be used after replacing the exponent n=3 (and 1/3) applicable to steel components with the exponent n=17.95 (and 1/17.95) which is obtained from the S-N curves developed by Perdikaris et al (1993).

1.3.4 Effect of Truck Weights on Highway Pavements

Equivalent Single Axle Load Method (ESAL)

New York State pavement design procedures follow those set in the AASHTO (1993) Manual *Guide for Design of Pavement Structures* which are primarily based on empirical equations that estimate the response of the roadway to applied traffic loads. The input parameters include standardized traffic loading data, physical properties of pavement materials, and environment conditions. The design equations are based on results from field experiments collected during the road tests performed under the auspices of the predecessor to AASHTO known as the American Association of Highway Officials (AASHO) in the 1960s (Carey and Irick , 1960) and on practical experience. The main objective of the AASHO Road Test was to determine the relation between the number of repetitions of specified axle loads (different magnitudes and arrangements) and the performance of different flexible and rigid pavement structures. The results of the test were incorporated into the current AASHTO (1993) design criteria to ensure that pavement stresses and strains remain below critical levels.

The 1993 AASHTO pavement design guide specifies the following empirical design formula for flexible pavements:

$$\log(W_{18}) = Z_R \cdot S_0 + 9.36 \cdot \log(SN+1) - 0.20 + \frac{\log(\Delta PSI)/(4.2-1.5)}{0.4 + 1094/(SN+1)^{5.19}} + 2.32 \cdot \log(MR) - 8.07 \qquad (1.4)$$

where: W_{18} = accumulated number of 80 kN (18.000 lb.) ESALs

ZR = standard normal deviate, (reliability factor)

S_0 = combined standard error of the traffic prediction and performance

SN = Structural number

ΔPSI = difference between the initial design serviceability index, Initial PSI-Terminal PSI

M_R = Subgrade resilient modulus (psi)

The Pavement Serviceability Index (PSI) is a parameter that accounts for the loss in serviceability, it is obtained from measurements of roughness and distress, e.g., cracking, patching and rut at a particular time during the service life of the pavement (AASHTO, 1993). Roughness is the dominant factor in estimating the PSI of a pavement. The expression for the structural number SN is shown in the following formula:

SN = structural number (an index that is indicative of the total pavement thickness required)

= $a_1D_1 + a_2D_2m_2 + a_3D_3m_3 + ...$

a_i = i^{th} layer coefficient

D_i = i^{th} layer thickness (inches)

m_i = i^{th} layer drainage coefficient

(1.5)

For rigid pavements, a widely used equation has the following form:

$$\log_{10}(W_{18}) = Z_R \cdot S_0 + 7.35 \cdot \log_{10}(D+1) - 0.06 + \frac{\log_{10}\left(\frac{\Delta PSI}{4.5-1.5}\right)}{1 + \frac{1.624 \cdot 10^7}{(D+1)^{8.46}}} + (4.22 - 0.32 p_t) \cdot \log_{10}\left[\frac{S_c^t \cdot C_d \cdot (D^{0.75} - 1.132)}{215.63 \cdot J \cdot \left(D^{0.75} - \frac{18.42}{\left(E_c/k\right)^{0.25}}\right)}\right] \qquad (1.6)$$

where: W_{18} = predicted number of 80 kN (18.000 lb.) ESALs

ZR = standard normal deviate

S_0 = combined standard error of the traffic prediction and performance

D = slab depth (inches)

pt = terminal serviceability index

ΔPSI = difference between the initial design serviceability index, po, and the design terminal serviceability index, pt

S_c^t = modulus of rupture of PCC (flexural strength)

Cd = drainage coefficient

J = load transfer coefficient (value depends upon the load transfer

Ec = Elastic modulus of PCC

k = modulus of subgrade reaction

Equations (1.4) and (1.6) can be used to find the Equivalent Single Load (ESAL) that can be allowed on the pavement if the pavement thickness is known or alternatively to find the required pavement thickness given the expected ESALs.

The ESAL factor relates various axle load combinations to the standard 18,000 lb single axle load. Simplified models and rule-of-thumbs have also been recommended to simplify the calculation of the ESALs.

Mechanistic-Empirical Pavement Design Guidelines (MEPDG)

The method in the Mechanistic-Empirical Pavement Design Guidelines (MEPDG) was developed to update the 1993 AASHTO Guide for Design of Pavement Structures, which is primarily based on empirical observations from the AASHO Road Test in the 1950s. By using newer data collected as part of the Long-Term Pavement Performance (LTPP) program, the MEPDG allows for pavement damage inferences that would be harder to justify from the limited designs and traffic levels covered by the AASHO Road Test. The AASHTO Mechanistic-Empirical Pavement Design Guide (MEPDG) which was based on the models assembled in NCHRP 1-37A (NCHRP, 2004) is a mechanistic-empirical (M-E) method for evaluating pavement structures. Structural responses (stresses, strains and deflections) are mechanistically calculated based on material properties, environmental conditions, and loading characteristics. The responses can be obtained by performing advanced finite element analyses, or other mechanics-based analyses of pavements under the effect of actual truck loads considering the nonlinear material properties and thickness of the different pavement layers and how these properties may be affected by temperature changes. While one can use any advanced finite element software package for the analysis, specialized programs have been developed to ease the modeling and computational process (see for example Huang, 2004). The stresses, strains and deflections obtained from the analysis are subsequently used as input in an empirical model to compute distress performance predictions. Thus, the MEPDG still depends on empirical models to predict pavement performance from calculated structural responses and material properties. The accuracy of these models is a function of the quality of the input information and the calibration of empirical distress models to observed field performance. Two types of empirical models are used in the MEPDG. One type predicts the distress directly (e.g., rutting model for flexible pavements, and faulting for rigid); the other type predicts damage which is then calibrated against measured field distress (e.g., fatigue cracking for flexible pavements, and punchout for rigid).

The use of MEPDG to design a pavement section is not as straightforward as the ESAL method in the 1993 AASHTO guide, in which the structure's thicknesses are obtained directly from the design equation. Designing pavements with MEPDG requires a very elaborate iterative process in which predicted performance of a selected pavement structure is compared against the design criteria. The structure and/or material selection need to be adjusted until a satisfactory design is achieved. For these reasons, the use of MEPDG for the design of ne pavements may not be practical. However, MEPDG equations have been found to be useful for evaluating the damage to existing pavements as will be explained in Chapter 4 of this Report.

1.3.5 Cost Allocation Models

FHWA Cost Allocation Study

Most studies that attempted to put a dollar figure to the damage caused by heavy trucks on pavements and bridges used the approach followed in the FHWA (1997) cost allocation project as the model to follow. In the FHWA model, highway costs were divided into four primary categories: pavement costs, bridge costs, system enhancement costs, and other attributable costs. Costs for the additional pavement thickness needed to accommodate anticipated traffic are allocated based on the ESAL pavement design procedures. Costs for pavement reconstruction, rehabilitation, and resurfacing (3R), which are estimated to represent 25 percent of total Federal obligations in 2000, are allocated to different vehicle classes on the basis of each vehicle's estimated contribution to pavement distress. The pavement costs are allocated incrementally and assigned to various vehicle classes on the basis of their vehicle miles travelled (VMT). Costs of constructing new bridges are also allocated to vehicles using an incremental approach. As with new pavements, costs for constructing the base facility of a new bridge are allocated to all vehicle classes in proportion to their VMT. Incremental costs to provide additional structural capacity are allocated to those vehicle classes that require the greater strength. It is however noted that, unlike the pavement analysis process, the FHWA bridge cost allocation model did not consider bridge distress from repetitive loading which other studies have focused exclusively on.

NCHRP Study

Several other organizations and state departments of transportation have also sponsored studies on evaluating the cost inflicted by heavy trucks on highway bridges and pavements. Among these one should mention the National Cooperative Highway Research Program (NCHRP) report that studied the effect of changes in truck weight regulations on bridge network cost (Fu et al, 2003). The study showed that both cyclic fatigue and overstress impact bridge costs. The fatigue analysis for existing steel bridges followed the AASHTO (1990) procedure to evaluate the reduction in bridge fatigue life. The service life of the concrete deck is also calculated using a model similar to that used for steel bridges. Analysis of bridge overstress was considered both for existing and new bridges. The analysis of existing bridges was based on a rating factor approach. The rating factor relates the ratio of the moments of the design vehicle and the overweight vehicle to the existing rating factor. If the rating factor for an existing bridge is already below 1.0 for current legal loads, then the bridge should have been already slated for replacement and the cost is considered with those of new bridges. If under the effects of an overweight truck the rating factor for an existing bridge falls below 1.0, the bridge is considered to be overstressed for that overweight truck. For new and replacement bridges, an incremental cost method is used. The impact cost is calculated by comparing the cost of a new bridge under the design load and the cost of designing the same bridge under the overweight truck load. It is noted that the object of the NCHRP study was to evaluate the cost to upgrade the existing bridge stock if new truck weight regulations are enacted. This is different than the goal of this study which aims to evaluate the effects of currently running overweight trucks on the New York State highway infrastructure. Thus, the cost analysis model used in the NCHRP study is not directly applicable to the needs of this project.

Louisiana DOTD Study

The Louisiana Department of Transportation and Development (LADOTD) sponsored a study (Roberts et al, 2005; and Saber and Roberts, 2008) on the effect of overweight vehicles on Louisiana highways and bridges. The researchers analyzed the economic impact of different types of trucks categorized according to the FHWA classification. The first report analyzed the effects of different types of overweight permitted vehicles transporting timber, lignite coal and coke fuel for four load scenarios: 1) Maximum truck weight for 3S-2 vehicles carrying these products remain within 80,000 lbs; 2) the maximum permit weight is 86,000 lb; 3) The maximum permit weight is 88,000 lb; and 4) the permit weight is 100,000 lbs. The weight of 100,000 lbs is the maximum GVW allowed for trucks carrying sugarcane in the state of Louisiana. An additional analysis studied the effects of using a tandem load of 48,000 lb. (48-kips). The second report (Saber and Roberts, 2008) showed the effect of a truck type known as sugarcane truck hauler (having configurations consistent with Class 9 or Class 10 vehicles according to FHWA classification).

The methodology for analyzing the effect of these loads on pavements followed the 1986 AASHTO design guide (AASHTO, 1986) which may have slight differences from the 1993 method (AASHTO, 1993). Differences in the life of an overlay were calculated for different GVW scenarios and overlay thickness and costs were determined for a 20-year analysis period. The present-net-worth costs were calculated to represent the cost for control sections on the routes carrying each commodity. The methodology for analyzing the bridge costs was developed by determining the shear forces, moments and deflections induced on each bridge type and span, and developing a cost to repair fatigue damage for each vehicle passage.

Ohio DOT Study

Campbell et al (2009) published a study by the Ohio Department of Transportation (ODOT) describing a methodology for allocating the costs of damage to pavements and bridges due to overweight trucks. They accounted for the effect of trucks on pavements using the ESALs methodology with a focus was on concrete pavements. In a first phase, they designed the pavement thickness using traffic data including overweight trucks heavier than the 80,000-lb legal limit. Subsequently, they removed the overweight trucks from the database and performed the design and found the required thickness for legal weights up to 80,000 lb. The results show that the pavement thickness was reduced by 1 in if the heavy trucks were removed from the data. In addition to the design cost, the cost of applying the required 2-in overlay at every 12-year maintenance cycle is included. A percentage of the costs is allocated to each type of overweight trucks based on either their ESAL's or their VMT. Specifically, the additional 1.0-in design thickness, which was necessary to account for the effect of overweight vehicles, was allocated to the overweight trucks based on their ESALs. The cost of the 2-in overlay was divided to all the vehicles based on their VMT's.

The same approach followed for pavements was also used for the analysis of the 12,618 Ohio bridges. The road network was divided into functional classes according to the FHWA classification and the number of bridges in each class was determined. A bridge design life of 75 years is assumed. The Ohio study assumed that the designs did not differentiate between designs with overweight or no overweight and no differentiation was made based on the existing current ratings. The analysis simply focused on allocating the costs of the current bridge assets and the preservation costs. The asset cost represents the replacement cost of a bridge, while the preservation cost includes the deck replacement that is due every 40 years of the bridge life and the superstructure painting cost that happens generally every 25 years. The preservation costs exclude the costs of regular maintenance, deck overlay, annual inspection, deck patching and sealing and deck cleaning, because the total costs are relatively minor and did not substantially affect the results of the analysis. The ODOT study estimated the unit asset cost based on the surface area of the bridge for each type based on typical bridge dimensions.

In the ODOT study, the authors did not design any bridges and did not perform the incremental cost analysis. Instead, they multiplied the total area of each bridge category by the unit cost estimated from historical data to obtain the total asset cost which they estimated to be $191,292,000. After completing this operation, they multiplied the total value by the percentage of overweight vehicles (11.08%) as provided by the FHWA (1997) study to allocate the costs due to overweight trucks.

To allocate the preservation costs, they estimated the number of deck replacement (every 40 years) and painting application (every 25 years) required in the entire bridge life. The allocation of bridge preservation costs to the overweight categories was performed by multiplying each road category cost times the percentage of the VMT of overweight vehicles relative to the total VMT for all vehicle types.

Summary of Previous Cost Analysis Studies

In addition to the studies listed above, many other studies attempted to put a dollar figure to the damage caused by heavy trucks on pavements and bridges. Most of the studies used the FHWA (1997) cost allocation project as the model to follow. The results have been summarized in different ways. For example, the Indiana DOT study (Sinha et al, 2005) estimated an annual cost for pavement in the range of $354-397 million, while the annual average cost of bridge preservation was estimated to be in a range of $185-199 million. The Arizona DOT study (Strauss and Semmens, 2006) estimated the cost of damage to their pavements from heavy vehicles to be around $210 million per year. Other States such as Virginia and Kansas provided the damage cost in terms of cost per mile. The Virginia DOT study (Virginia Transportation Research Council, 2008) estimated a cost of $0.036 per ESAL-mile for pavements. Considering that the range in ESAL is around 0.4 to 1.8 EASL/truck, the estimated cost for trucks per mile becomes $ 0.011-0.065. The Kansas DOT study (Bai et al, 2009) estimated the pavement damage cost per permit truck-mile to be approximately $0.02. The Louisiana DOT study (Saber and

Roberts, 2008) provided an impact cost per truck in the range of $346 – 4,377 per truck/year depending on the axle weight distribution for the permit trucks leading to a cost per permit truck per mile of $0.017 to $0.21 per truck per mile. The higher value is related to an increase in the allowed tandem weight of the permit to about 48,000 lb per tandem. The Ohio DOT study (Campbell et al 2009) estimated that annually, overweight trucks constitute about 1.00% of the total truck annual VMT. Also they estimated the annual cost per mile due to the ESAL damage to pavements and bridges to be about 0.05 $/mile. The annual preservation cost per mile was found to be about 0.008 $/mile.

1.4 Research Approach

Based on the review of the literature related to topics of interest to this project concerned with the effect of overweight trucks on New York State highway bridges and pavements, the research plan followed during the course of this study includes the following features:

- Data on the types and weights of legal trucks as well as overweight trucks currently traveling over New York State highways are collected from Weigh-In-Motion data.
- The WIM data is analyzed and statistical estimates on the percentages of overweight trucks that may have been issued permits compared to those that may be illegally overweight are obtained based on comparisons between the WIM databases and the NYSDOT database of permit trucks. This information is supplemented with the limited data available from one site where the NYSDOT has installed WIM equipment and cameras capable of identifying those overweight trucks that have been issued permits as opposed to the illegally overweight trucks.
- Account for the fatigue damage for steel and prestressed concrete bridges using the AASHTO LRFD fatigue method.
- Because most damage to bridges from overweight truck is reflected in the decks, it is proposed to include the fatigue damage of decks as done in the NCHRP 495 study.
- The proposed fatigue damage is related to the reduction of bridge life due to the overweight vehicles. The damage costs is allocated proportionally to the life reduction for each overweight category i.e. trucks carrying divisible load permits, those carrying special hauling permits and those that are illegally overweight.
- The effect of overstressing bridge members is also estimated and the cost allocation for overstress is based on the classical incremental cost analysis approach.
- The proposed base line for evaluating overstress bridge damage is the design capacity. The reduced capacity of currently underrated bridges should not be assigned to the current network users as the causes of the low ratings are not necessarily due to overweight trucks but could be due to the use of previous generation codes, material deterioration, or environmental factors. On the other hand, the costs of overdesigned bridges should be assigned to the overweight trucks because they are benefiting from the excess capacity.
- The final bridge costs will be an envelope for the fatigue and overstress costs.

- The MEPDG method is used for estimating the damage to pavements instead of using the ESAL's method adopted by the Ohio DOT and the Louisiana studies.
- While the Ohio Dot study concentrated on the cost of designing pavements to higher standards to accommodate the heavier truck weights assuming that maintenance is due to the effect of environmental deterioration, the LADOTD study used existing design as the base line and considered the cost of increasing overlay thickness needed when maintenance is scheduled to accommodate the overweight trucks. The approach proposed in this report utilizes a hybrid of both methods. The cost of designing pavements to higher standards will be supplemented with the cost of increasing the number of maintenance cycles if warranted by the presence of overweight trucks.
- For pavement design this Report uses the ESAL method because that is the current standard method in New York and because of the difficulty of applying the MEPDG method for pavement design. For evaluation of pavement distress, the MEPDG method is used because the ESAL equations are not suitable for that purpose while the MEPDG approach was particularly calibrated for that purpose.

1.5 Report Outline

The procedures proposed in this study to monetize the effects of overweight trucks on New York State highway pavements and bridges are presented through the following Chapters:

- Chapter 2: Statistical Analysis and Classification of New York State Trucks
- Chapter 3: Modeling the Effects of Overweight Trucks on New York State Bridges
- Chapter 4: Modeling the Effects of Overweight Trucks on New York State Pavements
- Chapter 5: Summary of Findings

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Chapter 2

Statistical Analysis and Classification of New York State Trucks

2.1 Background

Recently observed increases in the numbers of permitted and illegal overweight trucks traveling over U.S. highways have raised concerns over their contributions to the reduction in the service lives of pavements and bridges and the costs of maintaining, upgrading and replacing the highway infrastructure system. Cost allocation studies are used by transportation officials to help their asset management processes and to establish truck traffic and permitting policies taking into consideration information on the composition of overweight trucks and their permit classifications. In recent years, cost allocation studies have heavily relied on data assembled by Weigh-In-Motion (WIM) systems which provide information on traffic counts, truck axle configurations and weights for various highway classes and economic regions. However, WIM data by themselves do not provide information on the numbers of illegal overweight trucks because many of the overweight trucks may have been issued permits that allow them to operate on a yearly basis or on a trip-by-trip basis. The object of this Chapter is to develop a data mining procedure to identify and classify overweight vehicles collected by New York State WIM systems into different permit and illegal categories.

The first step of the proposed data mining procedure establishes a set of rules that are satisfied by different types of permit trucks. These rules are inferred from a review of NYSDOT permit databases. In a second step, a search algorithm is used to check each vehicle in a WIM database and identify whether it violates any of a jurisdiction's legal weight limits. Subsequently, every overweight truck's axle weights, axle spacings, total length and gross vehicle weight are checked to verify whether these characteristics match any of the criteria established when setting up the data mining rules. The overweight vehicles are separated based on the likelihood of having been issued a particular type of permit or if they are potentially illegal. The validity of the algorithm is demonstrated by analyzing truck data collected at a WIM site in upstate New York. The algorithm's output showed reasonable agreement when compared to the results of a truck survey performed by the New York State Department of Transportation (NYSDOT). A parametric analysis is executed to assess the sensitivity of the results to the level of accuracy of the WIM system. The data classified through this algorithm are important for allocating the costs that overweight trucks impose on New York State highways to the different types of trucks based on their effects on pavements and bridges.

2.2 WIM Technology

WIM systems record information on truck axle weights, axle spacing, speeds and times of arrival without stopping the vehicle. Because the data is collected while the vehicle is moving without the truck drivers' knowledge, these systems are believed to provide unbiased statistical information on truck weights and traffic patterns. WIM system technology is quite diverse; the most common systems are those that use piezoelectric strips and bending plates although other systems that use fiber optics and instrumented bridges and culverts have also been deployed (Sivakumar et al ,2011). The information collected on individual trucks over long periods of time can be used to assemble statistical databases of truck volumes, Gross Vehicle Weights (GVW) as well as the class and axle configuration of vehicles that are travelling over the highway network. The implementation of WIM technology is quickly spreading and an enormous amount of data is currently being collected by various transportation agencies. Specifically, the New York State Department of Transportation currently operates twenty-five WIM stations placed at representative locations covering major portions of the state's highway network. Although WIM data have been mostly used for transportation planning purposes (Huang et al, 2012), they have also been used to develop live load models for application in bridge design codes, for evaluating the safety of existing bridges and for permit checking (Ghosn et al, 1986; Sivakumar et al, 2011; Ghosn, Sivakumar and Miao, 2012). WIM data have also been used for assessing the damage to highway pavements as well as for cost allocation studies (Parker et al, 2009 and Conway, 2009).

The most important WIM data input for cost allocation purposes and for bridge safety and permit rating are truck weights and truck axle configurations and classifications including those of regular, permit and illegal trucks. As an example, Figure 2.1 shows a typical frequency histogram of gross weight data for all the trucks assembled for the year 2009 from the WIM station in Schodack in upstate New York located on Interstate I-90 after exit 12, near Albany. The figure shows how the histogram of gross weights for trucks heavier than 20,000 lb follows a tri-modal distribution, with the first mode representing the mostly unloaded or lightly loaded trucks, the second mode representing heavy trucks close to the legal gross weight limit and the third mode representing trucks that exceed the U.S. Federally-mandated 80,000 lb gross weight limit..

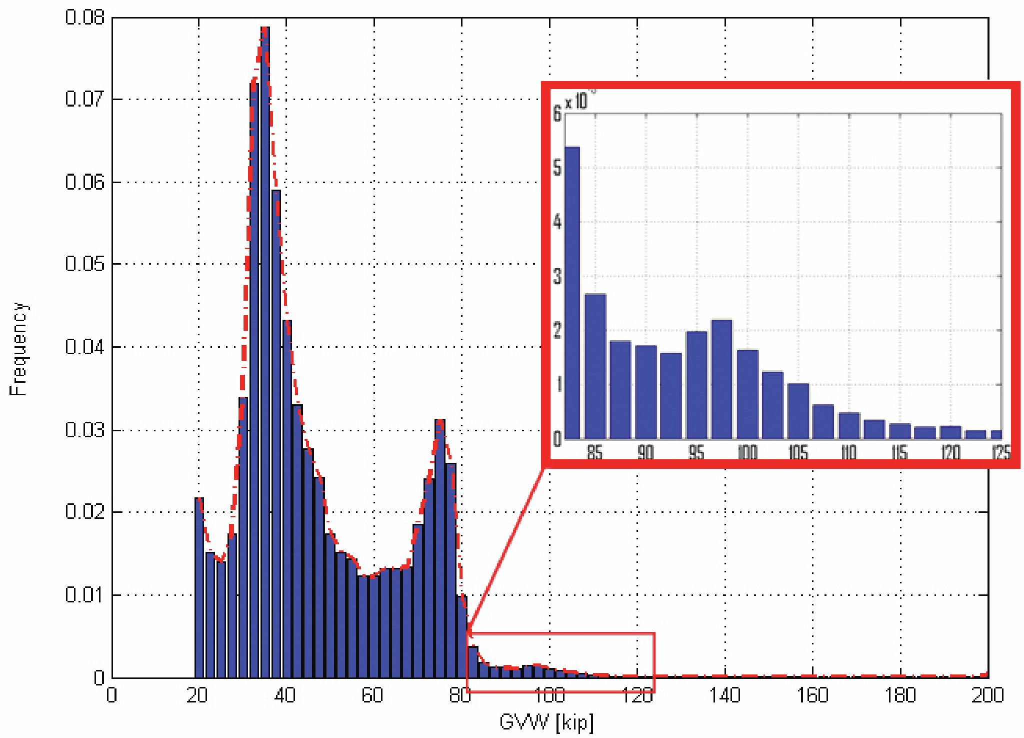

Figure 2.1 – Gross Vehicle Weight (GVW) Distribution

Heavy trucks are usually grouped into classes following the U.S. Federal Highway Administration (FHWA) truck type classification. FHWA provides thirteen classes based on the number of axles and axle spacing. Class 1 vehicles are motorcycles, Class 2 is for passenger cars, Class 3 vehicles are Pickups/Vans and Class 4 is for buses. These four classes are not usually included in cost allocation studies, bridge safety analyses or pavement damage assessment because their effects are usually smaller than those of heavy trucks. Figure 2.2 shows the FHWA classes for the most typical heavy vehicle profiles. FHWA provides an algorithm that classifies WIM data into these 13 categories based on truck axle configurations rather than on traditional classifications which usually rely on visual observations (Quinley, 2010). In this study, trucks that do not fit any of the FHWA classes are placed into a new class identified as Class 14.

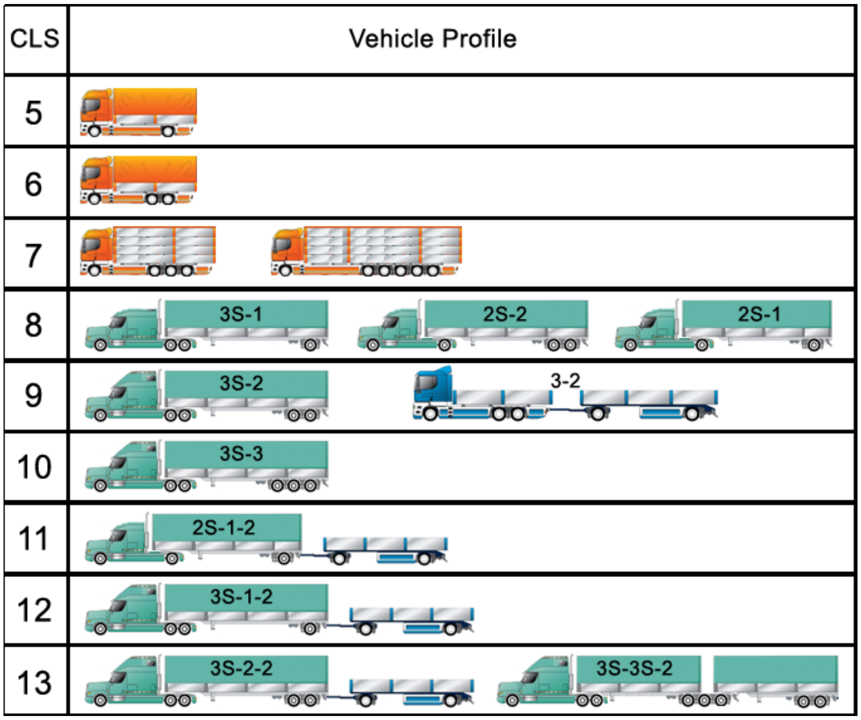

Figure 2.2 – List of FHWA Vehicle Classes

As an example, Figure 2.3 shows the FHWA classes of the 175,432 trucks that crossed the instrumented lane of the two-lane I-90 WIM site in upstate New York in the year 2009. The figure shows that the highest percentage of trucks consists of semi-trailers of Class 9 which dominate the histogram at about 80%. The second highest percentage belongs to the single unit trucks of Class 5 which are about 10% of the trucks. Semi-trailer trucks of Class 8 constitute about 8% of the total trucks. These observations are consistent with those made at other sites throughout the U.S. (Sivakumar et al, 2011).

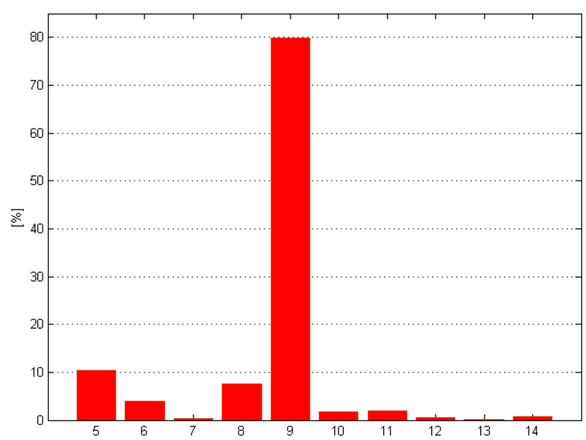

Figure 2.3 – Distribution of I-90 Trucks per FHWA Class

2.3 Permit Vehicles

The New York State Department of Transportation (NYSDOT) specifies two types of permits. The first type is issued for vehicles that carry divisible loads and are of legal size. These trucks are issued divisible Load (DV) permits which are generally valid for one year. The second type is issued to vehicles carrying a cargo that cannot be split into smaller parts and may require a special vehicle configuration. Such trucks are issued special hauling (SH) permits. Special hauling permits may be issued for a single trip on a pre-designated route or for a year with a limited range from a source location. More information on New York State permit policies and criteria is provided on their web site (NYSDOT, 2011).

WIM systems in New York State are not connected to plate readers that are capable of identifying whether an overweight truck has been issued a permit. Furthermore, a review of the NYSDOT permit databases has revealed that many of the divisible load and the special hauling permit vehicles have axle configurations that make them indistinguishable from those of regular non-permit trucks. This is why a special data mining algorithm augmented by a statistical analysis is used in this study to help estimate the percentages of the overweight trucks that may be holding permits and those which are potentially illegally carrying overweight.

NYSDOT like most states maintains a database giving the weight and axle configuration for each permit issued during a calendar year. This information will form the basis for developing the data mining rules and constitutes the main component of the proposed search algorithm described next.

2.4 Data Mining Parameters

A necessary first step in the process of establishing the data mining method for estimating the percentage of permit trucks consists of identifying the types and classes of the trucks in the permit and the WIM databases. This requires the classification of the trucks into groups based on the legal weight limits that they exceed. To identify which limit is being exceeded, an OverWeight-Vehicle-Code (OWVC) system is first developed. Another proposed grouping is related to the axle configuration of the trucks. Although the FHWA classification is regularly used for that purpose, a review of the permit and WIM databases has demonstrated that the FHWA classes are too broad for the purposes of this study and need to be complemented by additional information. An additional axle Spacing-Code (SPC) is therefore proposed along with a Total Length-Code (TAL). This section describes the implementation of the proposed overweight coding, total length and axle spacing coding for New York permit and WIM databases.

2.4.1 Overweight Truck Categorization (OWVC)

According to NYSDOT, a vehicle is considered overweight if it does not conform to the weight and axle spacing limits stipulated in Title III, Article 10, Section 385 of the New York State Vehicle and Traffic Law (NYS Government Documents 2005-2006) (NYS Section 385). According to this document, legal-weight vehicles are generally defined as vehicles with 3 or more axles weighing a total of 34,000 lb plus 1,000 lb per foot measured from the first to the last axle. This limit, which was established before the FHWA laws were instituted, is known as the New York State grandfather exclusion. If the vehicle's gross weight is less than 71,000 lb, the higher value from the above stated limit or the limit imposed by the Federal Highway Administration (FHWA) Bridge Formula B (FBF) will govern (FHWA, 2006). Bridge Formula B is given as:

$$W = 500\ \left(\frac{L \times N}{N-1} + 12N + 36\right) \tag{2.1}$$

where: W = overall gross weight on any group of two or more consecutive axles in pounds,

L = distance in feet from the center of the foremost axle to the center of the rearmost axle of any group of two or more consecutive axles,

N = number of axles in the group under consideration

Two consecutive sets of axles closer than 96 inches apart are defined as a tandem and may carry a gross load of 36,000 lb in New York State (NYS Section 385). Three consecutive axles spaced at a distance of 144 inches or less should not exceed 45,000 lb (FHWA, 2011). For any vehicle or combination of vehicles having a total gross weight of 71,000 lb or greater, Formula B shall apply to determine the maximum legal weight for any consecutive group of axles. The gross weight of legal vehicles should be less than 80,000 lb and the maximum axle weight cannot exceed 22,400 lb (NYS Section 385). (See section 385 of The New York State Vehicle and Traffic Law for additional restrictions).

The five digit coding scheme developed in this study as illustrated in Figure 2.4 is used to identify the legal weight status of each truck and the legal weight limit that is exceeded. Each digit identifies which legal limit is exceeded. If none of the legal limits is exceeded, the number zero is entered for each of the five digits. Starting from left to right, the first digit indicates whether the vehicle exceeds the New York State grandfather rule (code 1), both the FBF formula and the New York rule for trucks under 71,000 lb (code 2) or the FBF formula for all other trucks (code 3). The second digit indicates whether the vehicle exceeds the gross weight limit of 80,000 lb (code 4). The third digit shows whether the vehicle exceeds the axle limit of 22,400 lb (code 5). The fourth digit indicates whether the truck exceeds the tandem limit of 36,000 lb (code 6), and finally the fifth digit indicates whether the vehicle exceeds the limit of 45,000 lb for the three-axle group (code 7). Figure 2.4 shows an example of a vehicle that was assigned the overweight code 34060. The code indicates that the truck which weighs more than 71,000 lb exceeds the FBF formula that is why the first digit is assigned the code 3, the gross vehicle weight limit is also exceeded which explains the code 4 assigned to the second digit, and exceeding the tandem weight limit is indicated by the code 6 assigned to the fourth digit. Since neither the axle weight nor the tridem limits are exceeded, the third and fifth digits are assigned the code 0.

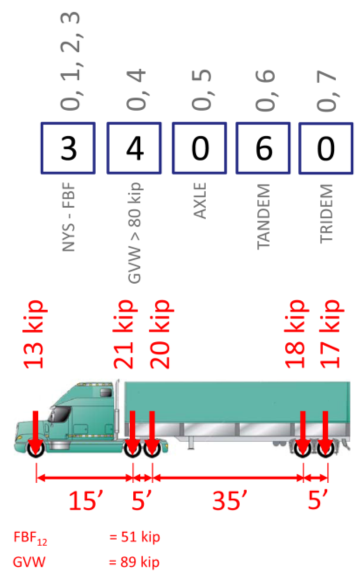

Figure 2.4 - Overweight Code Classification (OWVC)

The information provided by existing WIM systems is sufficient to determine whether a truck in the WIM database is within the legal weight limits. If the legal limits are not met and a truck is assigned a five-digit different than 00000, the truck is classified as overweight but it will not be possible to determine from the WIM data alone whether it is a permit truck or it is illegally overweight. To help estimate the numbers of trucks that may be illegal, the data mining algorithm described in Section 2.5 of this chapter has been proposed.

2.4.2 Total Axle Length Code (TAL)

In addition to grouping the trucks into different classes based on the FHWA classification explained earlier, the trucks in each class are further divided into subgroups based on their Total Axle Length (TAL). The subgrouping is executed based on the codes defined in Table 2.1. For instance, trucks with total axle length less than 30-ft are grouped into TAL=0, those with total axle length equal to or greater than 30 but less than 40-ft are assigned a code TAL= 1, and so no. This classification which is not related to any physical properties of the trucks or their load effects is simply adopted to increase the level of accuracy in the definition of the data mining rules.

Table 2.1 – Vehicle Classification According to the Total Axle Length (TAL)

total axle length, L_{axle} [ft.]	**Code**
L_{axle} < 30	TAL=0
30 <= L_{axle} <40	TAL=1
40 <= L_{axle} <50	TAL=2
50 <= L_{axle} <60	TAL=3
60 < =L_{axle} <70	TAL=10
70 <= L_{axle} <80	TAL=11
80 <= L_{axle} <90	TAL=12
L_{axle} >= 90	TAL=13

2.4.3 Axle Spacing Code (SPC)

A key element in the permit recognition analysis is the axle configuration of a vehicle. In fact, an overweight permit record, either divisible (DV) or special hauling (SH) is characterized by two parameters, the axle weight distribution and the axle spacing configuration. The concept for complementing the FHWA classification is to assign a code similar to a "bar code" based on the spacing between consecutive axles. For axle spacings greater than 0 and less than 5 ft, the rule assigns a value equal to 1; for axle spacings greater or equal than 5 but less than 10 ft, the code is assigned a value equal to 2; for axle spacings greater or equal to 10 but less than 25 ft, the code is assigned the value of 3; for axle spacings greater or equal than 25 but less than 40 ft, the code assigned is equal to 4. Finally if the axle spacing is greater than or equal to 40 ft, the rule assigns a code equal to 5. Figure 2.5 shows the axle spacing coding system as well as three example codes for three types of vehicles. For example, the last vehicle in the figure is a six-axle semi-trailer, with axle spacings equal to 13 ft, 4 ft, 18 ft, 19 ft and 4.2 ft. The axle spacing code (SPC) assigned for this configuration is a 5 digits code equal to 31331. Like the total length grouping explained in Section 2.4.2, this axle spacing grouping is not related to physical properties of the trucks or their load effects and is only adopted for simplifying the data mining algorithm.

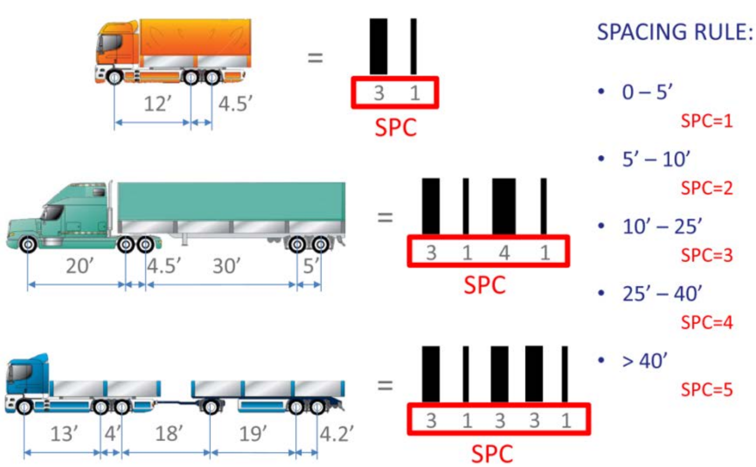

Figure 2.5 - Axle Spacing Code Classification (SPC)

2.5 Data Mining Algorithm

2.5.1 Establishing Data Mining Rules

The New York State Department of Transportation assembles different permit databases for the special hauling and the divisible permits. The special hauling database is also split into single trip permits and multi-trip permits. The analysis of these databases serves to establish a dense matrix of clusters obtained by grouping the vehicles in each database according to the trucks' FHWA classification, number of axles, their Spacing Code (SPC), and their total vehicle axle length code (TAL). The characteristics of the trucks in each cluster are used to associate them with minimum and maximum values for the axle spacings, the maximum axle weights and the maximum gross vehicle weight. The process of dividing the permit trucks into clusters and sub-clusters can be schematically described using the analogy of dividing units into containers which are in turn divided into crates which are split into cases containing different boxes. This process is schematically described in Figure 2.6. A permit truck that belongs to one box will be known to have a unique association with a set of axle weight and spacing characteristics. Since all the permits are identified with a particular cluster, these clusters can then be used to identify whether a particular truck extracted from the WIM database could potentially have been issued a permit.

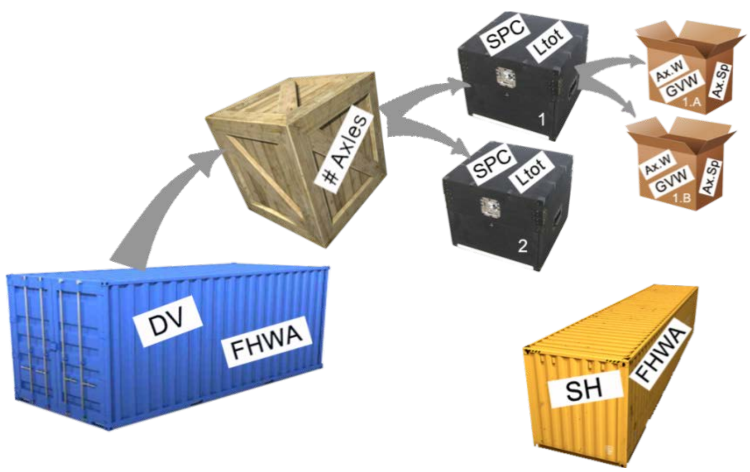

Figure 2.6 – Permit Database Clustering Rules

The process of clustering is explained using the four trucks shown in Figure 2.7. Assuming that the clustering process is to be executed for a group of divisible permit trucks (DV) having the configurations of Truck 1 and Truck 2 of Figure 2.7 and another clustering is to be executed for another group of special hauling permits (SH) having the configurations of Truck 3 and Truck 4. First, the two divisible trucks are checked whether they are overweight, which as seen from the information provided in Figure 2.7 is true for both trucks which weigh more than 80 kip. Also, both trucks belong to FHWA Class 9. Both trucks have five axles (NAX). Both trucks have total lengths greater than 50-ft and less than 60-ft which indicates that their total axle length code is TAL=3 as explained in Section 2.4.2. The spacing of their axles indicates that the spacing code is SPC=3242 as explained in Section 2.4.3. Therefore, the cluster rule for this set of permit trucks is identified as: Class 9, with TAL=3 and SPC=3242. The weight characteristics for this DV cluster are associated with a maximum gross weight equal to

91 kip, while the first axle weight has a maximum weight of 15 kip, the second axle weight an upper limit of 22 kip, the third, fourth and fifth axle weights have maximum values of 20, 18 and 19 kip respectively. The axle configuration characteristics are associated with a minimum value of 12 ft and a maximum of 14 ft for the first axle, the second axle spacing has minimum and maximum values of 5 and 5.1 ft. The third axle spacing ranges between 32 and 35 ft. The fourth axle spacing is between 5 and 5.2 ft. The same procedure is applied to the two special hauling trucks. Both of these are overweight with gross weights larger than 80 kip: Truck 3 has a gross weight of 100 kip, while Truck 4 has a gross weight of 99 kip. Both trucks belong to FHWA Class 9 and have 5 axles. Their length has the code TAL=3 and a configuration SPC=3242 following the coding rules described in Sections 2.4.2 and 2.4.3. Therefore, the cluster rules that identify these trucks are: Class 9, with TAL=3 and SPC=3242. For this SH cluster, the maximum gross weight is equal to 100 kip, the first axle weight has a maximum weight of 16 kip, the second axle weight an upper limit of 23 kip, the third, fourth and fifth axle weights a maximum value of 22, 20 and 21 kip respectively. The first axle spacing has a minimum value of 13 ft and a maximum of 14.5 ft, the second axle spacing's minimum and maximum values are 5.2 and 5.3 ft. The third axle spacing ranges between 30 and 31 ft. The fourth axle spacing is between 5.1 and 5.3 ft. The two rules are summarized in Tables 2.2 and 2.3.

Similar clustering rules are implemented for all the trucks in the divisible and the special hauling permit databases. These rules will serve in the pattern recognition algorithm to identify whether a truck from a WIM database could potentially be a permit truck or not. The pattern recognition process is explained next.

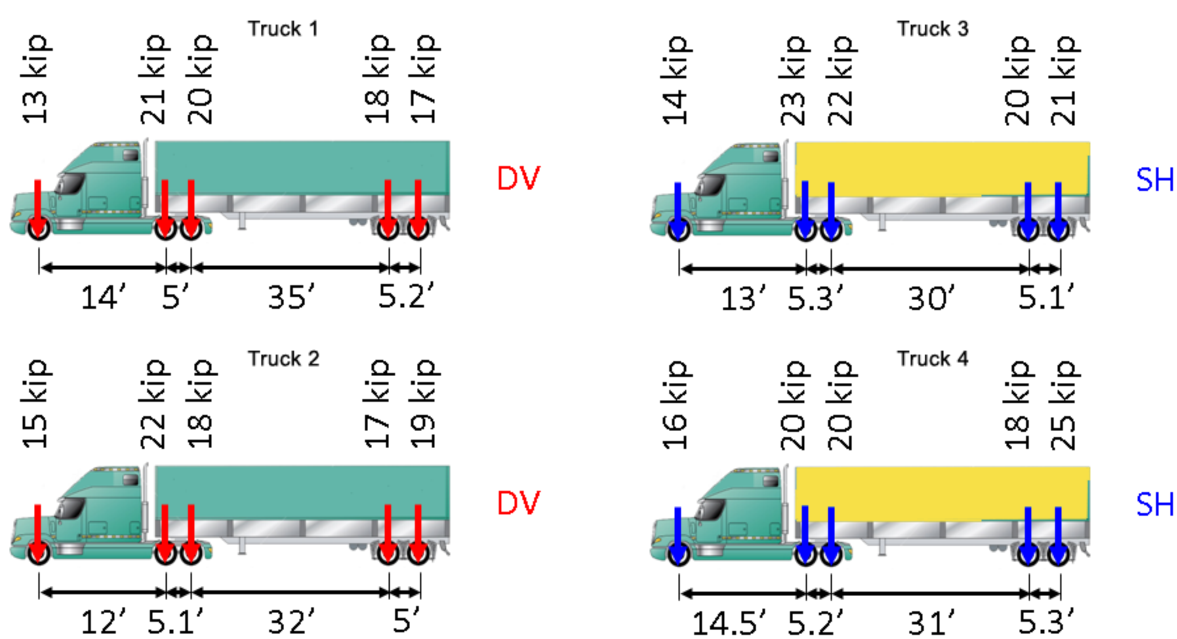

Figure 2.7 – Example of Divisible (DV) and Special Hauling (SH) Permits Analyzed for Clusters

Table 2.2 – Example DV Data Mining Rule

	FHWA class	TAL	SPC	NAX		Sp1 [ft.]	Sp2 [ft.]	Sp3 [ft.]	Sp4 [ft.]	GVW [kip]	Ax1 [kip]	Ax2 [kip]	Ax3 [kip]	Ax4 [kip]	Ax5 [kip]
DV	9	3	3242	5	Min	12.0	5.0	32.0	5.0						
					Max	14.0	5.1	35.0	5.2	91	15	22	20	18	19

Table 2.3 – Example SH Data Mining Rule

	FHWA class	TAL	SPC	NAX		Sp1 [ft.]	Sp2 [ft.]	Sp3 [ft.]	Sp4 [ft.]	GVW [kip]	Ax1 [kip]	Ax2 [kip]	Ax3 [kip]	Ax4 [kip]	Ax5 [kip]
SH	9	3	3242	5	Min	13.0	5.2	30.0	5.1						
					Max	14.5	5.3	31.0	5.3	100	16	23	22	20	25

2.5.2 Pattern Recognition

The characteristics and the rules associated with the data mining clusters are used to identify whether a particular truck in the WIM database could potentially be a permit. First, the axle weights and gross weight of the WIM truck are checked to verify whether they are within the legal limits as described in Section 2.4.1. If the OWVC is 00000, the truck is legal and there is no need to check whether it is a permit. Otherwise, the FHWA Class, the TAL and SPC codes for the truck are extracted using the process described in Sections 2.4.2 and 2.4.3. These FHWA, SPC and TAL codes are compared to the rules. If these codes fit into any of the previously established rules, we proceed to checking the axle weight and axle spacing characteristics. If all the truck's characteristics match all the characteristics for a particular divisible permit cluster, the vehicle is labeled as a "potential" divisible (DV*) permit. The star next to the DV group indicates the preliminary result of the pattern recognition analysis. If all the truck's characteristics match all the characteristics for a particular special hauling permit cluster, the vehicle is labeled as a "potential" special hauling (SH*) permit. If the vehicle is labeled as both a DV* and SH*, then the truck may potentially have been issued a permit but is an "unidentified" permit; the truck is labeled as (DS*). Finally, if the truck's characteristics do not match any single characteristic of the clusters, the vehicle is an illegal overweight truck and is labeled as illegal (IL*). The reverse is not necessarily true, i.e. if an overweight truck matches the characteristics it is not necessarily a permit because many illegal trucks may have characteristics that resemble those of permits.

This pattern recognition procedure is illustrated using the rules and the associated characteristics established in Tables 2.2 and 2.3 of Section 2.5.1. As an example, the two vehicles labeled Truck 5 and Truck 6 shown in Figure 2.8 are checked to verify whether they could potentially be permit trucks. Both trucks are overweight because their gross weights are greater than 80 kip. In fact, Truck 5 has a GVW equal to 85 kip, and Truck 6 has a GVW equal to 96 kip. These two trucks had been identified in the NYSDOT survey to be illegal overweight vehicles. The check begins by identifying the FHWA class, the SPC code and the TAL code of each truck. Both trucks are Class 9 vehicles and each has a total length between 50 and 60 ft; therefore, their TAL=3. Additionally, the spacing configurations of both trucks classify them as SPC=3242. Thus, these two trucks match the clustering rules of the divisible permits and special hauling permits defined in Tables 2.2 and 2.3. This means that their configurations are similar to those of permit trucks. Next, we check the axle weight and axle spacing characteristics of these trucks to the characteristics of the permit clusters. The axle spacings of Truck 5 and Truck 6 which are 13 ft, 5 ft, 33.5 ft and 5.1 ft are within the ranges of the axle spacings labeled SP1, SP2, SP3 and SP4 further confirming that these two trucks could potentially be permit. The gross weight of Truck 5 being 85 kip indicates that it is lower than the maximum allowed GVW in Tables 2.2 and 2.3 which confirms that it can be either a DV or SH permit. On the other hand, the gross weight of Truck 6 which is 96 kip is higher than the maximum GVW of the divisible permit cluster but is lower than the 100 kip maximum value of the special hauling cluster. This indicates that this truck could not be a divisible permit but could potentially be a special hauling permit. The individual axle weights of Truck 5 are all lower than the maximum allowed values shown in Tables 2.2 and 2.3. This leads to the conclusion that Truck 5 could potentially be a divisible permit and also could potentially be a special hauling permit. Thus, truck 5 will be labeled DS* even though it is in actuality an illegal truck. Finally, the axle weights of Truck 6 are compared to the maximum values given in Table 2.3. We observe that axle 2 weighs more than the allowable 23 kips shown in Table 2.3. This indicates that even though the configuration and the axle spacing of Truck 6 match the cluster of special hauling trucks of Table 2.3, the truck is identified as illegally overloaded because it exceeds one of the upper weight limits associated with the cluster. Because Truck 6 does not fit the DV cluster and because it violates the upper limit characteristics of the SH cluster, the vehicle is finally labeled as an illegal (IL*) vehicle.

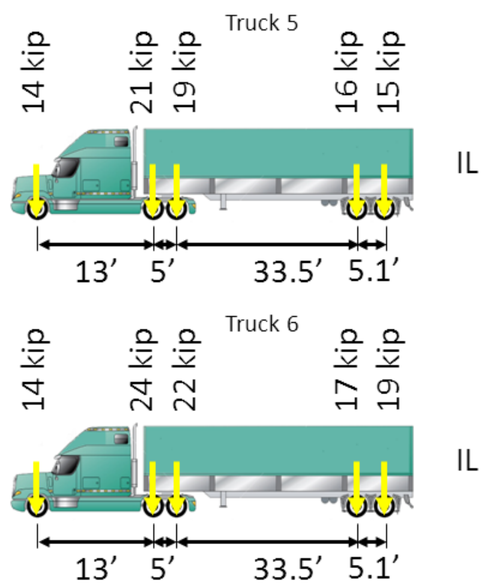

Figure 2.8 – Example Illegal (IL) Vehicles

It is noted that if a truck is identified as illegal it is certainly illegal. On the other hand, there is no assurance that a truck that is identified as a potential permit is actually in possession of a permit. Furthermore, if a truck matches the rules and the characteristics of both a divisible permit cluster and simultaneously those of a special hauling cluster, the truck could be either a divisible permit, a special hauling permit or possibly illegal. It is not possible for the pattern recognition algorithm by itself to identify which of the trucks labeled permits are actually permits and which ones are illegal that resemble permit trucks. However, a statistical Bayesian updating procedure can be used to obtain statistical estimates of the percentage of illegal trucks in a WIM database using previously available survey information as will be explained next.

2.5.3 Bayesian Updating

The pattern recognition procedure described in Section 2.5.2 classifies WIM vehicles into four preliminary categories DV*, SH*, IL* and DS*. The stars next to DV, SH, IL and DS indicate that the output from the pattern recognition represents a preliminary assessment for each truck in the WIM database. A statistical distribution of the trucks in each category can be assembled. Figure 2.9 gives a schematic representation of the possible sets. The striped domain in Figure 2.9 shows the set of vehicles classified as potentially permits of unknown type (DS*). Trucks in this set DS* can actually be divisible permits, special hauling permits or illegal. The percentage of trucks from the total population of WIM trucks have to be redistributed into the three other groups DV, SH, and IL. Because the pattern recognition algorithm provides a lower bound on the estimates of DV, SH and IL vehicles, the upper bound of the divisible permit vehicles is represented by the union of the DV* set plus the entire set of unknown vehicles DS*, (DV*+DS*). The upper bound of the special hauling permits is given by the union of the SH* set plus the set of the unknown vehicles DS*, (SH*+DS*), and finally the upper bound of the illegal group is represented by the union of the sets IL* and DS*, (IL*+DS*). A rescaling of the sets DV*, SH* and IL* can then be executed if the probability that a truck that has been labeled DS is actually a divisible permit (DV) expressed as $\Pr(DS/DV)$ is known. Also, the probability that a truck that has been identified as DS is actually a special hauling truck (SH), or $\Pr(DS/SH)$ must also be known. Finally, the probability that a truck that has been identified as DS is actually an illegal or $\Pr(DS/IL)$, also must be known. The probabilities $\Pr(DS/DV)$ and $\Pr(DS/SH)$ can obtained by running the pattern recognition algorithm on the DV and SH permit databases. The percentage $\Pr(DS/IL)$ can be either obtained from Eq. (2.3) or from a pattern recognition analysis on a representative sample of illegal vehicles collected from a field survey. The rescaling of Eq. (2.2) will give an improved estimate of the percentage of trucks in a WIM database that are divisible permits (DV), special hauling permits (SH), or illegal (IL).

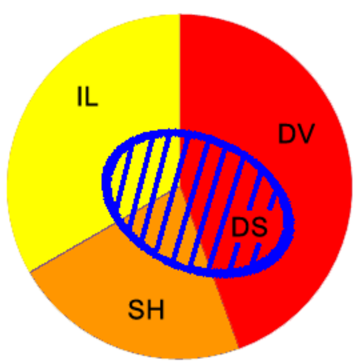

Figure 2.9 – Distribution of Trucks after Pattern Recognition into DV, SH, IL and DS Groups

$$\Pr(DV^{**}) = \frac{\Pr(DV^*)}{1-\Pr(DS \mid DV)}; \quad \Pr(SH^{**}) = \frac{\Pr(SH^*)}{1-\Pr(DS \mid SH)}; \quad \Pr(IL^{**}) = \frac{\Pr(IL^*)}{1-\Pr(DS \mid IL)} \tag{2.2}$$

With

$$\frac{\Pr(DV^*)}{1-\Pr(DS \mid DV)} + \frac{\Pr(SH^*)}{1-\Pr(DS \mid SH)} + \frac{\Pr(IL^*)}{1-\Pr(DS \mid IL)} = 1 \tag{2.3}$$

where: $\Pr(DV^*)$ = percentage of divisible trucks (DV) from the analysis

$\Pr(SH^*)$ = percentage of special hauling (SH) trucks from the analysis

$\Pr(IL^*)$ = percentage of special hauling (SH) trucks from the analysis

$\Pr(DS/DV)$ = probability that a truck labeled as combination (DS) is actually divisible (DV)

$\Pr(DS/SH)$ = probability that a truck labeled as combination (DS) is actually special hauling (SH)

$\Pr(DS/IL)$ = probability that a truck labeled as combination (DS) is actually an illegal (IL)

$\Pr(DV^{**})$ = percentage of divisible trucks (DV) after the rescaling

$\Pr(SH^{**})$ = percentage of special hauling (SH) trucks after the rescaling

$\Pr(IL^{**})$ = percentage of special hauling (SH) trucks after the rescaling

The adjustments of Eq. (2.2) serve to redistribute the trucks identified as DS* into the divisible, special hauling and illegal categories. However, as mentioned earlier, some illegal trucks may resemble divisible permit trucks while others may resemble special hauling trucks. Therefore, another adjustment of the percentages of trucks labeled DV** and SH** must be made to account for the probability that trucks in these categories may actually be illegal. This adjustment can be performed using Bayes updating rule following the formulation presented in Eq. (2.4) and (2.5) (Lapin, 1983; Ang and Tang,2006).

$$\Pr(IL/DV) = \frac{\Pr(DV/IL)\times\Pr(IL^{**})}{\Pr(DV/IL)\times\Pr(IL^{**}) + \Pr(DV/DV)\times\Pr(DV^{**}) + \Pr(DV/DS)\times\Pr(DS^{**})} \tag{2.4}$$

$$\Pr(IL/SH) = \frac{\Pr(SH/IL)\times\Pr(IL^{**})}{\Pr(SH/IL)\times\Pr(IL^{**}) + \Pr(SH/SH)\times\Pr(SH^{**}) + \Pr(SH/DS)\times\Pr(DS^{**})} \tag{2.5}$$

where: $\Pr(DS^{**})$ = percentage of trucks labeled as combination (DS) after rescaling, the term is zero

$\Pr(DV/DS)$ = proportion of vehicles labeled as combination (DS) to be divisible truck (DV)

$\Pr(SH/DS)$ = proportion of vehicles labeled as combination (DS) to be special hauling (SH)

$\Pr(IL/DV)$ = proportion of vehicles labeled as divisible (DV) to be illegal truck (IL)

$\Pr(IL/SH)$ = proportion of vehicles labeled as special hauling (SH) to be illegal truck (IL)

$\Pr(IL/DS)$ = probability that a truck that is illegal (IL) has been labeled as a combination (DS)

2.5.4 Summary of Data Mining Algorithm

The overall data mining procedure is implemented into a computer algorithm and the process can be summarized as follows:

1. Analyze the NYSDOT permit databases for divisible (DV) and special hauling (SH) trucks to build the pattern recognition rules that cluster the overweight permits into groups based on their axle spacing code (SPC), FHWA classes, and truck lengths (TAL).
2. For each cluster obtain the characteristics of the cluster which consist of the upper and lower bounds for the axle spacings, and the upper bound of the allowed gross weights and axle weights.
3. Use WIM systems to assemble a truck database representative of the trucks that travel on a given highway.
4. Use the algorithm presented in Section 2.4.1 to identify the overweight category (OWVC) and FHWA class of each truck in the WIM database. Also, use the algorithm described in Section 2.4.3 to assign the axle spacing code (SPC).
5. If a given truck's OWVC is 00000, then the truck is within the legal limits and no further investigation is necessary.
6. If a given truck's OVWC is different than 00000, check to see whether the combination of its SPC, FHWA and TAL codes match any of the clusters identified in step 1. If no match is found, the truck is labeled illegal (IL*) and no further investigation is necessary. Otherwise go to step 7.
7. Compare all axle spacing values with the minimum and maximum values associated with the cluster identified under step 6. Also, compare the gross vehicle weight and the axle weights with those of the identified cluster.
8. If an overweight truck from the WIM database matches any one of the DV permit cluster characteristics, it is labeled as "potentially DV*".
9. If an overweight truck from the WIM database matches any one of the SH permit cluster characteristics, it is labeled as "potentially SH*".
10. If an overweight truck from the WIM database matches any one of the SH or DV permit cluster characteristics, the truck is classified as a potential permit of unknown type and labeled (DS*).
11. If the overweight vehicle matches a cluster but exceeds the characteristic limits of the identified cluster, it is labeled as illegal (IL*).
12. Once a preliminary estimate of the percentages of trucks in a WIM data are classified as DV*, SH*, DS* and IL*, a Bayesian statistical adjustment of the percentages should be performed as described in Section 2.5.3.

The data mining algorithm is implemented using WIM data collected at the I-90 WIM site in upstate New York using the rules established based on the divisible and special hauling permit databases provided by the NYSDOT as will be described next.

2.6 Implementation and Validation

2.6.1 Implementation of Clustering Procedure

To illustrate the procedure, clustering rules for permit trucks were established using the databases for divisible and special hauling permits for the year 2009 which were provided by NYSDOT. The divisible permit database contains 453,434 vehicles and the special hauling database consists of 25,749 vehicles. Some of the divisible permits were issued for truck size while 446,427 permits were issued for overweight. All 25,749 special hauling permits are for overweight. Generally, the divisible permits are valid for one year. Occasionally, some of these vehicles have to be checked by the NYSDOT office of structures to verify that they will not jeopardize the safety of bridges along a specific route. The special hauling permit database contains vehicles that are generally issued for a single trip, but also contains permits issued on an annual basis for a limited radius around a specific location. The special hauling database also contains permits checked by the NYSDOT office of structures.

The distribution of the divisible and special hauling vehicles into FHWA classes is shown in Figure 2.10. The highest percentage of divisible vehicles fall into Class 9 that represents about 46% of the total population, while the highest percentage of special hauling vehicles is represented by Class 10 that represents about 36% of the total number of trucks. The second highest percentage for the divisible permits is represented by FHWA Class 10 with about 38% of the vehicles. The second highest percentage of special hauling trucks is found in Class 9 with about 32% of the entire population. Classes 9 and 10 alone represent 84% and 68% of the entire population of divisible and special hauling permits respectively. It is observed that both databases show a significant number of trucks in Classes 13 and 14. The trucks classified into these classes may not necessarily be double trailers, as NYSDOT does not normally issue permits for double trailers, but may have axle spacings similar to those of double trailers.

Figure 2.10 also shows the distribution of the gross vehicle weights (GVW) of the divisible and special hauling trucks in the databases. The divisible permit GVW distribution shows a high concentration of vehicles with a gross weight of 100,000 lb and a small variability, indicating that almost all Class 9 and 10 vehicles are issued for gross weights of about 100,000 lb. A different scenario is depicted for the special hauling permit vehicles. For these types of permits more than one peak is found for different gross weights and the dispersion is much higher compared to the divisible permit vehicles.

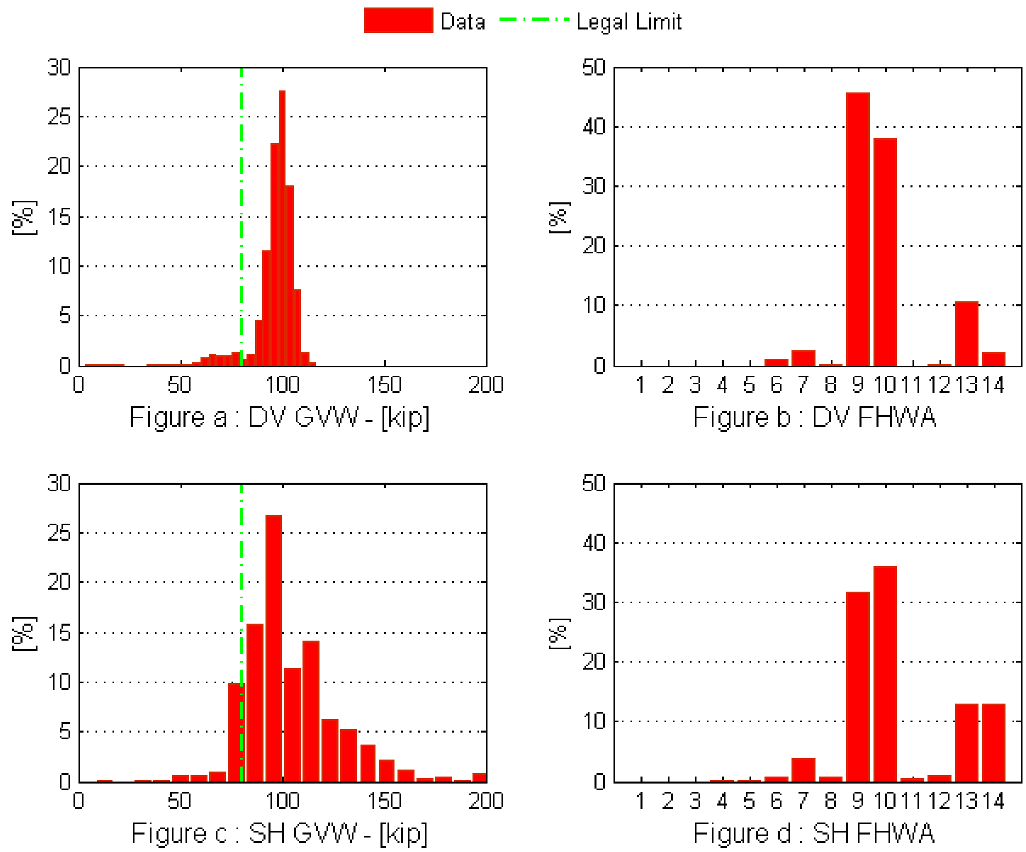

Figure 2.10 – Gross Vehicle Weight (GVW) and FHWA Class Distribution of the 2009 NYSDOT DV and SH Databases

The two NYSDOT permit databases are processed to create clusters for the rules used in the data mining algorithm as explained in Section 2.5. The analysis of the divisible database led to fourteen cells, one for each FHWA Class including Class 14 which contains trucks that did not meet any of the regular FHWA Class configurations. The first four cells are empty because these types of vehicles are not issued permits. Cell 5 that corresponds to FHWA Class 5 contains 2 clusters. Each cluster contains specific TAL, SPC, and axle characteristics. Cell 6 contains 4 clusters. Cells 7, 8, 9 and 10 contain 46, 20, 56, and 100 clusters respectively. Cell 11 is empty because no vehicles in this FHWA class were issued divisible permits. Cells 12, 13, and 14 contain 4, 200, and 226 clusters respectively. A high number of clusters indicates that the vehicles in the specific cell are very heterogeneous in terms of their SPC and TAL codes. The analysis of the special hauling database also produced fourteen cells. Cell 5 contains 36 clusters, while Cells 6, 7, 8, 9 and 10 contain 4, 72, 56, 108, and 90 clusters. Cells 11, 12, 13, and 14 contain 36, 102, 462, and 816 clusters of rules. It is noted that the special hauling database contains a higher number of clusters per FHWA class than the divisible database because of the larger variation in the configurations and truck lengths. This dense matrix of clusters and cells is used in the following set of analyses to assemble the conditional probabilities needed for the Bayesian updating steps and to implement the data mining algorithm.

2.6.2 Establishing Bayesian Conditional Probabilities

As a first test of the methodology, the two NYSDOT permit databases of divisible and special hauling trucks are run through the pattern recognition algorithm. One objective of this test is to check that the program will correctly classify all known permit trucks. A second objective is to assemble the conditional probabilities needed to perform the Bayesian updating described in Section 2.5.3. Running the pattern recognition algorithm and the clustering system adopted in Sections 2.5.1 and 2.5.2 confirmed that all the overweight permit trucks in the databases were correctly identified as permits and none of them was labeled illegal. The analysis provides the distribution of vehicles classified as divisible permit, special hauling permit or unknown permit per each FHWA Class. Table 2.4 shows the distribution for the analyzed database of divisible permits. For example, Class 9 is composed of a total number of 201,913 overweight vehicles. Out of these, the pattern recognition rules correctly recognized 8,097 (4.0%) as divisible permits and the remaining 193,816 (96.0%) as unknown permits. Overall, the pattern recognition algorithm correctly classified 114,308 (25.6%) out of the 446,427 divisible vehicles. The remaining 332,119 (74.4%) were classified as unknown permits. These percentages are listed in Table 2.6. These results are also depicted in Figure 2.11, where Figure 2.11.a and Figure 2.11.b show the distribution of the GVW and the FHWA classes for the divisible overweight permit vehicles that were recognized as divisible. Figures 2.11.c and 2.11.d show the histograms for the divisible permits that were classified as unknown permits.

Similarly, Table 2.5 shows the distribution of trucks in the special hauling database. Overall the pattern recognition algorithm correctly classified 10,012 (38.9%) out of the 25,746 special hauling vehicles, while the remaining 15,734 special hauling vehicles were classified as unknown permits. These results are plotted in the histograms of Figure 2.12. Figures 2.12.a and 2.12.b show the distribution of the GVW and the FHWA classes for the special hauling permits that were recognized as special hauling. These results are summarized in Table 2.6. Figures 2.12.c and 2.12.d show the histograms for the special hauling permits that were classified as unknown permits.

Looking at Figures 2.11.d and 2.12.d in conjunction with Figures 2.11.c and 2.12.c, it is clear that the trucks classified as unknown permits are those that have configurations which mostly correspond to Class 9 and 10 trucks and have common ranges of gross weights. This is due to the large number of trucks in the divisible and special hauling databases that have similar axle spacing and axle weight characteristics.

Additionally, because many illegal trucks may have weights and configurations similar to those of permit trucks, it is expected that the final algorithm will misclassify many illegal trucks into permit truck categories. In order to estimate the percentage of illegal trucks that could be wrongly categorized as permit trucks, a database of illegal trucks provided by NYSDOT was assembled. The sample contains 269 illegal vehicles checked by NYSDOT personnel. This database was run through the pattern recognition algorithm to find that the vast majority or 95.2% of the 269 illegal vehicles have been classified as unknown permits while only 1.1% of the trucks were classified as divisible permits and 0.7% as special hauling. The remaining 3.0% were properly identified as illegal.

The percentages of divisible vehicles classified as unknown permit as well as the percentages of special hauling vehicles classified as unknown as listed in Table 2.6 will be used as input for the Bayesian updating procedure described in Section 2.5.3.

For the illegal trucks, the conditional probabilities are not divided into classes because of the sample size of the illegal truck database that is available at this stage. Therefore, the distribution obtained from the sum of the illegal trucks is assumed to be valid for each class.

Table 2.4 – Pattern Recognition Distribution of the Divisible Permit Database per FHWA Class

(1)	DIVISIBLE DATABASE										
	C-5 (2)	C-6 (3)	C-7 (4)	C-8 (5)	C-9 (6)	C-10 (7)	C-11 (8)	C-12 (9)	C-13 (10)	C-14 (11)	TOT (12)
DV	-	23	1120	143	8097	63137	-	12	34652	7124	114308
SH	-	-	-	-	-	-	-	-	-	-	-
DS	-	4982	9721	60	193816	108221	-	-	13441	1878	332119
IL	-	-	-	-	-	-	-	-	-	-	-

Table 2.5 – Pattern Recognition Distribution of the Special Hauling Permit Database per FHWA Class

(1)	SPECIAL HAULING DATABASE										
	C-5 (2)	C-6 (3)	C-7 (4)	C-8 (5)	C-9 (6)	C-10 (7)	C-11 (8)	C-12 (9)	C-13 (10)	C-14 (11)	TOT (12)
DV	-	-	-	-	-	-	-	-	-	-	-
SH	4	-	646	141	1711	1225	144	254	2655	3232	10012
DS	17	172	346	18	6466	7994	-	2	643	76	15734
IL	-	-	-	-	-	-	-	-	-	-	-

Table 2.6 – Conditional Probabilities

(1)	SAMPLE		
	IL (2)	DV (3)	SH (4)
DV	1.1%	25.6%	-
SH	0.7%	-	38.9%
DS	95.2%	74.4%	61.1%
IL	3.0%	-	-

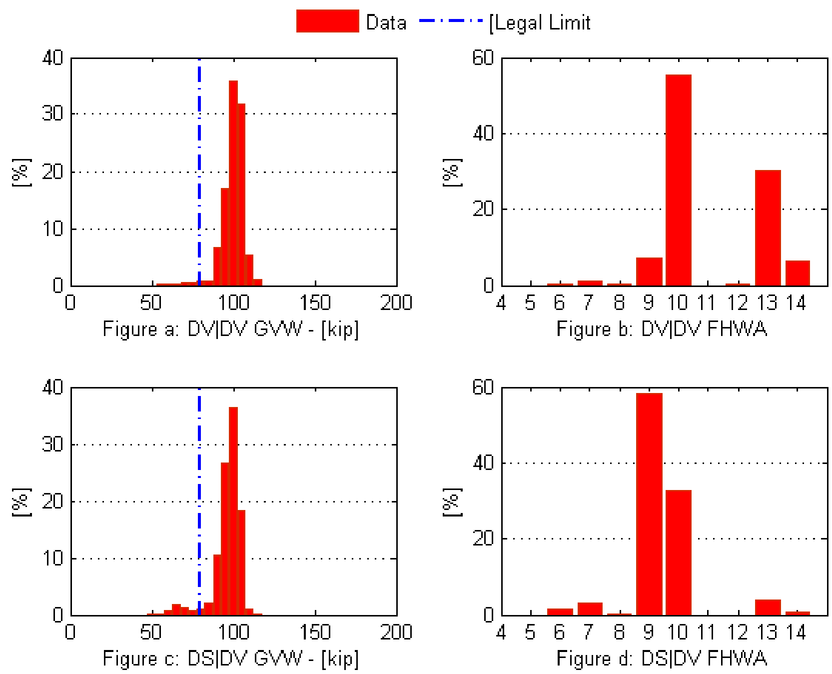

Figure 2.11 – Gross Vehicle Weight (GVW) and FHWA Class Distribution of DV vehicles after Pattern Recognition

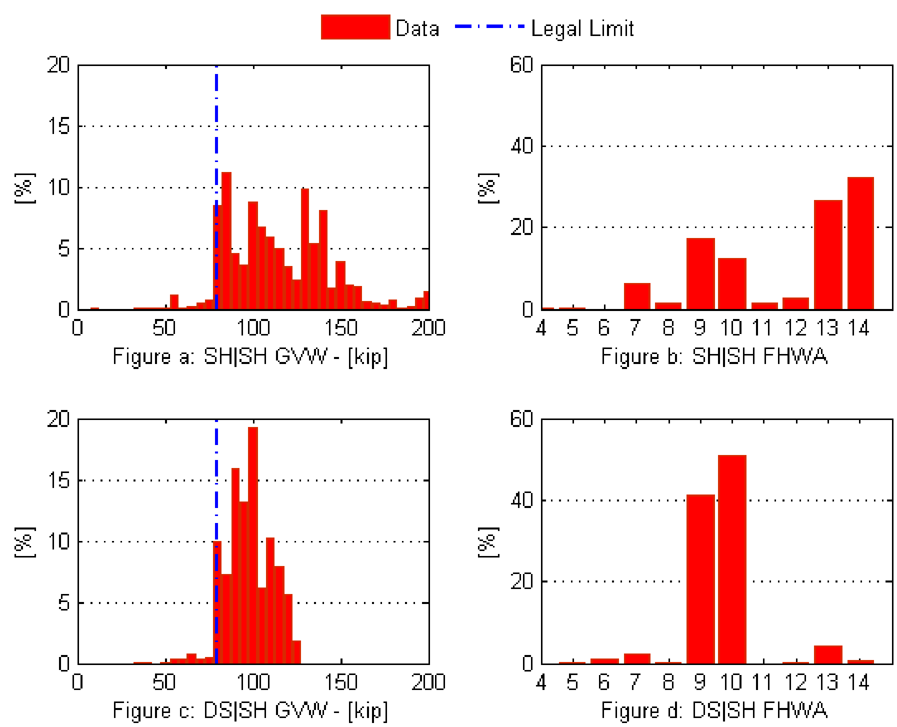

Figure 2.12 – Gross Vehicle Weight (GVW) and FHWA Class Distribution of SH vehicles after Pattern Recognition

2.6.3 Validation of Data Mining Algorithm

To verify the validity of the proposed approach, the data mining algorithm including the pattern recognition and the Bayesian updating procedures are implemented on a WIM database collected at the I-90 WIM Schodack site in upstate New York. The clustering rules as well as the conditional Bayesian probabilities used during the search were those obtained from the analysis of permit databases as described in Sections 2.6.1 and 2.6.2. The data from the I-90 site were used to compare the output of the data mining algorithm to the results of a survey of trucks conducted by the New York State Department of Transportation (NYSDOT) in 2009. The application of the data mining algorithm developed in this study on the WIM database consisting of 175,432 trucks returned that 15,544 (8.9%) vehicles are overweight by exceeding at least one of the axle, group of axle, or gross weight legal limits. Specifically, 5,673 (3.2%) are overweight with a gross weight exceeding 80 kip. The 3.2% is found to compare reasonably well with the results of the NYSDOT survey that found that about 4.9% of commercial trucks traveling over that site are overweight. Some difference is expected because the survey was performed over a much limited period of time than the length of time used to collect the WIM database.

More specifically, the NYSDOT survey checked 989 overweight vehicles that exceeded the gross weight limit of 80,000 lb and found that 1.2% of these held special hauling permits, 49.2% were divisible permits and 49.6% were illegal. These results are summarized in column (6) of Table 2.7. The data mining algorithm applied on the 5,673 overweight vehicles that violated the GVW limit identified 443 out of 5,673 (7.8%) vehicles as potentially divisible permit (DV*), 109 (1.9%) vehicles as special hauling permit (SH*), 3,927 (69.2%) vehicles as unknown permit (DS*), and 1,194 (21.1%) vehicles as illegals (IL*). These preliminary estimates obtained from the pattern recognition procedure are summarized in columns (2) and (3) of Table 2.7. As mentioned above, the category of unknown permits (DS*) consists of trucks that can actually be either divisible permits, special hauling, or possibly illegal. Therefore, the trucks in the DS* category must be redistributed into the other three categories using the Bayesian updating approach given in Eq. (2.2), (2.3) and (2.4). The input data for the probabilities used in the Bayesian updating process are those obtained from the analysis of the permit databases as described in Section 2.6.2 and summarized in Table 2.6. The prior estimates as obtained from the U.S. census and the New York State permit databases are Pr(DV)=70.9% Pr(SH)=4.1%, Pr(IL)=25% on the entire population of trucks. The adjusted percentages after the updating of the unknown permits are provided in column (4) of Table 2.7.

Table 2.7 – Comparison of Results

Overweight Type (1)	**No. of Trucks (2)**	**Preliminary Percentage (3)**	**Bayesian Updating of Unknown Trucks (4)**	**Bayesian Updating of Illegal Permits (5)**	**NYSDOT Overweight Survey (6)**
Divisible (DV)	443	7.8%	53.8%	**52.0%**	**49.2%**
Special Hauling (SH)	109	1.9%	4.1%	**3.4%**	**1.2%**
Unknown Permit (DS)	3,927	69.2%	0%	**-**	**-**
Illegal (IL)	1,194	21.1%	42.1%	**44.6%**	**49.6%**

Because some illegal vehicles resemble configurations of both divisible and special hauling permit, parts of the vehicles classified as divisible or special hauling are in reality illegal. For this reason, the second Bayesian updating procedure of Eq. (2.8) and (2.9) is performed on the rescaled results of column (4) of Table 2.7. The conditional probabilities used as input are those listed in Table 2.5 as obtained using the pattern recognition procedure applied on the permit and illegal databases. The output obtained from Eq. (2.8) and (2.9) are $\Pr(IL/DV)$ =3.3% and $\Pr(IL/SH)$ =16.5%. This means that 3.3% of the trucks that have been so far classified as divisible may be actually illegal. Also, the Bayesian updating indicates that 16.5% of the trucks that have been so far classified as special hauling may be actually illegal. The final result after Bayesian updating is shown in the penultimate column (5) of Table 2.7. These values are compared to those of the NYSDOT survey which gave a distribution of 49.2% for divisible, 1.2% for special hauling permits and 49.6% for the illegals as shown in column (6) of Table 2.7. The comparison shows a relatively good agreement with a maximum under-prediction in the percentage of illegal trucks by about 5% which is deemed acceptable given that the population sample used during the NYSDOT survey is relatively small and collected over a limited period of time, while the WIM database analyzed involves an entire year of data.

2.6.4 Implementation to Entire I-90 WIM Data

The same analysis performed on the Schodack I-90 WIM sample containing the overweight vehicles exceeding the gross weight of 80 kip described in the previous section was done for the entire I-90 WIM database consisting 175,432 trucks. Out of these, a total of 15,544 vehicles are found to be overweight exceeding any of the legal limits defined in Section 2.4.1. The total number of overweight trucks is more than 2.7 times the 5,673 trucks that exceed the 80 kip GVW limit because in this example we are accounting for trucks that exceed the FBF formula and trucks that have axle weights, tandem and tridem weights that exceed the legal limits for these axle units. The results of the pattern recognition process are provided in Table 2.8 which shows that the number of vehicles labeled as potentially divisible (DV*) is equal to 532. The vehicles in the special hauling (SH*), unknown permit (DS*), and illegal (IL*) categories are respectively 492, 13,058 and 1,462. These results are summarized in column (2) of Table 2.8, and the corresponding percentages are listed in column (3). The redistribution of the unknown permits (DS*) into DV**, SH**, and IL** obtained using Eq. (2.2) through (2.7) is shown in column (4) of Table 2.8, using the conditional probabilities of Table 2.6. Finally, the results after applying the Bayesian updating steps of Eq. (2.8) and (2.9), which shift some of the divisible and special hauling vehicles into the illegal truck category, are reported in column (5) of Table 8. The results in column (5) of Table 2.8 for the entire overweight truck population are compared to those in Table 2.7 which was obtained for trucks over 80,000 lb. A slight difference is noted in the estimate of the special hauling permits and a corresponding reduction in the percentage of the illegals when the entire I-90 WIM population is analyzed. The percentage of the divisible permits increases from 52.0% to 57.9%.

Table 2.8 – One year WIM Data Analysis Results

Overweight Type (1)	No. of trucks (2)	Preliminary Percentage (3)	Bayes updating of unknown trucks (4)	Bayes Updating of illegal permits (5)
Divisible (DV)	532	3.4%	59.4%	**57.9%**
Special Hauling (SH)	492	3.2%	5.8%	**5.3%**
Unknown Permit (DS)	13,058	84.0%	0%	-
Illegal (IL)	1,462	9.4%	34.7%	**36.8%**

The results in Table 2.8 are the summations for all the truck classes. The results per each class are presented in the pie charts of Figure 2.13. The Bayesian updating is performed using the statistics reported for each FHWA class listed in Tables 2.5 and 2.6 for the divisible and special hauling vehicles. Each pie chart shows the distribution of overweight trucks into divisible permit (red), special hauling permit (orange), and illegal vehicles (yellow) for each class. In addition, the label below the pie chart gives the percentage of the WIM trucks in each class (TR), and the percentage of overweight vehicles (OW) per each class. For example, the total number of WIM trucks that can be classified as Class 5 is 9.7% x 175,432 = 17017 trucks and the number of Class 5 trucks that are overweight is 351 trucks. These 351 trucks are divided into 51% x 351 = 182 trucks that are divisible permits and 49% x 351 = 169 trucks that are illegal.

The results of the analysis show that for Classes 5, 7, 9, 10 and 13, the percentage of divisible vehicles (DV) is greater than 50%, and more specifically 74% of the overweight Class 7 trucks are divisible permits. Similarly, about 65% of overweight Class 9 and 10 trucks are divisible permits. Similarly, 57% of overweight Class 13 trucks and 51% of those in Class 5 are divisible permits. For this sample, the classes with the highest percentage of special hauling vehicles are those in Class 8 and 13, with respectively 36% and 13% of the overweight trucks. Also, Class 8 shows the highest percentage of illegal vehicles. Class 14 as explained earlier is an additional class generated to account for vehicles having configurations that cannot be classified into any of the FHWA Classes 5 through 13. The highest percentage of illegal trucks is for the overweight trucks in this Class 14. Also, it is noted that NYSDOT does not allow trucks with double trailers on their highway system. Therefore, many of the trucks classified in Class 13 are probably trucks that have axle configurations that resemble those of double trailers while others may be operating illegally.

The classification of overweight trucks into different permit and illegal truck categories and their division into different classes is useful for implementation in cost allocation studies and the results can be used for estimating the contribution of different truck categories to highway pavement and bridge damage.

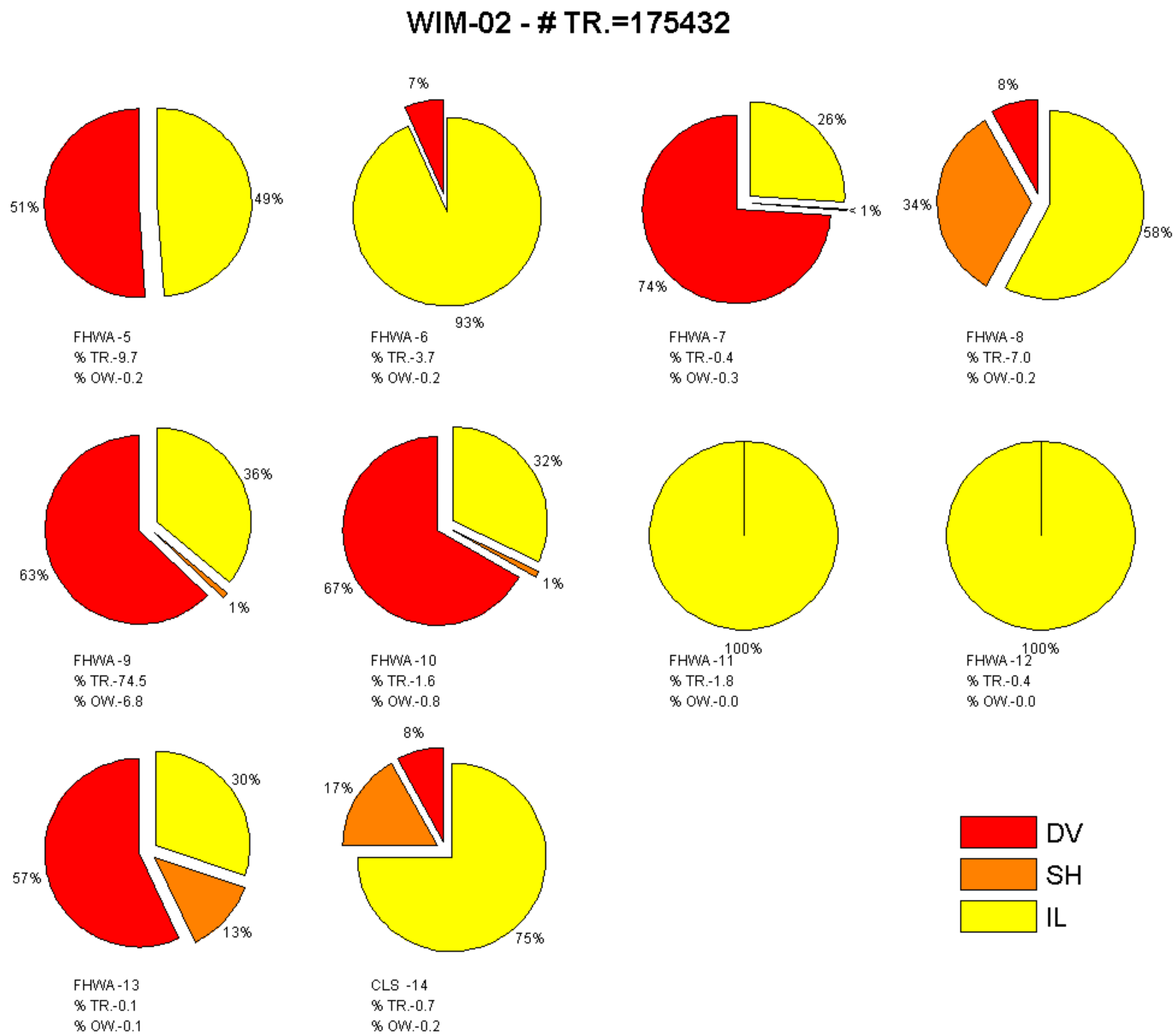

Figure 2.13 – Distribution of Overweight Trucks per Permit Type and Class

2.7 Sensitivity Analysis

WIM data are affected by many sources of errors. These include the inherent error in the WIM data collection system, WIM station calibration, variation of humidity and temperature, effect of vehicle velocity, truck impact on pavement, and other parameters (Sivakumar et al, 2011). The percent error for an individual truck WIM measurement is defined in terms of the ratio of the WIM output to the static scale measurement for the same vehicle. Different WIM technologies may produce different measurement errors. For example, bending plates have generally an overall accuracy of +/- 12%, while load cell WIM systems may have errors on the order of +/-6%. Ceramic piezoelectric WIM systems have an overall accuracy that could vary between +/-3% and +/- 30% on the axle loads (CTRE, 2007; Papaggianakis et al, 2008). Data obtained from a calibration of a WIM system in New York shows a standard deviation of the error on a single axle on the order of 10% (Sivakumar et al, 2011). Table 2.9 summarizes the level of accuracy of the most common WIM systems in use in the U.S. collected from different sources including CTRE (2007).

Table 2.9 – WIM System Accuracy on Axle Weights

Technology	Accuracy
Bending Plate	12%
Capacitive Mat	1.5%
Load Cell	6%
Piezoelectric	3 to 30%
Quartz cables	10%
Bridge-WIM	10%

To study the effect of WIM system errors on the results of the permit classifications obtained from the application of the proposed data mining algorithm, a sensitivity analysis is performed by introducing a random error on each truck in the WIM database. The random error is assumed to have a mean value equal to 1.0 which is indicative of a calibrated unbiased WIM system. The error is also assumed to have a standard deviation which represents the particular system's general accuracy. Standard errors varying between 5%, 10%, 15% or 20% are assumed on each vehicle's axle spacing to study the effect of misreading these spacings. A second set of sensitivity analyses are performed by assuming that the WIM system has a random standard error of 5%, 10%, 15% or 20% in determining the vehicles axle weights. The results for the different simulations obtained after the pattern recognition process are shown in Tables 2.10 and Tables 2.11.

2.71. Effect of WIM Errors in Axle Spacing

The results shown in Table 2.10 demonstrate the following:

- Errors in the axle spacings of trucks in the WIM database produce variations in the number of overweight trucks and also variations in the percentages in each permit category.
- Specifically, when the standard WIM error is on the order of 5% on the axle spacing, the percentages of DV, SH and IL are changed from 3.4%, 3.2%, and 9.4% to 3.6%, 3.3% and 9.7%. When the error increases to 10%, the percentages of DV, SH and IL are changed to 3.8%, 5.5% and 10.4%. An error of 20% on the axle spacing will lead to even higher changes in the percentages.
- Although the distribution into the different categories remains somewhat stable for errors up to 10%, the number of overweight trucks increases from 8.9% to 17.7% which is high.
- According to the observations made above, it is clear that a well calibrated WIM system must produce errors with a standard deviation less than 10% on the axle spacing in order to produce an accurate distribution of trucks into the Divisible, Special Hauling and Illegal categories.

Table 2.10 – Sensitivity Analysis – Axle Spacing

TEST No.	1	2	3	4	5
STD	0%	5%	10%	15%	20%
No. Trucks	175, 432	175, 432	175, 432	175, 432	175, 432
No. Legal	91.1%	91.1%	82.3%	73.4%	64.6%
No. OW	8.9%	8.9%	17.7%	26.6%	35.4%
DV	**3.4%**	**3.6%**	**3.8%**	**4.0%**	**4.2%**
SH	**3.2%**	**3.3%**	**5.5%**	**9.4%**	**13.1%**
DS	**84.0%**	**83.4%**	**80.3%**	**74.8%**	**68.7%**
IL	**9.4%**	**9.7%**	**10.4%**	**11.8%**	**14.0%**

2.7.2 Effect of WIM Errors in Axle Weights

The results shown in Tables 2.11 demonstrate the following:

- Large errors in the axle weights of the WIM database produce a significant variation in the number of overweight trucks but only a slight variation in the percentages in each permit category.
- When an error on the order of 10% is applied on the axle weights of the trucks in the WIM database, the percentages of overweight trucks into each category do not change significantly even for the distribution of the trucks among permits and illegals, but the number of overweight vehicles is doubled, in fact it increases from 8.9% to 18.7% which is high.
- According to the observations made above. It is clear that a well calibrated WIM system must produce errors with a standard deviation less than 10% on the axle weights in order to produce an accurate distribution of trucks into the Divisible, Special Hauling and Illegal categories. As shown in Table 2.9 such range of errors should be within the margin of most existing WIM systems.

Table 2.11 – Sensitivity Analysis – Axle Weight

TEST No.	1	2	3	4	5
STD	0%	5%	10%	15%	20%
No. Trucks	175, 432	175, 432	175, 432	175, 432	175, 432
No. Legal	91.1%	91.3%	81.3%	69.8%	57.6%
No. OW	8.9%	8.7%	18.7%	30.2%	42.4%
DV	**3.4%**	**3.6%**	**3.6%**	**3.9%**	**4.4%**
SH	**3.2%**	**3.1%**	**3.2%**	**3.2%**	**3.3%**
DS	**84.0%**	**83.8%**	**83.9%**	**83.7%**	**83.0%**
IL	**9.4%**	**9.5%**	**9.3%**	**9.2%**	**9.3%**

2.8 Conclusions

This Chapter describes a procedure to classify trucks in a WIM database according to their legal status into different categories of overweight permits and illegally overweight trucks. The procedure is based on a data mining algorithm that uses a dense matrix of clusters that is calibrated based on a database of permit trucks.

The data mining algorithm consists of two procedures. The pattern recognition procedure helps identify likely permit trucks by comparing the axle configuration of each truck in the WIM database to clusters of trucks extracted from a permit database. The pattern recognition procedure is supplemented with a Bayesian updating method to further improve the results by statistically accounting for trucks that may be illegal but have configurations that are similar to those of permit trucks.

The validity of the algorithm was verified by comparing the results to those obtained from a survey conducted by NYSDOT using a WIM system equipped with a camera. The NYSDOT has checked manually the data to obtain percentages of trucks that are overweight and the percentage of those that have been issued permits. The NYSDOT survey results show similar percentages of permit and illegal trucks to those obtained by the data mining algorithm.

The accuracy of the WIM data plays an important role in the reliability of the procedure. It has been determined that WIM systems that produce a standard error less than 10% on both the axle spacing and the axle weights can lead to reasonably accurate estimation of the percentages of illegal and permit trucks in the WIM database.

The classification of overweight trucks into different permit and illegal truck categories is useful for implementation in cost allocation studies and the results can be used for estimating the contribution of different truck categories to highway pavement and bridge damage.

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Chapter 3

Modeling the Effects of Overweight Trucks

on New York State Bridges

3.1 Introduction

This Chapter describes a procedure to quantify the increased cost to bridges caused by overweight trucks. The methodology is demonstrated by first implementing it on a representative sample of twenty two bridges along the I-88 corridor between Binghamton and Schenectady in New York State and then applied to estimate the cost due to the New York state bridge network. The procedure is divided into three phases. In the first phase, the Weigh-In-Motion (WIM) data file which represents the truck traffic on the bridges is analyzed following the method described in Chapter 2 to estimate the percentages of legal trucks (LG) and overweight vehicles, which are divided in the three categories Divisible permits (DV), Special Hauling permits (SH) and Illegally overweight (IL). In the second phase, the maximum moment response of each vehicle contained in the WIM file is obtained for each representative bridge by sending each truck through the appropriate influence line. Finally, the truck's response is used to estimate the cost effect caused by each truck. The analysis of the effect of overweight vehicles is executed for two types of bridge responses: 1) overstress of main bridge members, and 2) cyclic fatigue accumulation for main members and decks. The overstress cost analysis used in this Report follows the classical FHWA cost allocation model. The fatigue damage analysis follows the AASHTO LRFD fatigue analysis method. The two different types of effects are divided into three categories: a) effect caused by divisible permit trucks, b) effect caused by special hauling permits and 3) effect caused by illegal overweight trucks and the associated costs are estimated based on those of bridge material and construction.

The remaining sections of this Chapter consist of the following: 3.2 Procedure for Evaluating Cost of overweight trucks; 3.2 Truck and Bridge Data Classification; 3.3 Structural Analysis; 3.4 Effect of Heavy Trucks on Bridges; 3.5 Bridge Cost Models, 3.6 Bridge Cost Allocation for Example Corridor; 3.7 Cost Allocation for NY State Bridge Network, and 3.8 Conclusions.

3.2 Procedure for Evaluating Cost Effect of Overweight Vehicles

3.2.1 Overview

This section presents a general overview of the procedure for evaluating the effect of overweight trucks to bridges. The procedure consists of four phases as shown in the flowchart of Figure 3.1. The first phase, "Data Analysis" consists of collecting the raw WIM data and classifying the Overweight Trucks into appropriate categories. The methodology follows the procedure described in Chapter 2 of this Final

Report to provide the percentages of legal and overweight vehicles for each class of vehicles. The overweight trucks are divided based on the likelihood of having been issued divisible permits, special hauling permits or being illegally overweight.

In the “Structural Analysis” phase, pertinent data for each bridge selected for analysis are collected from the National Bridge Inventory (NBI) files, “WINBOLTS” database, and detailed bridge plans when available. “WINBOLTS” is the bridge database assembled by NYSDOT for all the bridges under its jurisdiction. “WINBOLTS” contains more detailed information on New York bridges than provided in the NBI files. The information extracted from “WINBOLTS” is used to obtain the influence lines for each bridge’s most critical sections. These critical sections are those that give the maximum positive moment for each span and the most negative moment of the beam over each continuous support. Details of the influence line analysis program developed for the purposes of this study are described in the Report of Task 1.b. Each truck in the WIM database is sent through the influence lines of each bridge and the maximum positive bending moment, most negative moment, as well as the maximum moment range (maximum positive moment minus most negative moment) are calculated. Additionally, the responses of the AASHTO HS-20 load are obtained as well as those of the LRFD fatigue truck. The AASHTO HS-20 truck is used as the baseline for comparison because most existing New York State bridges were designed to meet the AASHTO standard design specifications with the HS-20 load model and also because the HS-20 load is the base line for the determination of the truck legal load limits stipulated in the Federal Bridge Formula (FBF).

In the third phase, “Damage Analysis”, the overstress effects and the fatigue effects for each truck and each bridge’s main longitudinal member are calculated and compared to the effects from the HS-20 load and the LRFD fatigue truck. Similarly, the fatigue effect to the deck of each bridge is calculated. The longitudinal member fatigue analysis follows the AASHTO LRFD approach, while the deck fatigue analysis follows the approach proposed by Perdikaris et al. (1993) which was also followed in NCHRP 495.

In the fourth and final phase, the effects produced by each overweight truck in the WIM database are used to estimate the costs of each bridge due to these trucks. For the overstress effect, the procedure follows the FHWA cost allocation methodology. For the fatigue effect, the AASHTO LRFD approach is followed. The cost of bridges used in this phase is based on the cost models presented in Report Task 1.b.

The following sections provide more details on the procedures followed in each phase. The cost analysis process is illustrated using as an example the cost of overweight trucks on 22 typical bridges located along the I-88 Corridor. The process is subsequently implemented on a representative sample of New York state bridges to estimate the cost of overweight trucks on the entire bridge network.

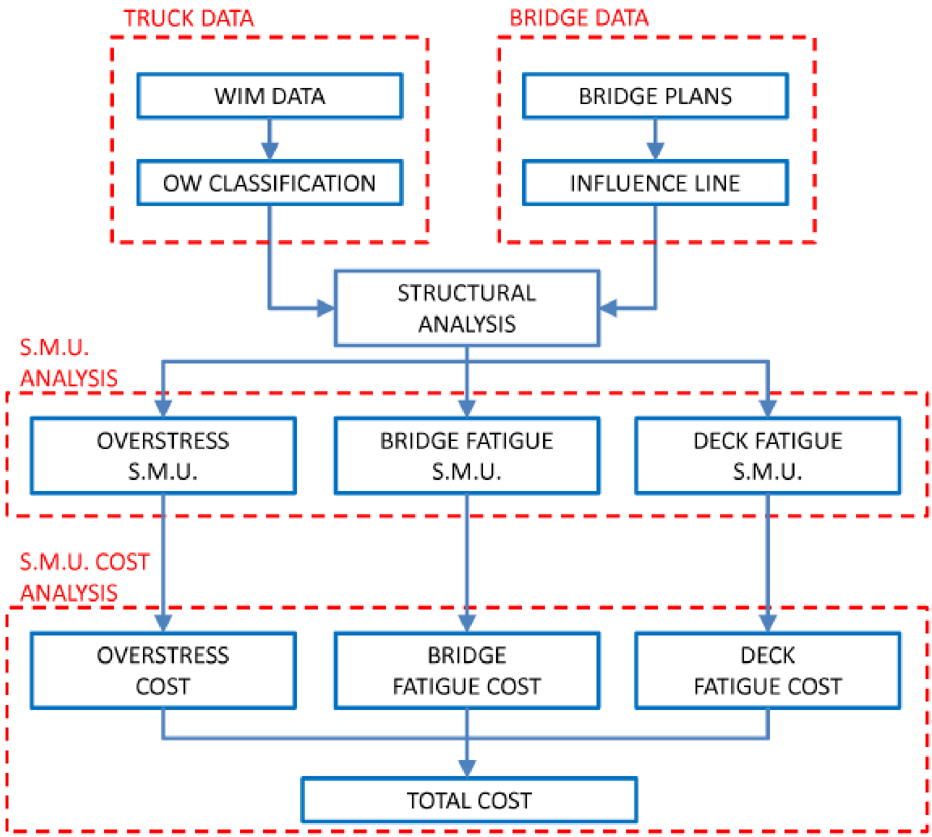

Figure 3.1 – Bridge Overweight Effect Quantification Procedure Flowchart

3.2.2 Truck Data Classification

The analysis of the WIM data is performed according to the program developed to implement the concepts outlined in Chapter 2. The program performs a statistical analysis of the overweight trucks to estimate the percentages of trucks that may have been issued divisible load permits (DV), those that may have been issued special hauling permits (SH) permits, and of the illegal overweight trucks (IL). The analysis is performed for each set of truck classes.

For example, a set of WIM data obtained for the I-88 Corridor contains 567,605 vehicles collected in the year 2011. Of these, 561,430 trucks (98.9% of the total) have axle configurations that matched the adjusted FHWA Classes 5 through 14. The adjustments were made to allow for the classification using the information available in the WIM data files rather than the exact FHWA classification which requires visual information on the truck configuration. Because of the adjustments to the FHWA classes the classification is referred to as NYSDOT classes. The permit analysis conducted on the set of 561,430 vehicles provided the values that are shown in Table 3.1. Column (2) of Table 3.1 gives the total number of trucks in each NYSDOT class. Column 3 gives the percentage of trucks in each class. Column (4) contains the number of overweight vehicles in each class. For example, in Class 9 there are 428,062 vehicles (76.2% of total) out of which 88,629 vehicles are overweight. As shown in column 5, this corresponds to 15.8% of the total truck population; or 20.7 % of those in Class 9. Overall, the number of overweight vehicles for the entire population is equal to 117,932 (21.0%). Columns 6, 7 and 8 in Table 3.1 contain the estimated percentages of divisible, special hauling and illegal vehicles respectively in each class. It is interesting to observe that prior estimates as obtained from the U.S. census and the New York State permit databases indicate that divisible permits (DV) constitute roughly 66.0% of the total overweight truck population while special hauling permits (SH) are about 4.0%, and illegally overweight (IL) trucks constitute about 30% of the entire set of overweight trucks. The results obtained in Table 3.1 are consistent with those estimates showing that 61.1% of the total overweight vehicles traveling over the I-88 Corridor are likely to be divisible permit, 1.9% are likely to be special hauling permits, and about 37.0% of the overweight trucks are likely to be illegal. The overweight trucks are those that may be exceeding the gross weight legal limit of 80,000 lb and those that do not obey the limits set by the FBF.

Table 3.1 – Permit Analysis Results of I-88 WIM Database

NSDOT (1)	**N. Trucks** (2)	**% Trucks** (3)	**OW Trucks** (4)	**% OW Trucks** (5)	**DV** (6)	**SH** (7)	**IL** (8)
5	38366	6.8%	862	0.2%	0.0%	33.0%	67.0%
6	22948	4.1%	1787	0.3%	8.1%	0.5%	91.4%
7	6178	1.1%	5277	0.9%	70.5%	0.0%	29.5%
8	23931	4.3%	933	0.2%	9.3%	33.0%	57.7%
9	428062	76.2%	88629	15.8%	63.5%	0.6%	35.9%
10	23357	4.2%	14302	2.5%	63.3%	1.1%	35.6%
11	9395	1.7%	488	0.1%	0.0%	0.0%	100.0%
12	1466	0.3%	187	0.0%	0.5%	0.0%	99.5%
13	5032	0.9%	4579	0.8%	59.7%	14.2%	26.2%
14	2695	0.5%	888	0.2%	21.9%	29.0%	49.1%
Total	**561430**	**100.0%**	**117932**	**21.0%**	**61.1%**	**1.9%**	**37.0%**

3.2.3 Bridge Data

The material and geometric characteristics of each bridge are needed to analyze the response of the bridge to each truck crossing. Part 1 of the Report for Task 1.d gives a detailed distribution of the types of bridges in the state of New York as listed in the NBI and “WINBOLTS” databases; which are summarized in Table 3.1. The NBI files have some bridges misclassified such as single span prestressed concrete slab bridges which have been removed from the table. The bridges omitted from the table consist of Frames, Trusses, Arches, Suspension, Movable, Segmental and Channel bridges. These bridge types require specialized individual analyses that cannot be accommodated in this project which addresses the effect of overweight trucks on bridge networks and corridors. Because of the dominance of the other factors on their behavior, these omitted bridges are generally not affected by individual crossings of overweight trucks. The most common bridge types in Table 3.1 that are analyzed in this study are highlighted in red. These consist of the simple span concrete slab, simple span T-beam concrete, simple span and continuous steel girder, simple span prestressed concrete, and simple multi-box prestressed concrete bridges. These most representative of New York State bridges make up about 68% of all bridges in the State of New York and also represent about 93% of the bridges in the NYSDOT database that are suitable for analysis in this research project because they prone to damage by individual overweigh trucks after omitting the special bridges listed earlier.

Specific information of the bridges is required to perform the analysis. For example, Table 3.3 lists information on the 22 bridges along the I-88 Corridor between Binghamton and Schenectady that have been selected for analysis to illustrate the process. Column 1 in Table 3.3 lists the Bridge Identification Number (BIN) for each bridge. Column 2 gives the type of the bridge. Columns 3 to 6 show the typical spacing, the total width, the deck thickness in feet, and the number of girders for each bridge. Characteristics such as beam spacing, deck thickness and number of girders are not available in the NBI or WINBOLTS databases and have to be collected bridge by bridge from the design plans. Column 7 lists the Average Daily Truck Traffic (ADTT) obtained from the WINBOLTS database by multiplying the ADTT percentage times the Average Daily Traffic (ADT) value. The values in column 8 indicate the bridge type code. The bridges are classified as steel girders with concrete deck (bridge type code = 0), prestressed concrete I-Girder (bridge type code = 1), prestressed box concrete (bridge type code = 2), concrete slab (bridge type code = 3), or cast-in-place concrete T-beam (bridge type code = 4). Column 9 indicates the material type for bridge steel (material code = 0) or concrete (material code = 1). Column 9 partially repeats the information on the material already contained in column 8 coding for fatigue analysis purposes. Column 10 indicates whether the bridge is simple span (value = 0) or continuous (value = 1). Column 11 gives the number of spans. Columns 12 to 14 provide the span lengths.

Table 3.2 - Categorization of NYS Bridges based on Material, Construction, and Number of Spans

Material Design	Design and/or construction	Number of Spans 1	2	3	4	multi
Concrete	Slab	440	25	7	3	9
	Stringer/Multi-beam or Girder	9	0	0	0	2
	Girder and Floorbeam System	5	0	2	0	0
	Tee Beam	88	11	2	1	4
	Box Beam or Girders – Multiple	34	5	3	0	0
	Box Beam or Girders - Single or Spread	0	0	0	0	0
Concrete Continuous	Slab	0	18	4	1	15
	Stringer/Multi-beam or Girder	2	0	1	0	0
	Girder and Floorbeam System	0	3	0	0	2
	Tee Beam	1	2	3	0	7
	Box Beam or Girders – Multiple	1	1	0	1	0
	Box Beam or Girders - Single or Spread	0	0	0	0	1
Steel	Slab	0	0	0	0	0
	Stringer/Multi-beam or Girder	5073	689	925	503	575
	Girder and Floorbeam System	284	36	34	12	65
	Tee Beam	0	0	0	0	0
	Box Beam or Girders – Multiple	5	1	0	2	1
	Box Beam or Girders - Single or Spread	0	1	0	0	0
Steel Continuous	Slab	0	0	0	0	0
	Stringer/Multi-beam or Girder	0	581	340	139	173
	Girder and Floorbeam System	0	7	10	6	14
	Tee Beam	0	0	0	0	0
	Box Beam or Girders – Multiple	0	6	1	1	2
	Box Beam or Girders - Single or Spread	0	0	0	0	0
Prestressed Concrete	Slab	unknown	9	7	5	2
	Stringer/Multi-beam or Girder	506	44	15	5	12
	Girder and Floorbeam System	0	0	0	0	0
	Tee Beam	27	3	11	0	9
	Box Beam or Girders – Multiple	1249	69	68	7	19
	Box Beam or Girders - Single or Spread	18	1	0	1	2
Prestressed Concrete Continuous	Slab	0	4	10	3	2
	Stringer/Multi-beam or Girder	0	10	7	0	4
	Girder and Floorbeam System	0	0	0	0	0
	Tee Beam	0	1	0	0	0
	Box Beam or Girders – Multiple	3	32	34	8	1
	Box Beam or Girders - Single or Spread	0	3	0	0	0

Table 3.3 – Geometric Characteristics of Sample I-88 Corridor Bridges

BIN (1)	Description (2)	SPAC. [ft] (3)	WIDTH [ft] (4)	DECK THICKNESS. [ft] (5)	N. GIRDER (6)	ADTT (7)	Bridge TYPE (8)	MAT. (9)	SPAN TYPE (10)	N. SPANS (11)	SPAN 1 [ft] (12)	SPAN 2 [ft] (13)	SPAN 3 [ft] (14)
1069871	Steel Simple	9.25	44	0.77	5	1127	0	0	0	1	116	-	-
1069981	PS Concrete Simple Box	4	44.4	0.5	11	1160	2	1	0	1	100	-	-
1070141	Steel Simple	7.75	44	0.71	6	1488	1	1	1	3	74	87	66
1070151	Steel Continuous	9.5	44	0.71	5	1488	0	0	1	2	141	141	-
1070162	PS Concrete Continuous I-Girder	7.67	44	0.71	6	1130	1	1	1	3	30	90	39
1070571	Steel Simple	9.25	44	0.71	5	1160	0	0	0	1	148	-	-
1070611	Steel Continuous	10	44.7	0.71	5	1397	0	0	0	1	114	-	-
1070621	Steel Simple	7.17	40	0.71	6	1397	0	0	1	3	165	210	165
1070641	Steel Simple	10.5	48	0.71	5	1280	0	0	0	1	155	-	-
1070642	Steel Simple	9.75	44	0.71	5	1280	0	0	0	1	178	-	-
1070661	Steel Simple	10.5	48	0.71	5	1280	0	0	0	1	210	-	-
1070720	PS Concrete Simple Box	9.5	44	0.71	5	2254	0	0	0	1	127	-	-
1070841	Steel Continuous	4	45	0.5	11	1127	2	1	0	1	86	-	-
1070991	Concrete Simple	10	44	0.71	5	1183	0	0	1	2	100	100	-
1071312	Steel Simple	4	45	0.5	11	639	2	1	0	1	98	-	-
1093991	Steel Simple	8.17	44	0.67	6	1364	0	0	0	1	86.5	-	-
1093991	Steel Continuous	8.17	44	0.67	6	1364	0	0	1	3	122	150	122
1094001	PS Concrete Simple Box	10	44	0.67	5	2728	0	0	1	3	116	146	116
1094141	Steel Simple	4	45	0.42	11	3787	2	1	0	2	64	64	-
1094161	Steel Continuous	9.33	44.3	0.67	5	1320	0	0	0	1	106	-	-
1094182	Steel Simple	10.25	44	0.67	5	3635	0	0	1	3	57	90	72
1094341	Steel Simple	10	44.8	0.67	5	3635	0	0	0	1	97	-	-
1094961	PS Concrete Continuous I-Girder	10.5	44	0.67	5	1300	0	0	0	1	112	-	-

3.3 Structural Analysis

The information on the number of spans and the span lengths are used to obtain the influence lines for each bridge's most critical sections. These critical sections are those that give the maximum positive moment for each span and the lowest negative moment for each continuous support. The methodology for the calculation of the influence line is available in structural analysis textbooks and the details of the algorithm applied in this study were described in Report Task 1.b.

The influence lines for each bridge are used to find the maximum moment and the minimum moment for each truck in the WIM database. The same approach is followed for the HS-20 load. These moments are used to estimate the overstress effect and the fatigue effect caused by each truck on each bridge's superstructure. Also, the weight of each axle of each truck is used to estimate the effect to the bridge deck. The procedures are explained next.

3.4 Effect of Heavy Trucks on Bridges

3.4.1 Overstress

The maximum moment from the HS-20 load is used to normalize the response of the trucks from the WIM data set for each bridge. A normalized value greater than 1.0 indicates that the applied truck produces overstress on the bridge when compared to the effect of the HS-20 load. HS-20 is used as the base line because many of the existing New York bridges were designed using the AASHTO LFD specifications with the HS-20 load as the design live load. Also, the HS-20 load effect was used as the basis for the FBF that determines the legal truck weight limits.

As an example, the results of the bending moment overstress analysis obtained for the bridge identified in the WINBOLTS database as 1069871 are provided in Figure 3.2. The bridge consists of a 116-ft simple span as shown in Table 3.3. Each histogram in Figure 3.2 shows the number of vehicles in the WIM database that exceed the effect of the AASHTO HS-20 moment for one NYSDOT class. The results are divided into groups in HS-5 increments. For example, the first group gives the number of trucks whose effect is larger than HS-20. The second group gives the number for those that produce an effect larger than HS-25, etc. Also, each group is divided into four subgroups representing the different types of overweight trucks. The LG subgroup, in blue, indicates the number of trucks that do not exceed any legal weight limit but exceed the effect of the HS load. The subgroups DV (in red), SH (in orange), and IL (in yellow) represent the numbers of the divisible, special hauling and illegal vehicles. The division of the groups into the subgroups is based on the percentages obtained during the overweight analysis as summarized in Table 3.1. Totally, the three subgroups DV, SH, and IL represent the total number of overweight vehicles that exceed the moment effect of the HS load. If one or more colors are missing in the bar diagram, this indicates that the specific subgroup is empty in that category of HS. For example, none of the histograms in Figure 3.2 show legal trucks that produce effects larger than HS-20.

The label (TR) underneath each histogram represents the percentage of vehicles in that NYSDOT class out of the total number of vehicles in the WIM dataset, while the label (OW) represents the percentage of overweights. As an example, for the trucks that have configurations classifying them as NYSDOT Class 9 per year, the number of vehicles that exceed the effect of the HS-20 is equal to 11,601. None of these vehicles is a legal weight truck (LG), 7,371 vehicles are likely divisible (DV), 99 are likely special hauling (SH) and 4,131 vehicles are likely to be illegals. The number of vehicles that exceed the effect of HS-25 is equal to 1,044. Of these 577 vehicles are likely divisible permits (DV), 11 are likely special hauling (SH), and 456 vehicles are likely to be illegals. The results of the moment effects of Class 9 trucks on bridge 1069871 are summarized in Table 3.4.

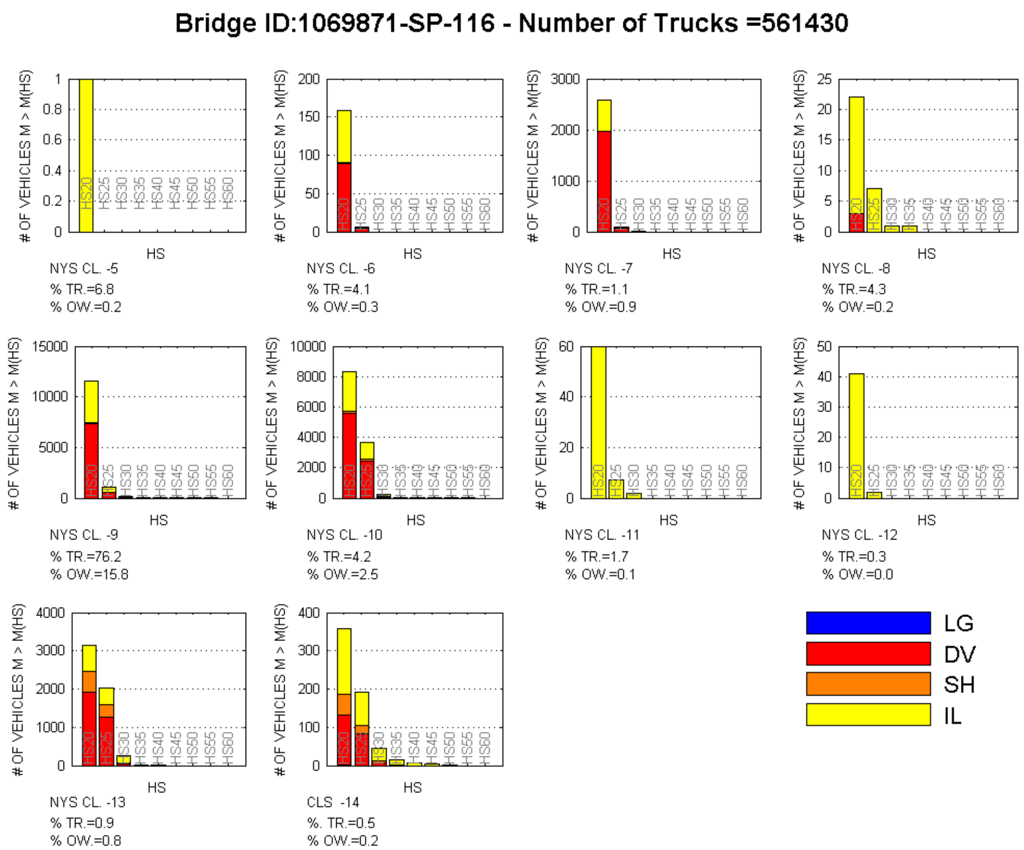

Figure 3.2 – Histograms of Moment Effect of I-88 Trucks in terms of HS Moment Effect for Bridge ID 1069871

Table 3.4 – Distribution of Moment Effect of Class 9 Trucks in terms of HS Categories

Type (1)	Total Trucks (2)	>HS20 (3)	>HS25 (4)	>HS30 (5)	>HS35 (6)	>HS40 (7)	>HS45 (8)	>HS50 (9)	>HS55 (10)	>HS60 (11)
LG	0	0	0	0	0	0	0	0	0	0
DV	7,949	7,371	577	1	0	0	0	0	0	0
		92.7%	7.3%	-	-	-	-	-	-	-
SH	117	99	11	5	2	0	0	0	0	0
		84.6%	9.4%	4.3%	1.7%	-	-	-	-	-
IL	4,820	4,131	456	152	48	18	10	4	1	0
		85.7%	9.5%	3.2%	1.0%	0.4%	0.2%	0.1%	0.0%	-
Total	12,886	11,601	1,044	158	50	18	10	4	1	0

The first row of Table 3.4 shows that none of the legal trucks exceeds the moment effect of the HS-20 load. It is also noted that four Class 9 trucks exceed the effect of the HS-50 load and one of these four trucks actually exceeds the HS-55 load effect. An inspection of the WIM data showed that these four trucks' gross weights exceed 200 kips and they all have approximately similar axle weight distributions. For example, one truck's first axle weight is 31 kip, the second axle is 52 kip, the third axle weighs 49 kip and axles four and five weigh 52 kip and 55 kip respectively. Based on the statistical estimates, it is assumed that these four very heavy trucks are likely to be illegal because their configurations did not match any of the trucks in the permit databases.

The results show that only one divisible permit truck produces a moment effect higher than that of the HS-30 truck on this bridge as highlighted in the shaded box of Table 3.4. Also, only two special hauling trucks produce moment effects between those of HS-35 and HS-40. An inspection of the data files revealed that these three special hauling trucks had similar spacings. For example, the spacings of one truck are: 8.86 ft, 5.25 ft, 6.89 ft and 5.25 ft. The axle weights are 24,464 lb, 24,023 lb, 31,958 lb, 27,109 lb, and 24,905 lb. The NYSDOT database for Special Hauling trucks shows that permits were actually provided for trucks with similar truck configurations as shown for example in Table 3.5 that gives the axle spacings and axle weights of one such truck. This particular truck produces a moment effect equal to that of the HS-38 load.

Table 3.4 only gives the information for the moments of trucks that have configurations fitting the Class 9 category for the bridge ID 1069871. Similar tables are provided in the task 1.b Report for all other classes.

Table 3.5 – Special Hauling Permit with Moment Effect between HS-35 and HS-40

ID	Date	Class	GVW (lb)	No. Axle	W1 (lb)	W2 (lb)	W3 (lb)	W4 (lb)	W5 (lb)	S1 (ft)	S2 (ft)	S3 (ft)	S4 (ft)
2195	8/12/09	9	137965	5	26921	27421	27700	27872	28051	8.83	5.42	8	5.58

3.4.2 Girder Fatigue

This section describes the fatigue damage model used in this Report based on the LRFD fatigue analysis procedure. The relationship between the stress range and the number of cycles expected during the life of the steel bridges is given in AASHTO LRFD Section 6.6.1.2.5 as:

$$\Delta F_n = \left(\frac{A}{365 \cdot Y_n \cdot n_n \cdot ADTT} \right)^{\frac{1}{3}} \tag{3.1}$$

Where: ΔF_n = Nominal fatigue stress range obtained for the fatigue design truck

A = Constant depending on the fatigue detail

Y_n = Bridge fatigue life

n_n = Amplification of number of cycles for the fatigue truck and span

ADTT = Average daily truck traffic

From Equation 3.1, it is possible to derive the equation of the bridge fatigue life, Y_n

$$Y_n = \frac{A}{75 \cdot 365 \cdot \Delta F_n^3 \cdot n_n \cdot ADTT} \tag{3.2}$$

Essentially, Eq. (3.1) and (3.2) are based on the stress range versus number of cycles to failure relationship shown in Figure 3.3 which gives the logarithm of the stress cycle versus the logarithm of the number of cycles and where the slope of the line is 1/3 which is the exponent of the expression in Eq. (3.1). The slope equal to 1/3, resulting into the exponent α=3 of ΔF_n in Eq. (3.2), is valid for all details of steel bridge components as long as the stress range ΔF_n remains above the horizontal line representing the stress range threshold. The intercept of the line represented by the parameter, A, changes depending on the detail. The assumption in Eq. (3.1) and (3.2) is that the AASHTO fatigue truck represents the typical fatigue behavior of all the trucks. Figure 3.4 gives the AASHTO LRFD Fatigue truck which is used in the fatigue damage calculations in this Report.

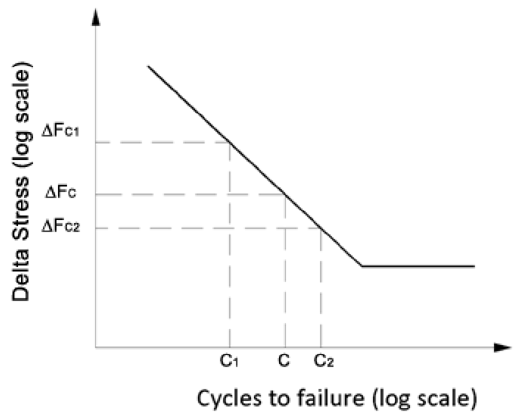

Figure 3.3 - Fatigue Stress Range versus Cycles to Failure

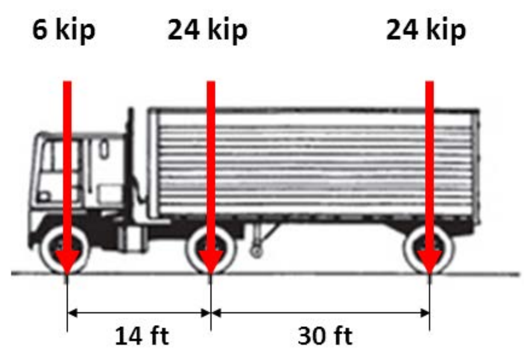

Figure 3.4 – AASHTO LRFD Fatigue Design Truck (including live load factor γ_L= 0.75)

The AASHTO LRFD uses the fatigue analysis of Eq. (3.1) for only steel bridge components. However, researchers have used a similar model for reinforced and prestressed concrete bridge members with a different slope that varies between 1/3.5 and 1/4.1 as shown in Table 3.6 (Prozzi et al, 2012). The same approach is also adopted in this study.

Table 3.6 – Exponents of Fatigue Stress Range for Different Bridge Components

Bridge Type	Alpha parameter (α)
Concrete Slab	4.1
Concrete T Beam	4.1
Concrete Box Beam	4.1
Concrete Continuous Slab	4.1
Concrete Continuous T Beam	4.1
Steel Girder	3.0
Steel Continuous Girders	3.0
Prestressed Concrete	3.5
Prestressed Concrete Box Beam	3.5
Concrete bridge decks	17.95

Eq. (3.1) is based on the fatigue damage accumulation model known as Miner's rule. According to the model, each crossing of the nominal fatigue design truck, n, that produces a stress range ΔF_n is associated with damage, D_n, equal to 1/c where c is the number of cycles to failure extracted from Figure 3.3. Accordingly, the nominal fatigue damage, D_n, for one crossing of the design truck is obtained from:

$$D_n = \frac{\Delta F_n^{\alpha} \cdot n_n}{A} \tag{3.3}$$

Where: α = inverse of the slope of the fatigue damage curve as listed in Table 3.6

n_n = is the amplification of the number of cycles per crossing of truck n.

Substituting Eq. (3.3) into Eq. (3.2), the fatigue life of the bridge can be inversely related to the damage per crossing such that:

$$Y_n = \sum_{\text{total cycles}} \frac{1}{D_n} = 75 \times 365 \times ADTT \times \frac{1}{D_n} \ . \tag{3.4}$$

Eq. (3.4) means that every crossing of the design truck will reduce the life of the bridge by $L_n = \frac{1}{D_n}$ years/crossing so that after 75x365xADTT crossings, the 75 year life of the bridge is exhausted.

Similarly, one can find the damage, D_i, for a single crossing of a truck, i, that produces a stress range ΔF_i as:

$$D_i = \frac{\Delta F_i^{\alpha} \cdot n_i}{A} \tag{3.5}$$

In Eq. (3.5) n_i is the amplification of the number of cycles per crossing of the truck having the configuration i and D_i is the damage for one truck crossing. The calculation of n_i for each truck is obtained using the rain flow analysis process that accounts for the number of sub-cycles within each truck crossing. The procedure is explained in the Appendix of Task 2.b Report.

The ratio of D_i over D_n gives the increase in damage caused by truck i compared to the damage caused by the nominal fatigue design truck n.

$$AFB = \frac{D_i}{D_n} = \frac{(\Delta F_i^{\alpha} \cdot n_i)}{(\Delta F_n^{\alpha} \cdot n)} \tag{3.6}$$

where: AFB = Amplification of Fatigue damage for Beams

AFB in Eq. (3.6) represents the amplification of the damage due to one crossing of truck i compared to the damage caused of the nominal fatigue design truck. This means that if the design truck reduces the life of the bridge by $L_n = \frac{1}{D_n}$, then truck i will reduce the life of the bridge by $L_i = \frac{1}{D_i} = AFB \times L_n$.

In the calculations executed for the fatigue damage of the bridges along the I-88 corridor, it is assumed that the design life of every bridge is 75 years as specified in the AASHTO LRFD. The number of truck crossings for each bridge per day is assumed to be the same as the ADTT provided in the WINBOLTS file for that bridge. The distribution of truck weights and configurations obtained from the WIM data collected on the I-88 WIM site is assumed to be valid for all the bridges along that corridor.

Table 3.7 shows the average damage ratio for the bridge ID 1069871 on the I-88 corridor for the trucks of Class 9 configurations. The results in Table 3.7 show the average fatigue damage ratio per truck based on the moment caused by the truck i.e. the average of the values of D_i/D_n for all the trucks in that category. It is observed that an average AFB value greater than 1.0 indicates an amplification of the damage caused by the trucks compared the damage caused by the AASHTO fatigue truck. For example, if all the trucks crossing the bridge were divisible overweight trucks causing a moment effect greater than HS20, then the average damage of the bridge would be amplified by 4.6 times the damage caused by the nominal fatigue design truck. The table also provides the number of vehicles in each group that would be expected to cross the bridge each year of its service life. For example, using the percentages of Class 9 in the WIM data and an ADTT=1127 trucks per day, it is expected that 6,794 Class 9 divisible permit trucks that cause moment effects greater than HS-20 will cross the bridge in one year. The same analysis is executed for all special hauling (SH) and illegal (IL) trucks and similar tables are also created for all truck classes and all bridges as provided in the Task 2.b Report.

Column 12 of Table 3.7 gives the average damage for each overweight category DV, SH and IL trucks. The last row shows the average fatigue damage for each HS group for all the overweight types.

Table 3.7 – Amplification of Nominal Fatigue Truck Damage for Class 9 Trucks on Girders of Bridge ID 1069871

Type (1)	Ratio & Count (2)	>HS 20 (3)	>HS 25 (4)	>HS 30 (5)	>HS 35 (6)	>HS 40 (7)	>HS 45 (8)	>HS 50 (9)	>HS 55 (10)	>HS 60 (11)	Weighted Average Per Type (12)
DV	Average Damage ratio per crossing	4.6	7.9	11.9	-	-	-	-	-	-	4.8
	No. of trucks per year	6794	577	1	-	-	-	-	-	-	
SH	Average Damage ratio per crossing	4.7	7.5	13.7	21.2	-	-	-	-	-	5.62
	No. of trucks per year	88	6	3	2	-	-	-	-	-	
IL	Average Damage ratio per crossing	4.6	8.8	14.0	22.7	32.8	44.7	60.7	75.7	-	5.72
	No. of trucks per year	3674	305	104	30	8	6	3	1	-	
Total	Weighted Average Damage ratio per crossing	4.6	8.3	14.0	22.6	32.8	44.7	60.7	75.7	-	

3.4.3 Deck Fatigue

Following the model proposed by Perdekaris (1993) and applied in NCHRP 495, the damage to bridge decks can also be modeled using a curve similar to that shown in Figure 3.3 with the log of the axle weight on the ordinate and the log of the number of axles to cause failure on the abscissa. The slope of the fatigue curve leads to an exponent of the load effect α=17.95. Accordingly, the same fatigue life reduction model derived in Eq. (3.6) can be used with minor modifications to account for the number of axles per truck.

There currently are no fatigue design criteria for bridge decks. Given that the fatigue truck in Figure 3.4 accounts for the overall moment effect while the deck damage is a function of the axle weights. It was decided to modify the fatigue truck of Figure 3.4 by splitting each tandem into two axles of 12 kips each as shown in Figure 3.5. The fatigue truck axle configuration and axle loads were the result of a statistical analysis performed on a wide set of vehicles available during the development of the LRFD fatigue criteria. The truck produces on the average the same fatigue effect that the envelope of the spectrum of vehicles produces on the girders. The single load of 24 kip in Figure 3.4 produces a concentrated point load that is higher than the average axle load of typical trucks. While the 24 kip single load is sufficiently accurate to model the effect of fatigue damage on bridge girders, it would cause higher than reasonable fatigue damage to bridge decks. For this reason, the tandem load of 24 kip is split in two forces of 12 kip each spaced at 4-ft in order to better simulate the effect of tandems on bridge deck damage. It is emphasized that this assumption is based on engineering judgment. Future research should develop a typical fatigue truck configuration that should be valid for bridge decks.

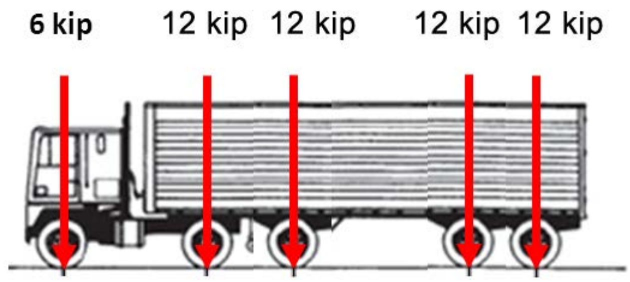

Figure 3.5 – Modified AASHTO LRFD Deck Fatigue Design Truck (including live load factor γ_L= 0.75)

Equation (3.7) is a modification of Eq. (3.6) developed to evaluate the amplification, AFD, to the fatigue damage of the design truck, n, of Figure 3.5 resulting from the crossing of an overweight truck, i. If the AFD ratio is greater than 1.0, then the expected design life of the deck is reduced.

$$AFD = \frac{D_i}{D_n} = \left(\frac{\sum_{axles} \left(P_j\right)_i^{17.95}}{\sum_{5} \left(P_j\right)_n^{17.95}} \right)^{1/17.95} \qquad (3.7)$$

where: AFD = Amplification Factor of Deck fatigue damage due to crossing of truck i

P_j = weight of axle j of the truck.

The variables with the subscript n are those associated with the fatigue design truck which has 5 axles. Eq. (3.7) means that if the design truck reduces the life of the deck by $L_n = \frac{1}{D_n}$, then truck i will reduce the life of the deck by $L_i = \frac{1}{D_i} = AFD \times L_n$.

In the calculations executed for the fatigue damage of the bridge decks, we assume that the design life of each deck is 40 years following NYSDOT standards. The number of truck crossings for each bridge is assumed to be the same as the ADTT provided in the WINBOLTS file for that bridge. The percentage of trucks per type and weight is assumed to be the same as that in the WIM database.

Table 3.8 shows the average damage ratios of the deck for the bridge ID 1069871 on the I-88 Corridor. This table is for Class 9 trucks. It is observed that an AFD value equal to 1.0 indicates that the modified AASHTO fatigue truck is representative of the fatigue damage caused on the bridge deck. The results in Table 3.8 show the average deck fatigue damage ratio per truck based on the weight of the axles of truck i. The value in each group is the average of the AFD's for all the trucks in that category that cause

AFD greater than 1.0. It is noted that the HS moment effect is only used for grouping the trucks while the deck damage is a function of the axle weights. For example, it is expected that 6,974 Class 9 divisible permit trucks that cause moment effects greater than HS-20 and a deck fatigue damage ratio AFD>1.0 will cross the bridge in a given year. The average deck damage caused by these trucks is 1.8 times the damage caused by the modified fatigue design truck. The same analysis is executed for all special hauling (SH) and illegal (IL) trucks and similar tables are also created for all truck classes and all bridges and provided in Appendix C of the Task 2.b Report.

Column 12 of Table 3.8 gives the average damage for each overweight category DV, SH and IL trucks. The last row shows the average fatigue damage for each HS group for all the overweight types.

Table 3.8 – Amplification of Nominal Fatigue Truck Damage for Class 9 Trucks on Deck of Bridge ID 1069871

Type (1)	**Ratio & Count** (2)	**>HS 20** (3)	**>HS 25** (4)	**>HS 30** (5)	**>HS 35** (6)	**>HS 40** (7)	**>HS 45** (8)	**>HS 50** (9)	**>HS 55** (10)	**>HS 60** (11)	**Weighted Average Per Type (12)**
DV	Average Damage ratio per crossing	1.8	2.2	2.5	-	-	-	-	-	-	1.8
	No. of trucks per year	6794	577	1	-	-	-	-	-	-	
SH	Average Damage ratio per crossing	1.8	2.0	1.9	2.3	-	-	-	-	-	1.8
	No. of trucks per year	88	6	3	2	-	-	-	-	-	
IL	Average Damage ratio per crossing	1.8	2.4	2.8	3.2	3.7	4.0	4.5	4.6	-	1.9
	No. of trucks per year	3674	305	104	30	8	6	3	1	-	
Total	Weighted Average Damage ratio per crossing	1.8	2.3	2.8	3.2	3.7	4.0	4.5	4.6	-	

3.4.4 Summary of Damage Analysis

In this section, three parameters are defined in order to quantify the effects of the overweight vehicles on bridges. The first parameter is related to the overstress induced by the bending moment of overweight trucks compared to the moment of the HS-20 design truck. The second parameter is the fatigue damage ratio caused by overweight trucks on bridge beam components compared to the damage caused by fatigue design vehicle. The third parameter concerns the fatigue of the bridge deck. Because there are no design criteria for deck fatigue life, the analysis is performed by using a variation on the AASHTO fatigue design truck which was originally developed to simulate the fatigue damage on steel bridge members caused by typical truck spectra. The proposed fatigue deck design truck will have five axles with the front axle being 6 kip and each other axle being 12 kip.

3.5 Bridge Cost Models

This section describes the bridge cost models that were developed during the course of this study for use in allocating the cost of bridge overstress, bridge beam fatigue, and bridge deck fatigue.

A bridge system may be considered to consist of two main subsystems: a superstructure and a substructure. The cost of constructing each component depends on different parameters. But, the cost of the material in each subsystem is generally about 50% of the cost of the subsystem as observed from the heavy construction cost data provided in RSMeans (2009). This indicates that the cost for field construction is on the average equal to the cost of the material. This observation has also been made by other researchers as for example noted by Garrell (2012). The following sections describe the models used to estimate the cost of a bridge's substructure and superstructure separately. The development of the models is addressed in the Task 1.d report and in Appendices D, E and F of the Task 2.b Report.

3.5.1 Concrete Deck Cost Model

The cost of a concrete bridge deck is estimated by multiplying the unit cost of the concrete used in constructing the deck in [$/cft] times the volume of the deck plus the cost of the reinforcing steel. The unit cost of the concrete is between 8.0$/ft^3 to 9.0 $/ft^3 according to RSMeans (2009). Based on the review of the plans of the twenty bridges on I-88, it is assumed that the volume of steel deck reinforcement is 0.1% of the concrete deck volume as obtained from the bridge plans at a cost of 1.0 $/lb as per RSMeans (2009).

3.5.2 Substructure Cost Model

The cost of the substructure depends on its geometric, material and design load characteristics. But, it is also a function of the topography and the geological characteristics of the site. Therefore, it is difficult in principle to obtain generalized estimates of the effective cost of substructures. However, a review of the bridge plans provided by NYSDOT and a rough survey of the quantities of the substructures and foundations for the sample of 22 bridges located along the I-88 corridor indicated that a trend between substructure cost and bridge deck area exists as shown in Figure 3.6. In this analysis, the cost estimates are obtained using the 2009 database in RSMeans (2009).

The data for the substructure cost is divided into two groups: single span and continuous bridges. In both cases, the cost is given as a function of the area of the deck. Although the NYSDOT WINBOLTS files estimate the cost based on the shoulder break area, in Figure 3.6, the cost is presented in terms of area of the deck in order to provide a basis for comparison with cost data in the literature which is typically presented in terms of dollars per square foot of deck.

Figure 3.6 shows a large scatter for the substructure cost in terms of the deck area along a general trend line. However, for the purposes of this study, linear relationships for the two groups of simple spans and continuous bridges are deemed acceptable. The results of the regression analysis give Eq. 3.8 and 3.9. The regression coefficients R^2 are 0.70 and 0.77 respectively. These two equations already account for labor and installation costs.

$$\begin{aligned} CSB_S &= 110.5 \cdot (TL \cdot WD) - 94{,}009 \\ R^2 &= 0.70 \end{aligned} \tag{3.8}$$

$$\begin{aligned} CSB_C &= 23 \cdot (TL \cdot WD) + 585{,}340 \\ R^2 &= 0.77 \end{aligned} \tag{3.9}$$

where: CSB_S = Cost of simply-supported bridge substructure

CSB_C = Cost of continuous bridge substructure

TL = Total length of the bridge

WD = Width of the bridge deck

R^2 = Regression coefficient (Coefficient of determination).

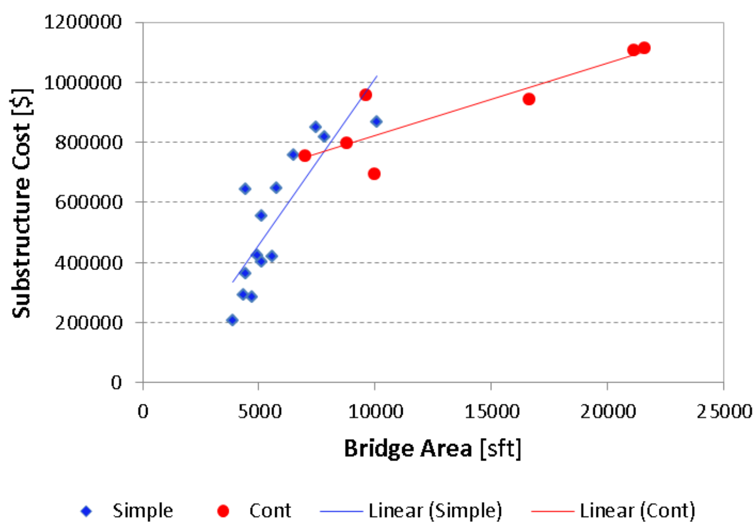

Figure 3.6 –Substructure Cost vs. Bridge Deck Area for 22 Bridges along I-88 Corridor

3.5.3 Superstructure Cost Model

The 22 bridges along the I-88 Corridor that have been selected for analysis represent four of the most typical five bridge superstructure topologies in the inventory of New York State. These represent the majority of bridge types throughout the state. The most common superstructure as shown in Table 3.2 consists of composite steel I-girders with concrete deck. The second most common type consists of prestressed concrete I-girder bridges with reinforced concrete deck, followed by prestressed concrete box girders, reinforced concrete slabs, and reinforced concrete T-beam structures. A description of the cost model is given in this Chapter for each bridge type. These models give the cost of the materials only based on the procedures presented in Task 1.d Report and Appendices D, E and F of the Task 3.b Report.

Steel Girder Bridges [ST]

The cost of steel multi-girder superstructures consists of two parts: the cost of the steel girders and that of the deck. An equation for estimating the cost of the steel in composite steel girder members is developed following an approach similar to that described Report. The equation gives the cost as a function of the span length, the girder spacing and the design live load moment for positive bending. The cost is expressed as:

$$CST = N \cdot \begin{bmatrix} X_1 + X_2 \cdot S + X_3 \cdot M + X_4 \cdot S^2 + X_5 \cdot M^2 + X_6 \cdot (S \cdot M) \\ + X_7 \cdot (S^2 \cdot M) + X_8 \cdot (S \cdot M^2) \end{bmatrix} \cdot \alpha \cdot L \cdot UCST \qquad (3.10)$$

where: CST = Cost of steel [$]
S = Girder spacing [ft]
L = Span length [ft]
α = Factor accounting for the effect of continuity
M = Live Load bending moment [kip-ft]
X_i = Constants of the regression analysis listed in Table 3.9 for different αL
$UCST$ = Unit cost of structural steel [$/lb]
N = Number of girders

Table 3.9 – Steel Girder – Positive Bending Regression Coefficients

Coeff.	**Value**						
	αL<= 30 [ft]	30<αL<= 50	50<αL<= 70	70<αL<= 100	100<αL<= 140	140<αL<= 200	αL> 200
X1	37.1	-35.1	94.3	55.6	83.1	207.7	258.6
X2	-7.57	19.76	0.9	8.7	22.86	0.64	13.17
X3	0.17	0.21	0.03	0.1	0.07	0.06	0.04
X4	0.688	-0.548	-0.045	0.104	-2.148	-0.07	0.318
X5	-1.94E-04	-5.20E-05	2.00E-06	-6.00E-06	1.20E-05	1.00E-06	-1.00E-06
X6	1.30E-02	-2.50E-02	-2.00E-03	-1.60E-02	-2.00E-02	-5.00E-03	-1.00E-03
X7	-1.70E-03	6.00E-04	2.00E-04	6.00E-04	1.70E-03	5.00E-04	4.00E-05
X8	1.74E-05	5.20E-06	-4.00E-07	1.10E-06	-1.20E-06	-2.00E-07	-1.00E-08

The parameter α in Eq. (3.10) is a factor that reflects the effect of span continuity on the length of segments that are in positive bending. For example, for the typical three-span continuous bridge in Figure 3.7, the portion of the span primarily subjected to positive moment for the first and last span is equal to $\alpha 1$ of the entire span length. This value is on the average equal to 0.7 for different span length configurations. For the middle span, $\alpha 2$ which gives the portion of the span that is primarily in positive bending is on the average 0.6 times the middle span length. This means that α in Eq. (3.10) is equal to 0.7 when estimating the cost of the first span while α=0.6 is used in Eq. (3.10) to find the cost of the interior span. When the bridge is simply supported α is equal to 1.0.

In this report, the unit cost of steel UCST ranges between 1.55 $/lb to 2.00$/lb according to the estimates of RSMeans (2009).

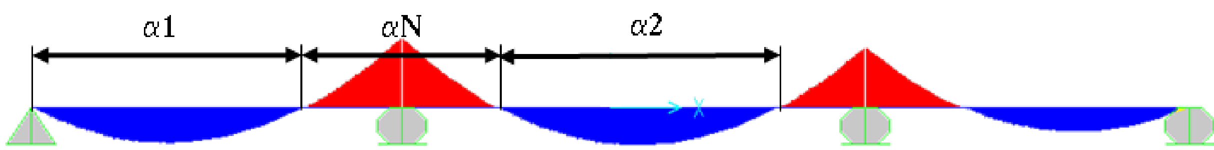

Figure 3.7 –Span Length Continuity Factor

A similar analysis as that reported in Task 1.d is also performed for the cost of steel for the sections of continuous I-girder bridges in negative bending. The procedure is explained in details in Appendix F of the Task 2.b Report. The cost for sections in negative bending can be estimated using the expression presented in Eq. (3.11).

$$CST = N \cdot \frac{[X_1 + X_2 \cdot M]}{144} \cdot \alpha \cdot (L_L + L_R) \cdot UWST \cdot UCST \qquad (3.11)$$

where: CST = Cost of steel [$]

L_L = Length of span at the left of the support [ft]

L_R = Length of span on the right of the support [ft]

α = Proportion of each span in negative bending (average value is equal to 0.25)

M = Bending moment [kip-ft]

X_i = Constants of the regression analysis tabulated in Table 3.10

$UWST$ = Unit weight of steel = 490 [lb/cft]

$UCST$ = Unit cost of steel [$/lb]

N = Number of girders

Table 3.10 – Steel Girder – Negative Bending Regression Coefficients

Coeff.	Value
X1	32.8
X2	0.0046

The combined length ratio in negative bending near one interior support from the two adjacent spans is indicated by αN in Figure 3.7. The average αN value is approximately 0.25 of the sum of the adjacent span lengths for negative bending.

The term $\left[X_1 + X_2 \cdot M\right]$in Eq. (3.11) represents the area of the section in square inches as a function of the moment in kip-ft. The unit cost for the steel in negative bending is assumed to be the same as that for positive bending taken to be between 1.55 $/lb to 2.0$/lb according to the estimates of 2009 [RSMeans (2009)] where the specific weight of steel is equal to 490 lb/ft^3.

Prestressed I-Girder Bridges [PS]

The cost of a prestressed concrete I-Girder superstructure is the sum of the concrete and the prestressing steel costs. The procedure to obtain the required cross section and prestressing steel area is explained in detail in Appendix D of Task 2.b Report. According to the cost database, the unit cost of a PS concrete I-Girder is 50 $/ft^3 (or 1350 $/yd^3) for the concrete (RSMeans, 2009). The properties of the cross section area are listed for each beam type in Table 3.11. The section type is chosen to satisfy the two inequalities given in Eq. (3.12) and Eq. (3.13) in function of the span length, the beam spacing, and the nominal truck intensity HS. The coefficients X_i, with *i* varying from 0 to 3 are listed in Table 3.12 for each section type.

$$S - X_0 - X_1 \cdot L \leq 0 \tag{3.12}$$

$$\frac{HS - 15}{5} - X_2 - X_3 \cdot S \cdot L \leq 0 \tag{3.13}$$

where: S = Girder spacing [ft]

L = Span length [ft]

HS = HS design truck level [20, 25 … 65]

X_i = Constants of regression analysis tabulated in Table 3.12

Table 3.11 – PS I-Girder - Cross Section Areas

Type	Area	Weight
	[in2]	[kip/ft]
I	276	0.287
II	369	0.384
III	560	0.583
IV	789	0.822
V	1,013	1.055
VI	1,085	1.13
VII	1,143	1.19
VIII	1,227	1.28
IX	1,365	1.43

Table 3.12 – PS I-Girder – Regression Coefficients

Type	X_0	X_1	X_2	X_3
II	16	-0.2	13.0	-0.040
III	22	-0.2	17.0	-0.017
IV	26	-0.2	14.3	-0.017
V	30	-0.2	15.3	-0.014
VI	32	-0.2	15.7	-0.013
VII	32	-0.2	18.0	-0.016
VIII	34	-0.2	17.0	-0.012
IX	-	-	23.0	-0.015

The area of prestressing steel in square foot is obtained from Eq. (3.14). This equation is obtained from a regression analysis of different bridges designed to carry different levels of HS trucks.

$$PSSA = \frac{X_4(HS) + X_5(HS) \cdot L + X_6(HS) \cdot S}{144} \tag{3.14}$$

where: $PSSA$ = Area of prestressing steel [sft]
S = Girder spacing [ft]
L = Span length [ft]
HS = HS design truck level [20, 25 … 65]
X_i = Constants of regression function depend on HS and coefficients listed in Table 3.13

The values of X_i depend on HS and the coefficients in Table 3.13 in the form of Eq. (3.15)

$$X_i = t_{i0} + t_{i1} \cdot \frac{HS - 15}{5} \tag{3.15}$$

Table 3.13 – PS I-Girder – PS Steel Area Regression Coefficients

Coeff.	t_0	t_1
X_4	-3.14	0.163
X_5	0.070	0.001
X_6	0.242	-0.014

The total cost of a prestressed concrete I-girder is obtained by summing the cost of the concrete girder and the cost of steel according to Eq. (3.16). The unit cost of the prestressing steel is taken to be between 1.0 $/lb and 1.5$/lb.

$$CPS = N \cdot \left[AS(L,S,HS) \cdot L \cdot UCPS + PSSA \cdot L \cdot UWST \cdot UCSP\right] \tag{3.16}$$

where: CPS = Cost of the girders [$]
L = Span length [ft]
$AS(L,S,HS)$ = Cross sectional area which depends on span length [ft], spacing [ft] and truck intensity
$UCPS$ = Unit cost of I-Girder [$/cft]
$PSSA$ = Area of the prestressing steel [sft]
$UWST$ = Unit weight of steel = 490 [lb/cft]
$UCSP$ = Unit cost of prestressing steel [$/lb]
N = Number of girders

Prestressed Multi-Box Beams [PB]

The cost of the box system is the summation of the precast concrete and prestressing steel. According to the cost database, the unit price of a PS concrete box is equal to 50 $/cft (or 1350 $/cyd) for the bare material (RSMeans, 2009). The procedure to obtain the required cross section and prestressing steel area is explained in detail in Appendix E of Task 2.b Report. The properties of the cross section area are listed for each beam type in Table 3.14. The section type is chosen to satisfy the inequality given in Eq. (3.17) in function of the span length and the nominal truck intensity HS. The coefficients X_i, with *i* equal to 0 and 1 are listed in Table 3.15 for each section type.

$$\frac{HS-15}{5} - X_0 - X_1 \cdot L \leq 0 \tag{3.17}$$

where: L = Span length [ft]

HS = HS design truck level [20, 25 … 65]

X_i = Constants of the regression analysis tabulated in Table 3.14

Table 3.14 – PS Box-Girder –Cross Section Area

Type	Area	Weight
	[in2]	[kip/ft]
B-I	692.5	0.721
B-II	752.5	0.784
B-III	812.5	0.846
B-IV	842.5	0.878
B-V	916.0	0.954
B-VI	1,006.0	1.048

Table 3.15 – PS Box-Girder – Cross Section Regression Coefficients

Type	X_0	X_1
B-I	12.5	-0.12
B-II	11.0	-0.10
B-III	12.0	-0.10
B-IV	13.0	-0.10
B-V	14.0	-0.10
B-VI	16.0	-0.10

The area of prestressing steel in square foot is obtained from Eq. (3.18). This equation is obtained from a regression analysis of different bridges for different live load intensities.

$$PBSA = \frac{X_2 + X_3 \cdot L + X_4 \cdot \frac{HS-15}{5}}{144} \tag{3.18}$$

where: $PBSA$ = Area of the pre-stressed steel for boxes [sft]

L = Span length [ft]

HS = HS design truck level [20, 25 … 60]

X_i = Constants of the regression analysis tabulated in Table 3.16

Table 3.16 – PS Boxes – PS Steel Area Regression Coefficients

Coeff.	Value
X_2	-1.52
X_3	0.076
X_4	0.053

The cost of the boxes is obtained by summing the cost of the concrete beam and the cost of steel according to Eq. (3.19). The unit cost of the prestressing steel is taken equal to 1.0 $/lb to 1.5$/lb.

$$CPB = N \cdot \left[AS(L,HS) \cdot L \cdot UCPB + PBSA \cdot L \cdot UWST \cdot UCSP \right] \tag{3.19}$$

where: CPB = Cost of the boxes [$]
L = Span length [ft]
$AS(L,HS)$ = Area of concrete cross section as a function of length [ft] and truck intensity
$UCPB$ = Unit cost of the boxes [$/cft]
$PBSA$ = Area of prestressing steel [sft]
$UWST$ = Unit weight of steel = 490 [lb/cft]
$UCSP$ = Unit cost of prestressing steel [$/lb]
N = Number of boxes

Concrete T-Beam Bridges [T]

A regression fit is used to relate the volume of concrete in T-beam bridges as a function of the span length as shown in Eq. (3.20). The coefficients X_0, X_1 and X_2 of the regression equation are provided in Table 3.17 where X_2 is the curvature, X_1 is the slope of the line and X_0 is the intercept.

$$V = X_0 + X_1 \cdot L + X_2 \cdot L^2 \tag{3.20}$$

where: V = Concrete volume in cubic foot
L = Span length in ft
X_i = Coefficients of the regression listed in Table 3.17

Table 3.17 – Coefficients of Regression Relating Section Volume and Span Length

Coeff.	Value
X_0	2.9360
X_1	0.1090
X_2	0.0011

The total area of reinforcing steel, A_s, is estimated as a function of the span length and live load intensity HS, using an equation of the form:

$$A_s = X_3 + X_4 \cdot L + X_5 \cdot HS + X_6 \cdot HS \cdot L + X_7 \cdot L^2 \tag{3.21}$$

where: A_s = Reinforcing steel areas in in^2

L = Span length in ft

HS = HS design truck level [20, 25 … 60]

X_i = Coefficients of the regression listed in Table 3.18.

Table 3.18 – Coefficients of Regression T-Beam Steel Reinforcement

Coeff.	Value
X_3	11.5980
X_4	-0.3186
X_5	0.1800
X_6	0.0037
X_7	0.0044

The total cost is found as a function of the concrete volume and volume of reinforcing steel.

Monolithic Concrete Slabs [CS]

A linear regression Eq. (3.22) with coefficients X_0, and X_1 provided in Table 3.19 is used to obtain the cross section volume of monolithic concrete slab bridges for a given span length, L:

$$V = X_0 + X_1 \cdot L \tag{3.22}$$

where: V = Concrete volume in cubic foot

L = Span length in ft

X_i = Coefficients of the regression listed in Table 3.19.

Table 3.19 – Coefficients of Regression Relating Section Volume of Slab Bridges to Span Length

Coeff.	Value
X_0	0.4670
X_1	0.0420

The area of reinforcement steel, A_s, can be estimated as a function of span length and live load intensity HS can be estimated using an equation of the form:

$$A_s = X_2 + X_3 \cdot L + X_4 \cdot HS + X_5 \cdot HS \cdot L + X_6 \cdot L^2 \qquad (3.23)$$

where: A_s = Reinforcing steel areas in in^2

L = Span length in ft

HS = HS design truck level [20, 25 … 60]

X_i = Coefficients of the regression listed in Table 3.20.

Table 3.20 – Coefficients of Regression Monolithic Slab Steel Reinforcement

Coeff.	Value
X_2	0.1175
X_3	0.0129
X_4	0.0210
X_5	0.0015
X_6	0.0002

The total cost is found as a function of the concrete volume and volume of reinforcing steel and the unit cost of each.

3.6 Bridge Cost Allocation for Example Corridor

The bridge cost models presented in Section 3.5 along with the methods proposed in Section 3.4 for analyzing the overstress and fatigue damage of main bridge members and the fatigue analysis of bridge decks are implemented in a computer program to estimate the cost of damage to a set of bridges along a given corridor for which WIM truck data are available. The procedure was implemented for the typical 22 bridges along the I-88 Corridor that are listed in Table 3.3. The members of each bridge were first designed to exactly meet the AASHTO LFD standard specifications for member bending strength with the HS-20 design load. As mentioned earlier, the HS-20 load is used because it is the basis of the FBF formula that identifies which truck axle groups are within the weight limits and which ones are considered overweight. The asset costs of these 22 bridges are then calculated using the cost models described in Section 3.4. The results are summarized in Table 3.21. The first column of the table shows the WINBOLTS bridge identification number. The second column identifies whether the bridge is simple span (S) or continuous (C). The third column identifies the bridge type where ST=steel I-girder, PB=multi-beam prestressed box, PS= prestressed I-girder. Column four shows the total cost of the bridge designed for the HS-20 load including labor and installation cost in dollars. Column five gives the bridge deck area in square foot, and finally the last column shows the cost of each bridge per square foot. The cost value in column (4) includes material cost and labor cost. As mentioned earlier, the labor cost is assumed to be equal to the material cost. The material cost was obtained by summing the cost of the substructure from Eq. (3.8) or (3.9) as appropriate to the cost of the deck and that of the superstructure obtained from Eq. (3.11) to (3.23) assuming that the bridge members are designed for HS-20 loading which is used as the base line.

The range of the cost per area in column (6) varies from a minimum of 145.11 \$/sft for bridge ID 1094001 to a maximum of 319.00 $\$/ft^2$ for bridge ID 1070661. Overall, the average bridge cost per square foot for the 22 bridges is equal to 202 \$/sft. Comparing this value to the average cost in the literature, it is found that the Federal Highway Administration in 2002 estimated the average bridge construction cost in the state of New York to be equal to 155 $\$/ ft^2$ (FHWA, 2003). Assuming an average inflation rate between 2002 and 2009 equal to 2.6%, the adjusted FHWA bridge cost for the year 2009 would be 185.5 $\$/ ft^2$. For the general purpose of this study and the assumptions made, the values obtained with the proposed procedure are found to be similar to those in the FHWA (2003) report.

Table 3.21 – Cost of Example Bridges on I-88 Bridge for baseline HS-20 designs

Bridge ID (1)	**Type** (2)	**Mat** (3)	**Asset Cost** [$] (4)	**Area** [sft] (5)	**Cost/Area** [$/sft] (6)
1069871	S	ST	919,220	5,104	180.10
1069981	S	PB	1,110,000	4,440	250.00
1070141	C	PS	1,557,800	9,988	155.97
1070151	C	ST	2,006,370	12,408	161.70
1070162	C	PS	1,412,100	6,996	201.84
1070571	S	ST	1,630,860	6,512	250.44
1070611	S	ST	910,290	5,096	178.63
1070621	C	ST	3,684,010	21,600	170.56
1070641	S	ST	1,814,860	7,440	243.93
1070642	S	ST	2,107,620	7,832	269.10
1070661	S	ST	3,215,480	10,080	319.00
1070720	S	ST	1,026,820	5,588	183.75
1070841	S	PB	898,590	3,870	232.19
1070991	C	ST	1,499,680	8,800	170.42
1071312	S	PB	1,093,090	4,410	247.87
1093991	S	ST	4,252,140	21,120	201.33
1094001	C	ST	2,413,420	16,632	145.11
1094141	S	PB	1,213,800	5,760	210.73
1094161	S	ST	827,440	4,696	176.20
1094182	C	ST	1,475,760	9,636	153.15
1094341	S	ST	736,310	4,346	169.42
1094961	S	ST	881,380	4,928	178.85

3.6.1 Overstress Cost Allocation

The method followed by the FHWA cost allocation study (FHWA, 1997) is used to allocate the costs that overweight trucks would be responsible for, using bridges designed for the HS-20 live load as the base line. This approach assumes that all the overweight trucks in the I-88 WIM database whose moments exceed the moment effect of the HS-20 load would overstress any bridge that is designed to exactly meet the HS-20 AASHTO design criteria. These trucks should be held responsible for this overstress which decreases the structural safety margin implied in the specifications and increases the risk to the traveling public. According to the FHWA cost allocation method, in order to maintain the same risk level that the public would be exposed to if none of the trucks is overweight, bridges that are subjected to overweight trucks should be upgraded to accommodate the overstress caused by the overweight vehicles. For example, to eliminate the additional risk incurred due to trucks whose moment effects is between HS-20 and HS-25, the bridges along the exposed corridor should be designed for the HS-25 load level. The additional cost incurred by increasing the design load level to HS-25 should be allocated to the trucks that produce moments higher than HS-20. The same logic is used to eliminate the additional risk caused by trucks whose moment effect is higher than HS-25, and so on. This concept is schematically explained in Figure 3.8.

The graphs in Figure 3.8 show on the ordinate the bending moment effect of the truck population, represented by the scatter dots. The abscissa gives the amplification of the fatigue damage for the truck and will be used in later sections to explain the fatigue damage allocation model.

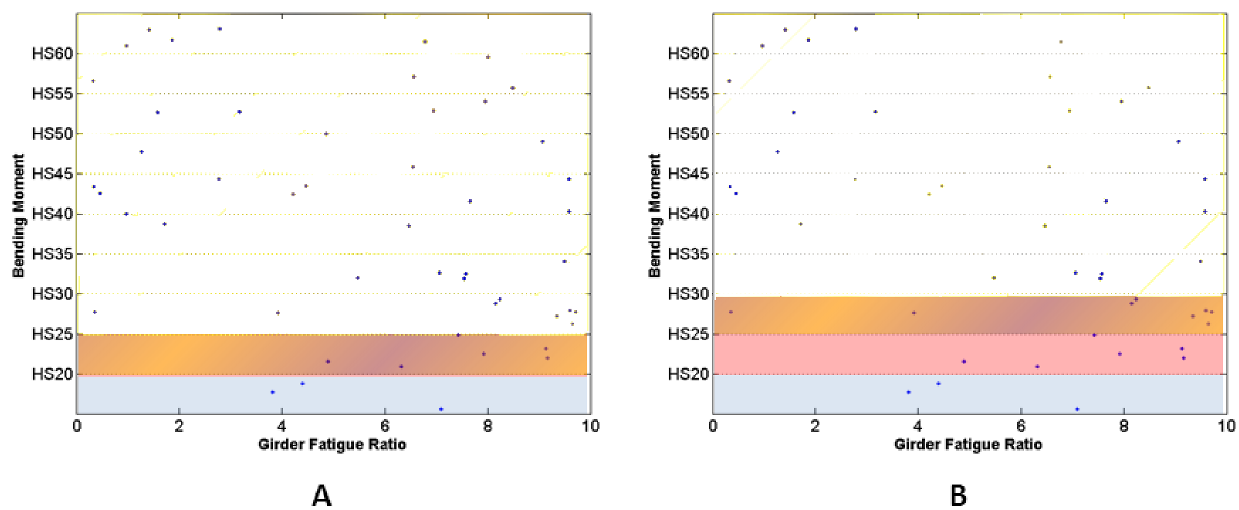

Figure 3.8 – Schematic Representation of NYS Overstress Cost Allocation Method

In Figure 3.8.A the grey band covers the trucks that are within the HS-20 limits. The orange-yellow-white zone covers all the trucks that exceed the HS-20 moment effect overstressing the HS-20 bridges. If we increase the design load to the HS-25 level, all the trucks in the orange-yellow-white band will have to contribute to the costs of that increase. Figure 3.8.B depicts the scenario followed for allocating the cost for vehicles that exceed the moment effect of HS-25 bridges. In this case, the trucks in the orange-yellow-white zone are allocated the cost of increasing the design level from HS-25 to HS-30. This cost burden is in addition to having already been charged for the cost of increasing the design level from HS-20 to HS-25. However, the trucks in the pink band should not be charged for increasing the design above HS-25. The same procedure is repeated for vehicles that exceed the moment effect of HS-30 loads and so on.

The procedure is followed for each of the 22 sample bridges by studying the effect of each truck from the I-88 WIM data. As an example, Table 3.22 shows the number of trucks in each HS category for the bridge ID 1069871. Each column from 2 through 11 represents the number of vehicles in each HS category expected to cross the bridge every year. These trucks are also divided into groups classified as divisible (DV), special hauling (SH) and illegals (IL). Equation (3.24) shows the cost of overstress per crossing. The cost is obtained by dividing the incremental cost for upgrading the bridge to the next design truck level by the number of vehicles exceeding the current HS-20 design load effect.

$$CT_{HS-I} = \frac{Cost_{HS-J} - Cost_{HS-20}}{DL \cdot NT_{HS-I}} \tag{3.24}$$

where: $HS-I$ = HS design class from 20 through 55

$HS-J$ = the next design class above I from 25 through 60

CT_{HS-I} = Cost per truck crossing of vehicles that exceed the design class I [$]

$Cost_{HS-J}$ = Cost of the bridge for design truck J [$]

$Cost_{HS-20}$ = Cost of the bridge for design truck HS-20 [$]

DL = Design life in years assumed equal to 75

NT_{HS-I} = Number of vehicles that exceed the effect of the design load HS-I per year

In Eq. (3.24) the cost is divided by the design life to obtain the cost per year and the count of trucks is per year. In this example, the design life is considered to be 75 years.

Table 3.22 – Total Number of Overstress Vehicles per Year for Bridge ID 1069871 Based on WIM Truck Sample

OW Type (1)	>HS20 (2)	>HS25 (3)	>HS30 (4)	>HS35 (5)	>HS40 (6)	>HS45 (7)	>HS50 (8)	>HS55 (9)	>HS60 (11)
DV	18,751	4,548	54	-	-	-	-	-	-
SH	1,138	630	79	4	-	-	-	-	-
IL	8,031	2,282	608	103	38	16	7	3	-
Total	27,920	7,460	741	107	38	16	7	3	-

The total cost of each bridge is estimated with the equations provided in Section 3.5. As an example, the analysis of the cost of the bridge ID 1069871 for HS-20 load is described to illustrate the cost analysis for the deck, the steel girders, and the substructure.

Deck material cost is calculated for a bridge width of 44 ft, a deck thickness of 0.792 ft (9.50 in) and a span length of 116 ft. We are also assuming a unit cost of 9$/cft for the concrete and 1.0$/lb for the reinforcing steel where the steel reinforcement constitutes 0.1% of the volume of deck. This leads to:

Deck Material Cost = (44 x 0.792 x 116) x 9.0 $/cft + (44 x 0.792 x 116) x 0.001 x 490 lb/cft x 1.0$/lb =
= $38,350

The cost of the material for five girders is obtained using Eq. (3.10) with an effective span αL=116 ft. The unit cost of $1.55/lb is assumed for structural steel. In this cost it is also included an additional 5% of material for the connections.

Beam Material Cost =

$$5\times\begin{bmatrix}83.1+22.86\times8.0+0.07\cdot M-2.15\times8.0^2+1.20\times10^{-5}\cdot M^2 \\ -2.00\times10^{-2}(8.0\cdot M)+1.70\times10^{-3}(8.0^2\cdot M)-1.20\times10^{-6}(8.0\cdot M^2)\end{bmatrix}\times(1.0\times116)\times1.55\times1.05$$

= $186,270

S = beam spacing= 8.0 ft

L = span length = 116 ft

α = percentage of span in positive bending = 1.0

M = required HS moment = 1,589.0 kip-ft

The moment M is the moment of the design load HS-20 per girder.

The cost of the substructure including material and labor is obtained using Eq. (3.8) for the simple supported bridge with a deck area 44 ft x 116 ft.

Substructure Cost = (44 x 116) x 110.5 – 94009 = $469,980

Assuming that the labor cost for the deck and beams is equal to the material cost, the total cost for the bridge is obtained as:

Total Cost for HS-20 load = 2 x (186,270 + 38,350) + 469,980 = $919,220

Similarly, the cost of the bridge ID 1069871 is estimated for an HS-25 design live load. The cost of the deck is assumed to remain approximately the same for different HS design loads; the deck asset cost is therefore kept at $38,350. The superstructure cost is estimated using the same Eq. (3.10) but with a different moment value M equal to 397.4 kip-ft which is the moment effect of the HS-25 truck. The cost of the steel girders becomes $198,590.

Cost =

$$5\times\begin{bmatrix}83.1+22.86\times8.0+0.07\cdot M-2.15\times8.0^2+1.20\times10^{-5}\cdot M^2 \\ -2.00\times10^{-2}(8.0\cdot M)+1.70\times10^{-3}(8.0^2\cdot M)-1.20\times10^{-6}(8.0\cdot M^2)\end{bmatrix}\times(1.0\times116)\times1.55\times1.05$$

= $198,590

S = 8.00 ft

L = 116 ft

α = 1.0

M = 1,986.4 kip-ft

The superstructure cost difference between the HS-25 and HS-20 designs is about 5.5%. The difference is assumed to be the same also for the substructure. The labor and installation costs together are assumed to be equal to the material cost. Accordingly, the total cost of the bridge designed for HS-25 live load becomes equal to:

Total Cost for HS-25 load = 2 x (198,590 + 38,350) + 469,980 x 1.055 = $969,710.

The costs of designing bridge ID 1069871 for different design truck levels are shown in Table 3.23 in thousands of dollars.

Table 3.23 –Asset costs for Different HS Design Load Levels for Bridge ID 1069871 [in $1,000]

HS20	HS25	HS30	HS35	HS40	HS45	HS50	HS55	HS60	HS65
919.22	969.65	1,023.58	1,081.03	1,141.99	1,206.47	1,274.45	1,345.95	1,420.96	1,499.48

The results in Table 3.23 are used to find the cost per crossing of an overweight truck having a given moment effect. For example, the unit cost per crossing of an overweight truck having a moment effect that exceeds the HS-30 load effect is calculated based on the 741 trucks in the WIM database that exceed the effect of the HS-30 design load. To envelope the HS-30 load effect, the bridge should be designed for a HS-35 live load. Therefore, the cost is obtained as:

$$CT_{HS-30} = \frac{Cost_{HS-35} - Cost_{HS-20}}{DL \cdot NT_{HS-30}} = \frac{(1{,}081{,}030 - 919{,}220)}{75 \times 741} = \$2.91$$

The cost analysis of the overstress per crossing of Bridge ID 1069871 is performed for all the overweight trucks in the I-88 WIM data grouped based on their equivalent HS moment effect. The results are shown in Table 3.24. The first set gives the cost to upgrade the bridge design to a new HS level. The second set gives the number of overweight trucks that require the upgrade, and the third set gives the cost per truck crossing. The example assumes the following unit cost input:

Steel Unit Cost : 1.55 $/lb
Conc. Unit Cost : 9.00 $/cft
Rebars Unit Cost : 1.00 $/lb

Table 3.24 – Overstress Cost per Crossing of Bridge ID 1069871

	HS20	HS25	HS30	HS35	HS40	HS45	HS50	HS55	HS60
Bridge Cost [$1000]	919.22	969.65	1,023.58	1,081.03	1,141.99	1,206.47	1,274.45	1,345.95	1,420.96
		> HS20	**> HS25**	**> HS30**	**> HS35**	**> HS40**	**> HS45**	**> HS50**	**> HS55**
Total N. OW Trucks		27920	7460	741	107	38	16	7	3
		HS20-HS25	**HS25-HS30**	**HS30-HS35**	**HS35-HS40**	**HS40-HS45**	**HS45-HS50**	**HS50-HS55**	**HS55-HS60**
Cost per Crossing [$]		0.02	0.19	2.91	27.76	100.79	296.03	812.82	2229.96

The results in the third row of Table 3.24 show that the maximum overstress cost per crossing for vehicles that are between HS-20 and HS-25 is negligible at 2.0 cents. For vehicles whose effect is between HS-25 and HS-30, the minimum cost per crossing is 19.0 cents. The highest cost per crossing is obtained for the three vehicles that exceed the effect of the HS-55 load. For these vehicles the cost is found to be equal to $2,230.

3.6.2 Girder Fatigue Cost Allocation

When we design a bridge to envelope the strength of a truck i, we are also assuming that the truck will not cause any additional fatigue damage to the new design. However, overweight trucks whose load effect is above that of the design load will also cause a reduction in the bridge's fatigue life. The fatigue damage is obtained using the AFB factor of Eq. (3.6). Therefore, the cost per crossing of trucks that have load effects higher than the design HS can be calculated from Eq. (3.25).

$$CT_{FB} = \frac{Cost_{HS-J}}{ADTT \cdot 365 \cdot DL} \cdot AFB \tag{3.25}$$

where: CT_{FB} = Cost of fatigue damage per crossing [$]

$HS-J$ = design envelope of level below the effect of truck "i " (HS-20 to HS-60)

$Cost_{HS-J}$ = Cost of the bridge [$]

AFB = Amplification of damage from Eq. (6)

$ADTT$ = Average daily truck traffic

DL = Design life in years assumed equal to 75

This procedure is schematically presented Figure 3.9. The graph 3.9.A shows the effect of the girder fatigue for two vehicles that exceed the effect of HS-40. Both trucks will equally contribute to the costs of providing a strength envelope for HS-45. However, the reduction each of these two trucks causes to the life of the bridge whose strength envelops the HS-40 load effect is widely different depending on the magnitude of each AFB. Assuming the first truck has a ratio AFB equal to 4.3 while truck 2 has an AFB equal to 7.9, the cost of fatigue damage per crossing for each of these two trucks that exceed the effect of HS-40 is:

Truck 1:

$$CT_{FB} = \frac{Cost_{HS-40}}{ADTT \cdot 365 \cdot DL} \cdot AFB = \frac{1{,}141{,}990}{1196 \times 365 \times 75} \times 4.3 = \$0.15$$

Truck 2:

$$CT_{FB} = \frac{Cost_{HS-40}}{ADTT \cdot 365 \cdot DL} \cdot AFB = \frac{1{,}141{,}990}{1196 \times 365 \times 75} \times 7.9 = \$0.28$$

Figure 3.9.B shows two other vehicles that exceed the load effect of HS-30 where the cost of the overstress they cause is the same value obtained based on the cost of designing the bridge for the HS-35 load. But, their contribution to the cost of fatigue damage is obtained based on the AFB of each truck.

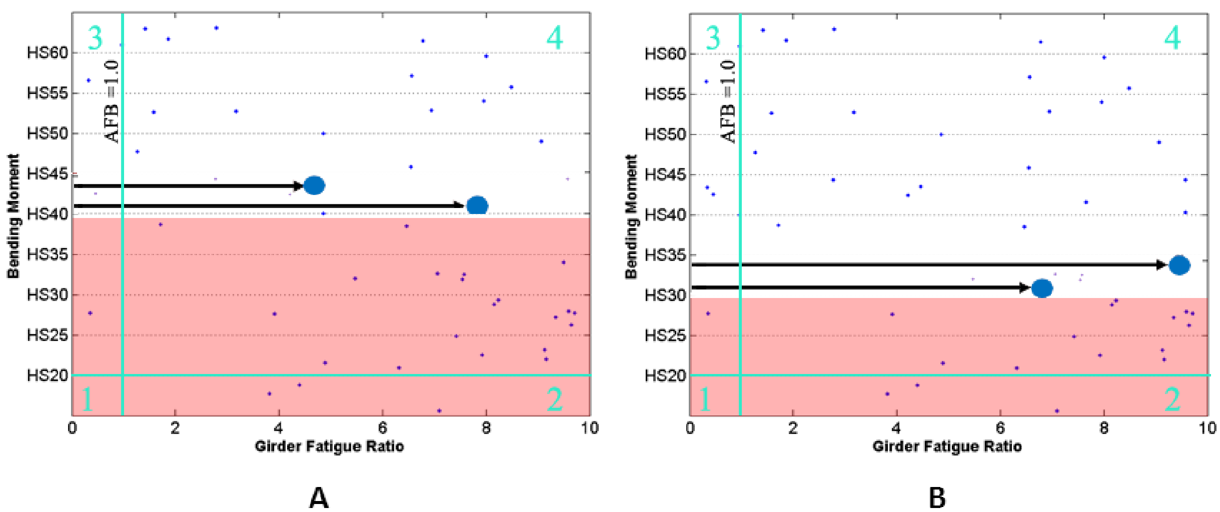

Figure 3.9 –NYS Fatigue Cost Allocation

Figures 3.9.A and 3.B also show the area of the graph can be divided into four quadrants. The lower left quadrant, labeled 1 represents the domain of the trucks with moment effect below that of the HS-20 load and a fatigue amplification AFB ratio less than 1.0. All trucks that fall in this area do not cause any damaging effect to the bridge. The trucks in the lower right quadrant labeled 2 consist of all the vehicles that have a moment effect below the effect of the HS-20 but have an AFB ratio greater than 1.0. These vehicles contribute to the reduction of the fatigue life of the bridge but do not produce any overstress. The upper left quadrant 3, groups all the vehicles that have moment effects greater than HS-20 load but have an AFB ratio lower than 1.0. This region consists of the overweight trucks that produce overstresses on the bridge but do not cause additional fatigue damage. Finally, quadrant 4 on the upper right corner represents the domain where the vehicles that overstress the bridge also contribute to a reduction in its fatigue life.

The cost of fatigue damage per crossing for each overweight truck is divided into the three overweight groups (DV), (SH), and (IL) as calculated for each NYS vehicle class. As an example, the result of the girder fatigue analysis for bridge ID 1069871 and overweight Class 9 vehicles is shown in Table 3.25. The slight differences in fatigue costs when comparing the three overweight groups is due to differences in the axle configurations of the trucks in the three overweight groups that is reflected in different amplifications of number of cycles defined as n_i in Eq. (3.6).

Table 3.25 –Girder Fatigue Cost per Crossing [$] – for Class 9 on Bridge ID 1069871

OW Type (1)	HS20-HS25 (2)	HS25-HS30 (3)	HS30-HS35 (4)	HS35-HS40 (5)	HS40-HS45 (6)	HS45-HS50 (7)	HS50-HS55 (8)	>HS55 (9)
DV	0.128	0.231	0.371	0	0	0	0	0
SH	0.130	0.235	0.428	0.699	0	0	0	0
IL	0.129	0.263	0.438	0.748	1.144	1.646	2.362	3.113

The results in Table 3.25 show that the minimum fatigue cost per crossing for vehicles with effect between HS-20 and HS-25 is equal to $0.128 for divisible trucks, while the maximum value of $0.130 is found for the special hauling vehicles. For vehicles having an effect between HS-25 and HS-30 in column 3, the cost per crossing is $0.231 for the divisible vehicles, $0.235 for special hauling vehicles and is equal to $0.263 for illegally overweight trucks. The highest cost per crossing is observed for the three vehicles that exceed the effect of the HS-55 load. For these vehicles the cost per crossing is found equal to $3.113.

By comparing Table 3.25 with Table 3.24, it is noted that the effect of the overstress is dominant when the load effects are high such as for trucks whose effects are at the HS-50 or HS-55 levels. The cost of fatigue per crossing for trucks whose effect exceeds HS-55 is $3.113 while the cost of overstress is $2,230 per crossing. The reverse is true for trucks whose effect is on the lower end of the scale. For example, the fatigue cost for trucks having effects between HS-20 and HS-25 is about $0.13 per crossing while the cost of overstress is only 2.0 cents per crossing. Although the cost per crossing is low for the low HS trucks, their large numbers may lead to higher total costs.

3.6.3 Deck Fatigue Cost Allocation

An approach similar to that described for bridge girder fatigue is also followed to obtain the cost for bridge deck fatigue. The method allocates the cost per crossing of the deck according to Eq. (3.26).

$$CT_{FD} = \frac{Cost_{Deck}}{ADTT \cdot 365 \cdot DL} \cdot AFD \tag{3.26}$$

where: CT_{FD} = Cost per crossing per truck for deck fatigue [$]

$HS-J$ = design envelope of level below the effect of truck "i" (HS-20 to HS-60)

$Cost_{Deck}$ = Cost of the deck [$]

AFD = Amplification of damage from the analysis according to Eq. (3.7)

$ADTT$ = Average daily truck traffic

DL = Deck design life in years assumed equal to 40 years

The cost per crossing for deck fatigue is divided into the three overweight groups (DV), (SH), and (IL) and calculated for each NYS vehicle class. As an example, the results of the deck fatigue cost analysis for the bridge ID 1069871 are shown in Table 3.26 for Class 9 trucks in each of the three overweight categories.

Table 3.26 – Deck Fatigue Cost per Crossing [$] for Class 9 on Bridge ID 1069871

OW Type (1)	HS20-HS25 (2)	HS25-HS30 (3)	HS30-HS35 (4)	HS35-HS40 (5)	HS40-HS45 (6)	HS45-HS50 (7)	HS50-HS55 (8)	>HS55 (9)
DV	0.004	0.005	0.006	0	0	0	0	0
SH	0.004	0.005	0.004	0.005	0	0	0	0
IL	0.004	0.005	0.006	0.007	0.008	0.009	0.01	0.01

The results in Table 3.26 show that the cost of deck fatigue per crossing is in the range of $0.004 to $0.01 depending on the HS load effect level. It is also noted that there is no significant difference between the costs associated with the three overweight classes.

3.6.4 Combined Cost Allocation

The final cost allocation combines the effects of overstress, girder fatigue, and deck fatigue. The overall cost per crossing is obtained by summing the values obtained from Tables (3.24), (3.25) and (3.26) for each HS range for each overweight category. As an example, Table 3.27 gives the combined cost per crossing of bridge ID 1069871 for Class 9 overweight trucks.

Table 3.27 – Total Cost per Crossing [$] per Year on Bridge ID 1069871

OW Type (1)	HS20-HS25 (2)	HS25-HS30 (3)	HS30-HS35 (4)	HS35-HS40 (5)	HS40-HS45 (6)	HS45-HS50 (7)	HS50-HS55 (8)	>HS55 (9)
DV	0.16	0.42	3.29	-	-	-	-	-
SH	0.16	0.43	3.34	28.46	-	-	-	-
IL	0.16	0.45	3.36	28.51	101.94	297.68	815.19	2,233.08

The combined cost per crossing is then multiplied by the number of Class 9 overweight vehicles per year in each HS group. Table 3.28 shows the number of Class 9 vehicles per year in each live load group for each type of overweight. For example, the number of divisible vehicles that have moment effects between HS20-HS25 is equal to 7,669.

Table 3.28 – Number of Class 9 Overweight Vehicles per Year on Bridge ID 1069871

OW Type (1)	>HS20 (2)	>HS25 (3)	>HS30 (4)	>HS35 (5)	>HS40 (6)	>HS45 (7)	>HS50 (8)	>HS55 (9)
DV	8227	558	1	0	0	0	0	0
SH	116	16	5	2	0	0	0	0
IL	3969	535	161	51	19	10	4	1
OW Type (1)	**HS20-HS25 (2)**	**HS25-HS30 (3)**	**HS30-HS35 (4)**	**HS35-HS40 (5)**	**HS40-HS45 (6)**	**HS45-HS50 (7)**	**HS50-HS55 (8)**	**>HS55 (9)**
DV	7669	557	1	0	0	0	0	0
SH	100	11	3	2	0	0	0	0
IL	3434	374	110	32	9	6	3	1

The total cost per truck is obtained by multiplying the combined cost per crossing in Table 3.27 times the number of trucks in Table 3.28. The combined cost for crossing of bridge ID 1069871 for all Class 9 overweight trucks (DV+SH+IL) is about $14,891 per year divided for each truck group as shown in Table 3.29.

Table 3.29 – Total Cost per Year [in $] for Class 9 Overweight Trucks on Bridge ID 1069871

OW Type (1)	HS20-HS25 (2)	HS25-HS30 (3)	HS30-HS35 (4)	HS35-HS40 (5)	HS40-HS45 (6)	HS45-HS50 (7)	HS50-HS55 (8)	>HS55 (9)	
DV	1,284.10	235.77	3.29	-	-	-	-	-	1,523.16
SH	18.34	6.82	16.72	56.93	-	-	-	-	98.81
IL	623.46	243.17	540.25	1,454.24	1,936.89	2,976.80	3,260.76	2,233.08	13,268.65
Total	1,925.90	485.76	560.26	1,511.17	1,936.89	2,976.80	3,260.76	2,233.08	14,890.62

The results of the analysis of all 22 bridges along the I-88 Corridor are summarized in Table 3.30 which gives the costs for all the overweight categories in all vehicle classes. For example, for bridge ID 1069871, the total cost for all classes is equal to $29,258 as shown in column 9. Columns 2 through 5 give the characteristics of the bridges, including the ADTT (column 2), construction cost for HS-20 design load (column 3). Colum 4 identifies whether the spans are simple (S) or continuous (C) and column 5 gives the number of spans. Columns 6, 7 and 8 provide the cost for the overweight vehicles per category for the divisible permits (column 6), special hauling (column 7) and illegal trucks (column 8). Finally columns 9 and 10 give the total overweight cost per year and the percentage of that cost relative to the HS-20 bridge construction cost. The input for the I-88 cost allocation analysis is based on the following unit costs:

Steel Unit Cost	:	1.55 $/lb
Conc. Unit Cost	:	9.00 $/cft
Rebars Unit Cost	:	1.00 $/lb
P.S.C. Unit Cost	:	50.00 $/cft
P.S.S. Unit Cost	:	1.00 $/cft

The values provided in Table 3.30 show that the combined cost due to the effect of the overstress and fatigue varies with the ADTT, the total cost of the bridge, the type of construction, and the bridge response.

Continuous bridges are associated with the highest costs because of the higher construction cost of these bridges compared to simple spans. Nevertheless, the percentage cost of overweight trucks per year does not vary significantly and remains in the range of 1.2% to 3.4% with an average of 2.5%. Although this percentage is relatively low per year, over 75 years, the overweight trucks are essentially doubling the effective cost of bridges.

Table 3.30 – I-88 Bridge Corridor Overweight Cost Effect per Year

Bridge ID (1)	**ADTT** (2)	**Bridge Cost** (3)	**Type** (4)	**N. SPANS** (5)	**DV [$]** (6)	**SH [$]** (7)	**IL [$]** (8)	**Total Cost Effect** (9)	**% of Bridge Cost** (10)
1069871	1196	919,220	S	1	4,473	706	24,079	**29,258**	3.2%
1069981	475	1,110,000	S	1	6,912	734	12,497	**20,144**	1.8%
1070141	1652	1,557,800	C	3	9,930	1,208	33,633	**44,770**	2.9%
1070151	1652	2,006,370	C	2	11,528	1,996	42,001	**55,524**	2.8%
1070162	1184	1,412,100	C	3	22,221	2,080	23,824	**48,125**	3.4%
1070571	475	1,630,860	S	1	10,315	2,040	38,172	**50,527**	3.1%
1070611	1430	910,290	S	1	4,456	706	20,464	**25,626**	2.8%
1070621	1430	3,684,010	C	3	17,430	3,203	74,652	**95,285**	2.6%
1070641	1774	1,814,860	S	1	10,494	2,212	43,513	**56,219**	3.1%
1070642	1774	2,107,620	S	1	10,476	2,398	47,200	**60,074**	2.9%
1070661	1665	3,215,480	S	1	7,763	2,438	27,168	**37,370**	1.2%
1070720	1508	1,026,820	S	1	5,497	981	26,192	**32,670**	3.2%
1070841	1196	898,590	S	1	5,931	644	14,792	**21,367**	2.4%
1070991	562	1,499,680	C	2	5,288	681	15,745	**21,714**	1.4%
1071312	887	1,093,090	S	1	6,805	745	16,345	**23,896**	2.2%
1093991	1406	4,252,140	S	4	23,110	4,161	96,693	**123,964**	2.9%
1094001	2811	2,413,420	C	3	11,015	1,395	38,894	**51,305**	2.1%
1094141	787	1,213,800	S	2	7,836	1,238	13,727	**22,801**	1.9%
1094161	1356	827,440	S	1	3,814	587	16,410	**20,812**	2.5%
1094182	1554	1,475,760	C	3	5,154	661	15,960	**21,776**	1.5%
1094341	1554	736,310	S	1	2,825	378	10,841	**14,043**	1.9%
1094961	1295	881,380	S	1	4,112	672	19,377	**24,161**	2.7%
TOTAL	-	-	-	-	**197,385**	**31,863**	**672,183**	**901,431**	

3.6.5 Summary of I-88 Bridge Cost Analysis

This Section 3.6 describes the implementation of the overweight trucks cost allocation methodology to 22 bridges along the I-88 Corridor. The cost is allocated based on the overweight trucks in each vehicle class configuration for divisible permit trucks, special hauling permit trucks and illegally overweight trucks. The cost allocation procedure involves three data analysis phases. The first phase analyzes the WIM dataset and extracts the percentages of overweight trucks. A statistical distribution of the overweight trucks into likely divisible and special hauling permit vehicles as well as illegal trucks is also performed in Phase I. As an example, using the procedure described in Chapter 2, the analysis of the WIM data for the year 2011 collected at the I-88 WIM site indicates that the overall percentage of overweight trucks in this population is about 21%. The results show that for the most common truck configurations which follow Class 9, the likelihood that an overweight truck is a divisible permit (DV) is equal to 63.5%, the likelihood that it is a special hauling permit (SH) is 0.6% and the likelihood of it being illegally overweight is equal to 35.9%. Overall, when considering all possible class configurations, the percentages of divisible, special hauling and illegal vehicles are respectively 61.1%, 1.9% and 37%.

The second Phase of the cost allocation calculates the moment load effect of each truck on each bridge along the corridor and classifies them into groups based on the moment effects of the AASHTO HS design load varying between HS-20 and HS-60. Three parameters are defined in order to quantify the cost of each overweight vehicle for each bridge. The first cost is related to the overstress induced by the bending moment of the overweight truck compared to that of the HS design load. The second cost is that related to the fatigue damage caused by each overweight truck compared to the damage of the LRFD fatigue design vehicle. The third cost defines the fatigue damage of the deck caused by each overweight truck compared to the damage from the modified AASHTO LRFD fatigue truck.

The last Phase of the methodology allocates the cost of each bridge to each overweight truck accounting for the cost of the superstructure and that of the substructure. The implementation of the procedure to the 22 bridges analyzed along the I-88 Corridor shows that the effect of overweight divisible permit trucks on the entire group of 22 bridges is $197,385 per year, the cost for the special hauling permit trucks is $31,863, and finally the cost for the illegal vehicles is $672,183 per year. The total cost of damage for the selected set of 22 bridges along I-88 Corridor is about $901,431 per year.

3.7 Cost Allocation for NYS Bridge Network

The methodology described in this Chapter 3 is implemented on a representative sample of fifty five bridges which represent the configurations and the material properties of the vast majority of bridges under the jurisdiction of New York State Department of Transportation (NYSDOT). The selected fifty five bridges represent about 14,500 simple supported span bridges and 495 continuous New York bridges. The selection of the fifty five bridges is based on the filtering process presented in Section 3.2 (and the Task 1.d Report) but updated to extract the information from WINBOLTS rather than the NBI files and to filter out bridges that are on parkways and thruways. The overweight truck database for the network analysis is obtained from sampling twenty-one Weigh-In-Motion (WIM) databases collected at different sites in New York State. The truck sample was selected based on the proportion of trucks in each of the WIM datasets.

The cost due to the effect of the overweight trucks is obtained following the methodology described in this Chapter (and Report Task 2.a/b). The procedure is divided into three phases. In the first phase, the WIM data file which is assumed to be representative of the entire network is analyzed to obtain the percentages of trucks not overweight that meet the New York state legal weight limits (LG) and overweight vehicles which are divided into the three categories Divisible (DV), Special Hauling (SH) and Illegals (IL). The categorization of the WIM data follows the procedure established in Chapter 2 (and the Report of Task 1.b). The analysis is performed for each heavy vehicle class.

In the second phase, the maximum moment response of each vehicle contained in the WIM file is calculated for each bridge by sending each truck through the appropriate influence line. Finally the truck's response is used to estimate the cost effect caused by each truck. The analysis of the effect of the overweight vehicles is executed for three types of response: 1) Overstress safety margin utilization, 2) girder fatigue damage, and 3) deck fatigue damage. The overstress cost analysis follows the classical FHWA cost allocation model. The girder fatigue damage analysis follows the AASHTO LRFD fatigue analysis method. The deck fatigue damage follows the model proposed by Perdikaris et al (1993) as described in Section 3.4. The overweight truck load effects are divided into three categories: a) effects caused by divisible permit trucks (DV), b) effects caused by special hauling permits (SH) and 3) effects caused by illegal overweight trucks (IL). The total network cost for each overweight category is then obtained by scaling the effect of the trucks in the representative WIM dataset by the statistics of DV, SH, and IL vehicles estimated for the entire state.

This Section 3.7 of the Final Report is divided into sub-sections that address the following topics: Bridge Sampling, WIM Analysis, WIM sampling method, WIM statistics, Network Cost Analysis, and Conclusions. The cost models adopted in the cost allocation study are described in Section 3.5.

3.7.1 Bridge Sampling

The most significant New York State bridge categories were extracted from the NBI files as reported in column (2) of Table 3.31. These bridges were filtered out from the data that include all bridges in the NBI file. The list was further filtered using WINBOLTS and also by eliminating bridges on parkways and thruways to consider only the bridges in the NYSDOT database. The modified updated list is provided in column (3) of Table 3.31.

Table 3.31 – Significant Bridge Categories in New York State Bridge Inventory

Significant Categories (1)	**NBI files** (2)	**WINBOLT** (3)
One Span Concrete Slab	440	415
One Span Concrete Tee Beam	88	81
One Span Steel Stringer/Multi-beam or Girder	5,073	3,597
Two Span Steel Stringer/Multi-beam or Girder	689	503
Three Span Steel Stringer/Multi-beam or Girder	925	769
Four Span Steel Stringer/Multi-beam or Girder	503	398
Multi-Span Steel Stringer/Multi-beam or Girder	575	360
One Span Steel Girder and Floor-beam System	284	203
Two Span Steel Continuous Stringer/Multi-beam or Girder	581	214
Three Span Steel Continuous Stringer/Multi-beam or Girder	340	146
Four Span Steel Continuous Stringer/Multi-beam or Girder	139	69
Multi Span Steel Continuous Stringer/Multi-beam or Girder	173	66
One Span Prestressed Concrete Stringer/Multi-beam or Girder	506	357
One Span Prestressed Concrete Box Beam or Girders-Multiple	1,567	1,799
Total	**11,883**	**8,977**

According to Table 3.31, the number of simple supported bridges is equal to 8,482. Some of these simple span bridges may have multiple simple spans that add up to a total number of 14,470 simple spans as shown in Table 3.32. The total number of continuous bridges shown in Table 3.31 is equal to 495.

Each of the categories listed in Table 3.31 is further divided into groups based on the bridge's span length in increments of 20 ft, ranging from 0 through 260 ft. The span-based grouping for the simply supported bridges is shown in Table 3.32 for the different structural types where column (3) lists the number of spans for each bridge type and span length range. A representative simple bridge is randomly selected to have the characteristics of the bridges in each row of Table 3.32. Thus, the number of simple supported bridges analyzed is equal to 27.

A similar grouping is performed for the continuous bridges and the results are shown in Table 3.33. A special consideration of the effects of adjacent spans must be included for the continuous bridges. This is because each truck traveling on a continuous span produces a moment effect on all the spans of the bridge and the constraints from all the spans affect the one span being analyzed. Because the response is mostly affected by the spans adjacent to the span being analyzed, just three spans are considered in each group. Specifically, the loaded span is considered in a group that includes the adjacent preceding and following spans. As an example, Figure 3.10 shows the set of configurations evaluated for a hypothetical five-span bridge.

Table 3.32 – Simple Span Bridges – Span Distribution

Significant Category (1)	**Span Range** [ft] (2)	**Number of Spans** (3)
Concrete Slab	0-20	358
	20-40	47
	40-60	10
	Total	**415**
Concrete Tee Beam	0-20	60
	20-40	21
	Total	**81**
Prestressed Concrete Box Beam or Girders	20-40	608
	40-60	553
	60-80	355
	80-100	244
	100-120	32
	120-140	7
	Total	**1,799**
Prestressed Concrete Stringer/Multibeam or Girder	40-60	135
	60-80	89
	80-100	61
	100-120	70
	120-140	2
	Total	**357**
Steel Stringer/Multibeam or Girder	0-20	398
	20-40	2595
	40-60	3154
	60-80	2230
	80-100	1464
	100-120	1033
	120-140	606
	140-160	223
	160-180	65
	180-200	32
	200-220	18
	Total	**11,818**
Global Total		**14,470**

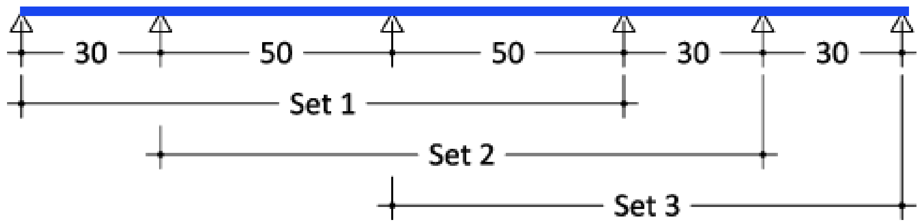

Figure 3.10 – 5-Span Continuous Bridges – Span Set Configurations

Figure 3.10 shows a five span continuous bridge with 30, 50, 50, 30 and 40 ft spans respectively. The number of consecutive span sets for this bridge is three as shown in the figure. The first set of spans labeled “Set 1” has three spans that are 30, 50 and 50 ft long. “Set 2” has span lengths equal to 50, 50 and 30 ft. Finally, “Set 3” has a three span configuration with span lengths equal to 50, 30 and 50 ft.

The most predominant continuous span configurations in New York State are listed in Table 3.33. The table shows that the significant numbers of continuous bridges are steel girder bridges as listed in column (1). Column (2) gives the span ranges for the first span from the left. Column (3) gives the span length for the main span and column (4) gives the span length for the third span. The dash indicates that the main span being analyzed is an end span. By symmetry, end spans at the end of the bridge or at the beginning of the bridges are counted together. Table 3.33 only includes the configurations with a count greater than ten. The total number of configurations is 922 as shown in the last row of Table 3.33 and this number is 60% of the total 1,562 configurations for the 495 continuous bridges described in Table 3.32. Given the low frequency of the remaining 40% of configurations and the low percentage of the continuous bridges with respect to the simple supported bridges (5.5% = 495/8,977), the effect of these bridges is not directly calculated in this study. But, their cost is estimated by scaling the results of the 922 configurations (28 bridges). This is done because the computational time required to analyze 300 bridges with all possible configurations is prohibitive.

Because the types of permits allowed in upstate counties are different than those in downstate counties, the cost allocation analysis of New York State bridges is further divided into two groups. According to the WINBOLTS database, and the definition of upstate and downstate for truck permit purposes as reported on the official document “Permit69” of the NYSDOT, the percentage of bridges located in upstate New York is about to 85% (7,630 bridges), while the remaining 15% (1,347 bridges) are distributed downstate. Accordingly, in the analysis, these two proportions are used to split the number of bridges in each group between downstate and upstate.

Table 3.33 – Continuous Span Bridges – Span Distribution

Significant Category (1)	**Left Span Range** [ft] (2)	**Middle Span Range** [ft] (3)	**Right Span Range** [ft] (4)	**Number of Configurations** [ft] (5)
Steel Continuous	-	0-20	0-20	19
	-	0-20	80-100	13
	-	20-40	20-40	23
	-	20-40	40-60	31
	-	20-40	60-80	21
	-	40-60	40-60	49
	-	40-60	60-80	71
	-	40-60	80-100	17
	-	60-80	60-80	35
	-	60-80	80-100	22
	-	60-80	100-120	14
	-	80-100	80-100	69
	-	80-100	100-120	49
	-	100-120	100-120	96
	-	100-120	140-160	27
	-	120-140	120-140	60
	-	120-140	140-160	20
	-	120-140	160-180	20
	-	140-160	140-160	50
	0-20	0-20	0-20	53
	20-40	40-60	40-60	21
	20-40	40-80	40-80	13
	40-60	40-60	40-60	30
	40-60	60-80	40-60	11
	40-60	60-80	60-80	45
	60-80	60-80	60-80	11
	80-100	80-100	80-100	19
	80-100	100-120	80-100	13
Total				**922**

In summary, the distribution of simple and continuous bridges around New York State is performed to include the most significant types of bridges. The distribution shows that 8,977 bridge types are predominant within the NYSDOT database. Out of these 8,977 bridges 8,482 (94.5%) are simple supported bridges, while 495 (5.5%) are continuous bridges. According to the span distribution, 27 categories of bridges can adequately represent the entire simple supported population, while another 28 continuous bridges describe 60% of the entire continuous population. Given the low frequency of the remaining configurations and the low percentage of continuous bridges in the entire bridge network, the effect of the omitted continuous bridges is inferred using statistical projections. This is done because of the prohibitive computational time required to analyze all possible configurations.

3.7.2 WIM Data Analysis

The analysis of the network cost effect performed in this study is based on the truck weight data collected at 21 WIM sites outside New York City during the year 2011. Figure 3.11 shows the distribution of the WIM stations around the state. Also, the map shows downstate counties colored in dark shade. According to the document "Permit69", the seven downstate counties are Dutchess, Orange, Putnam, Westchester, Rockland, Nassau, and Suffolk. It is noted that some of the WIM station ID numbers shown in Figure 3.11 may have been changed over time and may not be consistent with the ID numbers provided by the NYSDOT IT Department as listed in Table 3.34.

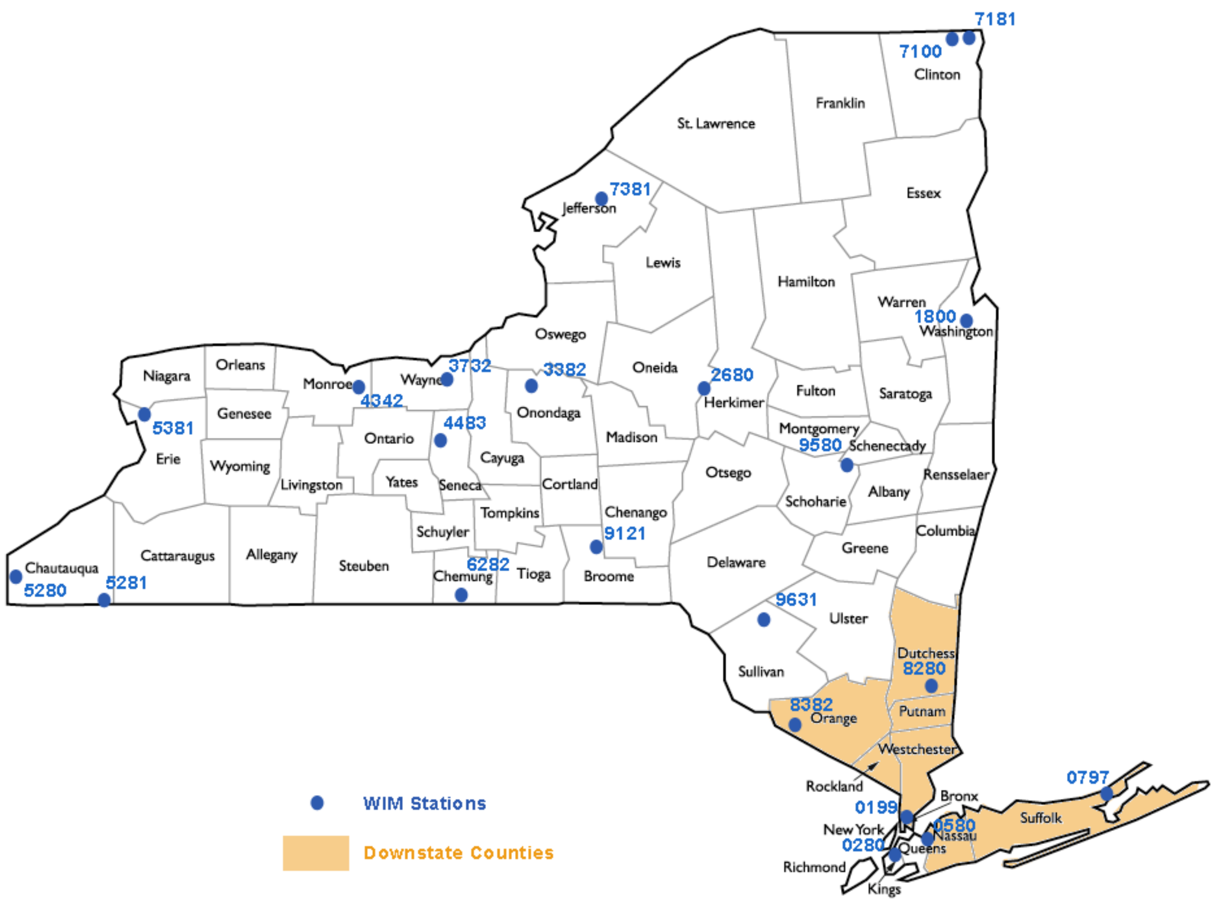

Figure 3.11 – NYS WIM Stations

Table 3.34 – 2011 WIM Datasets and Sample Size

WIM Dataset (1)	Number of Vehicles (2)	Number Overweights (3)	Overweights [%] (4)	Permit [%]			Sample Proportion OW [%] (8)	Sample Size (9)	Effective Sample (10)
				DV (5)	SH (6)	IL (7)			
1281	27,221	5,226	19.2	57.4	2.1	40.5	0.3	39,599	5,226
1400	652,304	121,329	18.6	64.2	2.5	33.3	5.8	1,610	1,610
1800	305,100	68,953	22.6	60.1	2.6	37.3	3.3	2,909	2,909
2680	154,740	37,757	24.4	59.1	3.2	37.7	1.8	5,395	5,395
3311	1,225,061	225,411	18.4	66.3	1.8	31.9	10.9	820	820
4342	477,552	67,812	14.2	51.1	4.3	44.6	3.3	2,959	2,959
4483	113,220	18,455	16.3	52.8	2.7	44.5	0.9	11,142	11,142
5183	467,485	99,107	21.2	65.3	1.9	32.8	4.8	1,993	1,993
5281	114,761	30,297	26.4	69.5	1.0	29.5	1.5	6,748	6,748
6282	67,350	14,346	21.3	58.1	2.8	39.1	0.7	14,362	14,346
6340	107,884	21,685	20.1	65.6	2.1	32.3	1.0	9,467	9,467
6482	822,958	142,372	17.3	64.9	1.6	33.5	6.9	1,357	1,357
7100	454,588	105,919	23.3	66.2	1.8	32.0	5.1	1,859	1,859
7181	149,752	39,834	26.6	63.6	2.2	34.2	1.9	5,108	5,108
7381	273,144	33,870	12.4	66.4	1.1	32.5	1.6	6,025	6,025
9121	1,149,657	174,748	15.2	63.4	1.9	34.7	8.4	1,087	1,087
9580	561,431	117,901	21.0	66.0	2.0	32.0	5.7	1,660	1,660
9631	226,993	57,883	25.5	60.1	2.1	37.8	2.8	3,484	3,484
0580	1,255,437	306,327	24.4	41.8	10.6	47.6	14.8	577	577
8280	1,733,022	202,764	11.7	62.7	2.8	34.5	9.8	923	923
8382	1,277,280	182,651	14.3	59.5	2.1	38.4	8.8	1,036	1,036
Total	**11,616,940**	**2,074,647**	**-**	**-**	**-**	**-**	**100.0%**	**120,120**	**85,731**

The analysis of the WIM data files following the procedure described in Chapter 2 (and Report of Task 1.b) led to the number of vehicles, number of overweight trucks, and permit distributions shown in Table 3.34. Column 1 of Table 3.34 gives the WIM station Identification Number (ID); column 2 indicates the total number of vehicles in the dataset. Column 3 gives the number of overweight vehicles. The values reported in column 4 show the percentage of overweight vehicles in the dataset. Columns 5, 6 and 7 indicate the percentages of overweight trucks estimated to be divisible (DV), special hauling (SH) and illegal (IL). The last three columns of Table 3.34 contain information about the sampling of vehicles used to build the network WIM database. The details of the sampling procedure are described in the next section. The three rows in Table 3.34 corresponding to WIM datasets collected in downstate counties are highlighted in a darker color. The last row gives the totals from all the WIM sites.

The results in Table 3.34 along with additional analyses show that the numbers of overweight trucks that exceed any of the NYSDOT weight limit criteria vary depending on the location of the station and the vehicle class. It is noted that overweight trucks are defined as those that exceed the 80,000 lb federal imposed gross vehicle weight (GVW) limit as well as those that have axles or axle groups that exceed the NYSDOT criteria which include meeting the Federal Bridge Formula (FBF) for trucks above 71,000 lbs.

The analysis of the data shows that on the average about 18% of the total truck population are overweight. It is estimated that 11% of the trucks travelling on New York highways may be carrying divisible load permits, 1% may be carrying special hauling permits while about 6% may be illegally overweight.

3.7.3 Proportional Sampling for Overweight Trucks

Because it is not possible to identify which truck in the WIM database crosses which bridges or how many times it crosses a specific bridge, a statistical sampling procedure must be used for the analysis of the New York state bridge network. The total number of overweight vehicles in all WIM files is found to be 2,074,647 in one year. The structural analysis of all these records for all the bridges is not possible given the huge amount of computational time that this will require. Therefore, the first task of this analysis is to create a representative sample of trucks that would be assumed to represent the overweight truck population for all the bridges over the entire state. A sampling procedure is used for this purpose by randomly selecting sets of trucks from each WIM file based on the number of trucks in the file compared to the total number of trucks. Column 8 of Table 3.34 gives the proportion of trucks in each WIM set that are extracted for use in the network analysis compared to the total number of vehicles recorded for that WIM site. The truck sample size, n, that is sampled from each WIM site is obtained using the classic sampling method as described by Lapin (1983) based on the total population size, N_t.

The expected percentage of trucks that should be sampled from one site E(P) and the standard deviation σ_p of the proportion are presented as:

$$E(P) = \pi$$
$$\sigma_P = \sqrt{\frac{\pi(1-\pi)}{n}} \qquad (3.27)$$

where $\pi = \frac{n_s}{N_t}$, σ_P is the standard error of the proportion and n_s is the size of the particular WIM file. Assuming a value for the standard error, it is possible to obtain the sample size as shown in Eq. (3.28):

$$n = \frac{\pi(1-\pi)}{\sigma_P^2} \qquad (3.28)$$

Column 9 in Table 3.34 shows the values for the sample size, n, based on the proportions for each WIM site assuming a Coefficient Of Variation (COV) equal to 10% where COV = σ_p/E(P).

For the assumed COV = 10%, the total number of vehicles to sample is equal to 120,120 vehicles. Column 9 of Table 3.34 lists the number of trucks required from each site based on Eq. (3.28). It is noted that for certain WIM files, where the proportion of overweight trucks is small, the number of vehicles required for sampling may exceed the actual number of overweight trucks in the file. In that case, the entire population of overweight vehicles is included. The final number of trucks sampled from each site is given in column 10 of Table 3.34. This will create a slight bias in the selected data set toward the overweight vehicles by slightly increasing their proportions in the truck sample.

Based on the sampling procedure outlined above, a total of 85,731 vehicles are extracted for this analysis of the network. This sample provides a good estimate of the mean structural response for all WIM sites when tested on a representative number of bridges extracted from the bridge network. The analysis of the trucks is also divided per truck class to verify that the sampling procedure adequately accounts for the effect of the truck classes as well as the overweight categories.

As an example, Figure 3.12 shows the cumulative distribution of the moment response of a simple supported bridge having a span length equal to 90 ft. The moment response is for all the overweight vehicles in each of the WIM files divided by class. The results are plotted in different shadings of blue for each WIM file. To verify the validity of the sampling process, a random sample of trucks is extracted from each WIM file based on the numbers given in Column 10 of Table 3.34. This random sample is then divided into classes and the moment effects of the sampled trucks are calculated for the 90-ft simple span bridge. The results are plotted in red in Figure 3.12.

Figure 3.12 demonstrates that all the WIM files show similar results and that the sampled trucks provide a good representation of the response of the entire WIM truck population. As expected, the only class where there is large variability between the WIM files is for the trucks in Class 14. This is because this Class 14 encompasses all the vehicles that the algorithm could not place in any of the standard vehicle classes. The trucks in Class 14 do have a wide range of unusual configurations.

The same analysis was performed for a total of 22 bridges showing similar trends. Appendix A of the Task 3.b Report contains the plots of the cumulative moment responses for the 22 representative simple supported bridges analyzed using the same method. These analyses confirm the validity of using the sampled trucks for the analysis of the entire bridge network.

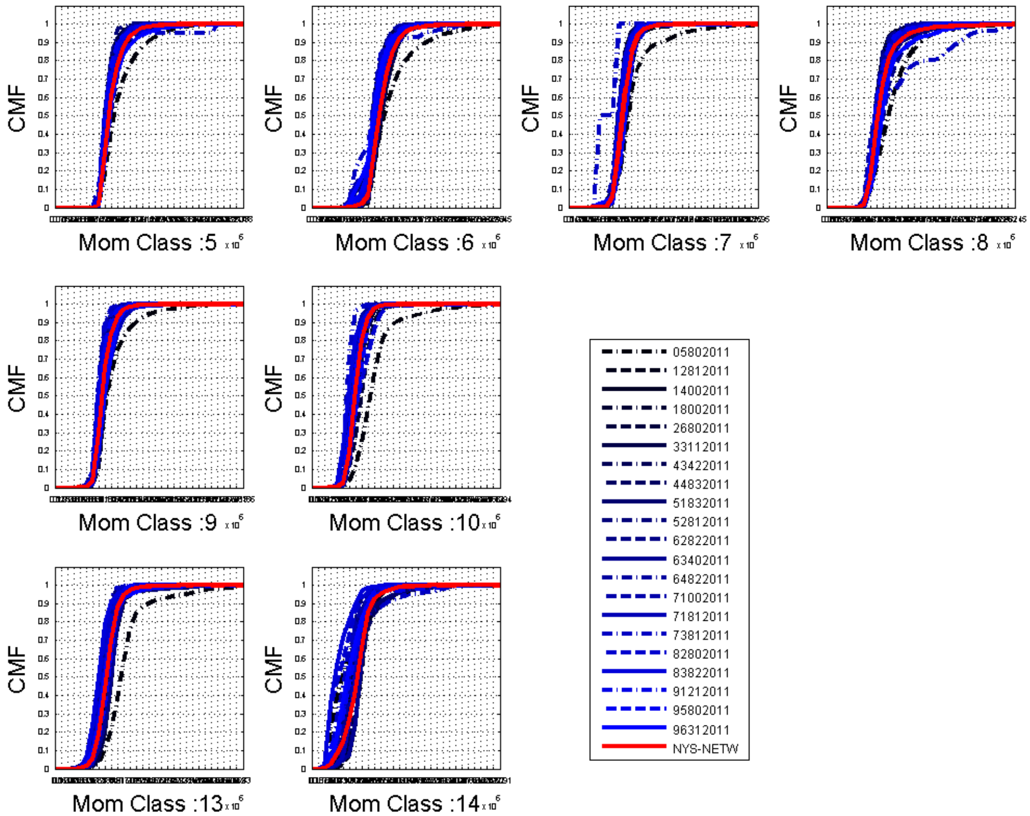

Figure 3.12 – Cumulative Distribution of Moment Response of 90 ft Simple Supported Bridge – WIM Comparison

3.7.4 WIM Statistics in NYS Network

As noted in the previous sections, a statistical analysis is necessary to quantify the effect of overweight vehicles on the New York bridge network. The analysis is executed for overstress, girder fatigue, and deck fatigue. The cost is based on the models described in Section 3.5 of this Report and Task 2.a/b report).

The verification of the approach is executed on a small sample of bridges as a test of the statistical analysis process. At a later stage, the process is extended to the entire network. The testing involves the following steps:

- Select a representative sample of 22 bridges for a preliminary analysis.
- Divide the WIM sites into upstate and downstate sites.
- Run each of the WIM files on all the bridges to find the moment effect of each truck on each bridge.
- Separate the effect of the trucks on each bridge based on truck overweight categories DV, SH, IL and all vehicle classes.
- Identify the percentage of overweight trucks in each category that cause damage.
- Perform a cost analysis for each bridge following the procedure described in this Chapter (and Task 2.a/b Report).
- Establish the cost per truck for only those overweight trucks that cause damage.
- Summarize the results and present them in terms of cost per truck per class per overweight category.

The results of the analysis are presented in Tables 3.35 and 3.36. The results are divided into 4 groups based on the total number of trucks per year, the percentage of those that are overweight, the different categories of the overweight trucks, and the number of overweight trucks that contribute to the costs. These groups are presented in Table 3.35 for an average upstate bridge and in Table 3.36 for an average downstate bridge. The results are divided into the usual 10 vehicle classes (Class 5 through 14). As an example, an average bridge in upstate New York will be crossed by about N_9=144,559 Class 9 trucks per year. The sum of all the classes leads to an Average Daily Truck Traffic (ADTT) for an average upstate bridge of 571 trucks per day. For downstate bridges, the average ADTT is about 1,606.

For upstate bridges, about p^{ow}_9 = 19% of the Class9 trucks are overweight. It is estimated that p^{DV}_9 = 68% of the overweight trucks are divisible permits (DV), p^{SH}_9 = 0.7% are special hauling permits (SH) and p^{IL}_9 = 31.2% are illegal (IL). The percentage of overweight divisible permit trucks that contribute to the cost is only p^{DVE}_9 = 17.1%. These percentages vary widely depending on the vehicle class and overweight category as shown in Table 3.35 for upstate bridges. Similar results are assembled in Table 3.36 for downstate.

Summarizing, the average number of divisible, special hauling and illegal vehicles that produce a cost effect on the bridge network per each vehicle class can be obtained from Eq. (3.29).

$$X_i = N_i \times p_i^{ow} \times p_i^{x} \times p_i^{xE} \tag{3.29}$$

Where X_i represents the number of trucks per overweight category DV, SH, or IL for class i, which is obtained by multiplying the total number of trucks N_i in class i, times the percentage of overweight vehicles $p^{ow}{}_i$ in class i, times the percentage of DV, SH or IL $p^{X}{}_i$ in each class, times the percentage of vehicles DV, SH, or IL $p^{xE}{}_i$, that contribute to the cost of the bridge.

As an example column 5 of Table 3.35 shows the number of Class 9 vehicles and their percentages. According to Eq. (3.29), the average number of divisible load vehicles that cause a cost effect on an upstate bridge is equal to DV_9 = 144,559 x 0.19 x 0.68 x 0.171 = 3,194 per year. Similarly, the number of Class 9 special hauling trucks is SH_9 = 144,559 x 0.19 x 0.007 x 0.209 = 40 per average upstate bridge per year. Finally, the number of illegal Class 9 trucks on an average upstate bridge is IL_9 = 144,559x 0.19 x 0.312 x 0.194 = 1,662 per bridge per year.

A similar calculation is performed for each class of vehicles to estimate the number of overweight vehicles that produce a cost effect for the entire network. This is done by multiplying N_i by the number of spans in upstate New York or downstate New York as appropriate. Accordingly, the total number of overweight vehicles in upstate counties that are included in the cost analysis is estimated to be about 11.0 million yearly-truck-bridge span, while this number is equal to 30.1 million for downstate counties. The values in Tables 3.35 and 3.36 and the representative WIM sample are used to estimate the cost effect on a set of bridges representative of the NYS network.

Table 3.35 – Upstate Overweight Vehicles Statistics

TOTAL NUMBER OF HEAVY VEHICLES										
	Cls 5	Cls 6	Cls 7	Cls 8	Cls 9	Cls 10	Cls 11	Cls 12	Cls 13	Cls 14
	18,331	12,602	3,023	10,834	144,559	9,652	3,181	915	1,354	1,726
PERCENTAGES OF OVERWEIGHT VEHICLES										
	Cls 5	Cls 6	Cls 7	Cls 8	Cls 9	Cls 10	Cls 11	Cls 12	Cls 13	Cls 14
	2.5%	10.8%	68.1%	4.1%	19.0%	52.3%	3.4%	4.8%	87.9%	37.4%
PERCENTAGES OF PERMIT VEHICLES										
	Cls 5	Cls 6	Cls 7	Cls 8	Cls 9	Cls 10	Cls 11	Cls 12	Cls 13	Cls 14
DV	0.2%	9.5%	71.1%	12.8%	68.0%	67.0%	0.0%	0.0%	49.4%	32.3%
SH	36.6%	1.3%	0.2%	24.3%	0.7%	2.3%	0.0%	0.0%	17.2%	10.1%
IL	63.2%	89.2%	28.7%	63.0%	31.2%	30.7%	100.0%	100.0%	33.4%	57.6%
PERCENTAGES OF OVERWEIGHT TRUCKS CAUSING COST EFFECT										
	Cls 5	Cls 6	Cls 7	Cls 8	Cls 9	Cls 10	Cls 11	Cls 12	Cls 13	Cls 14
DV	1.1%	100.0%	68.7%	9.6%	17.1%	63.0%	0.0%	0.0%	77.1%	87.1%
SH	2.2%	7.2%	38.2%	0.6%	20.9%	100.0%	0.0%	0.0%	78.6%	41.3%
IL	2.5%	20.0%	49.3%	6.1%	19.4%	56.7%	8.2%	8.9%	74.2%	52.8%

Table 3.36 – Downstate Overweight Vehicles Statistics

TOTAL NUMBER OF VEHICLES										
	Cls 5	Cls 6	Cls 7	Cls 8	Cls 9	Cls 10	Cls 11	Cls 12	Cls 13	Cls 14
	82,286	34,036	3,212	36,147	374,891	15,381	24,985	8,639	1,342	3,489
PERCENTAGES OF OVERWEIGHT VEHICLES										
	Cls 5	Cls 6	Cls 7	Cls 8	Cls 9	Cls 10	Cls 11	Cls 12	Cls 13	Cls 14
	8.9%	21.6%	79.5%	7.6%	15.8%	78.4%	4.3%	5.2%	88.5%	26.9%
PERCENTAGES OF PERMIT VEHICLES										
	Cls 5	Cls 6	Cls 7	Cls 8	Cls 9	Cls 10	Cls 11	Cls 12	Cls 13	Cls 14
DV	0.2%	15.7%	73.3%	5.0%	63.2%	63.8%	0.0%	0.0%	38.9%	3.6%
SH	53.5%	0.7%	0.1%	26.3%	0.9%	2.9%	0.0%	0.0%	24.4%	12.9%
IL	46.3%	83.6%	26.7%	68.6%	35.9%	33.3%	100.0%	100.0%	36.7%	83.4%
PERCENTAGES OF OVERWEIGHT TRUCKS CAUSING COST EFFECT										
	Cls 5	Cls 6	Cls 7	Cls 8	Cls 9	Cls 10	Cls 11	Cls 12	Cls 13	Cls 14
DV	14.2%	100.0%	71.6%	5.1%	15.5%	86.1%	0.0%	0.0%	99.5%	49.2%
SH	3.4%	12.4%	53.8%	1.0%	36.1%	53.9%	0.0%	0.0%	54.5%	43.0%
IL	19.1%	23.8%	73.1%	24.2%	30.1%	85.3%	27.6%	24.8%	92.1%	31.7%

3.7.5 Network Cost Analysis

Testing of WIM Sampling Methodology

The cost allocation method for overstress adopted in this study is similar to the approach followed in the FHWA cost allocation study. The approach is based on evaluating the cost associated with increasing the bridge design live load from the minimum HS-20 value required according to the LFD criteria to an HS value that envelops the effect of different levels of overweight trucks accounting for the overstress caused by the overweight trucks. The overall cost analysis also accounts for the additional fatigue effect of overweigh trucks to the bridge members and bridge decks. It should be emphasized that because of the presence of overstrength capacity, most overweight trucks do not lead to bridge collapses but reduce the existing safety margin. For this reason, the cost allocation approach followed in this study is identified as "Safety Margin Utilization (SMU) cost".

The procedure consists of calculating the moment response and the fatigue damage effect caused by each vehicle in the representative WIM sample on each bridge listed in Tables 3.32 and 3.33. This cost effect, which is related to the Safety Margin Utilization (SMU) factors as described in Section 3.4 (and Task 2.a/b report), is used to obtain the average overweight truck cost per crossing for the trucks that increased the risk to the traveling public by contributing to the safety margin utilization. The cost per bridge crossing is subsequently multiplied by the number of overweight vehicles in Tables 3.35 and 3.36 and the total number of bridges as grouped in Tables 3.32 and 3.33 that have similar characteristics to those of the analyzed bridges. This approach is used because the representative WIM sample produces on the average a moment response spectrum that is comparable to the ones generated by each WIM dataset taken separately. However, the number of overweight trucks that contribute to the cost may be different than that obtained by the entire WIM dataset and the adjustment in the percentage of the last groups in Tables 3.35 and 3.36 needs to be taken into consideration.

The accuracy of the procedure is tested on a sample of 22 simple supported bridges that represent more than 95% of the total number of bridge configurations selected for the network analysis statewide. This sample is shown in Table 3.37. It consists of 10 steel girder bridges (ST), 6 prestressed concrete multi-beam boxes (PB), 3 concrete slab (CS), 2 concrete T-beams (CT) and 1 prestressed concrete girder bridges (PS) as identified in WINBOLTS. The first column in Table 3.37 gives the bridge identification number. Column 2 indicates the type of the bridge. The third column shows the span length in feet, while column 4 lists the number of similar bridge spans in the database. The total number of spans in this sample is 13,908 and this value represents 96.1% (13,908/14,470) of the entire simple supported bridge population included in this study.

Table 3.37 – Simple Supported Bridge Sample

Bridge ID	Type	Span Length [ft]	N. of Similar Bridge Spans
1000420	PB	85	244
1000610	CS	27	358
1003940	CT	30	60
1004500	ST	32	2,595
1004740	ST	130	606
1005642	ST	90	1,464
1007880	PB	38	611
1007920	PS	48	135
1011610	PB	105	32
1015860	CS	20	47
1020560	PB	69	355
1029370	ST	197	32
1068229	CT	38	21
1070472	ST	109	1,033
1091280	PB	20	608
1092191	ST	147	223
1092269	ST	167	65
3201220	ST	49	3,154
3305380	ST	69	2,230
3317470	PB	123	7
3370630	ST	205	18
5510119	CS	43	10
Total Number of Bridge Spans			**13,908**

ST = Steel Girder
PB = Prestress Concrete Box
PS = Prestress Concrete Girder
CS = Concrete Slab
CT = Concrete T-Beam

Figure 3.13 shows the comparison between the total cost of the overweight vehicles for each bridge in the sample as generated by the representative WIM sample (NYS-WIM) and the mean value obtained if the analysis is performed for each of the 21 WIM datasets independently. The figure shows good agreement for each of the 22 bridges analyzed. This confirms the validity of the WIM sampling approach adopted and the projection of the results to the entire WIM dataset.

The total cost for the 22 bridges using the representative state WIM sample is equal to $629,055, while the mean cost when the individual WIM sets are used is equal to $631,442. The error is less than 0.3%. After performing many similar analyses with different truck samples, the average error was typically less than 10%. It is noted, that while the average value is robust, some variation in the results is expected for each WIM site. Specifically, it was found that the coefficient of variation for site-to-site differences (COV) is about 35%. This variation is due to the changes in the extreme weights of the overweight trucks in each WIM site and their numbers. The number of extremely heavy vehicles is a very important parameter in the cost analysis process. For this reason, the generated WIM sample that is used for the network analysis should not be used when analyzing the bridges along a particular corridor. Instead, the WIM data collected at a station located on the corridor should be used.

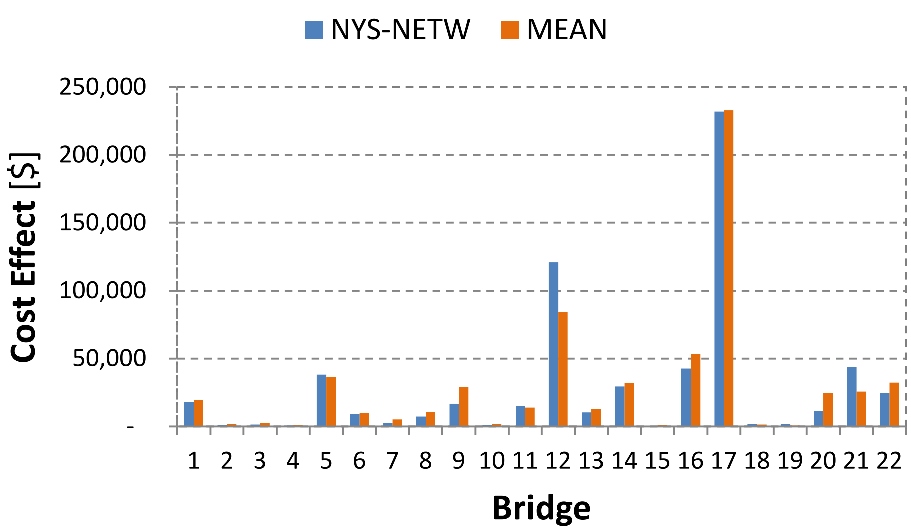

Figure 3.13 – Comparison of Costs from Network Truck Sample to Average of 21 WIM Data Sets

Implementation on Entire New York State Bridge Network

When the analysis is extended to the entire bridge network using the statistical breakdown shown in Tables 3.32 and 3.33, the results show that the total cost for upstate counties is equal to $58.5M. This cost includes the effects of divisible, special hauling and illegal trucks. The effect of the divisible vehicles is equal to $9.0M, while the cost of the special hauling vehicles is equal to $1.5M. The major contribution to the safety margin utilization cost is due to the illegal vehicles that account for $48.0M. The cost analysis is based on the following unit cost input in the NYS Bridge Cost Allocation Program: $8/ft^3 for concrete $1/lb for steel rebars, $2/lb for structural steel, $50/ft^3 for prestressed concrete and $1.5/lb for prestressing strands.

A similar cost breakdown is also executed for downstate counties. The cost of divisible vehicles is equal to \$2.0M per year while the effect of the special hauling trucks is as low as \$0.5M. The contribution to the downstate cost due to the illegal overweight vehicles is equal to \$34.0M. The total cost effect of the overweight trucks in downstate counties is equal to \$36.5M per year.

The total cost for the entire New York state bridge network is equal to \$95M per year. This estimate does not account for permits that are traveling in the counties of New York City, because they are managed by the New York City Department of Transportation.

The average unit cost per year per truck crossing in upstate and downstate counties can be obtained by dividing the total cost by the total number of overweight vehicles in upstate and downstate counties. According to Tables 3.35 and 3.36, the total number of upstate and downstate overweight vehicles is found to be 11.0M and 30.1M per year respectively. The average cost per truck is \$5.29 per truck crossing per upstate bridge and \$3.15 per truck crossing per downstate bridge.

3.8 Conclusions

This Chapter described a procedure to analyze the cost effect of overweight vehicles on the New York State Bridge network. The procedure is applied for the cost analysis of a single corridor where specific bridges are analyzed based on truck WIM data collected at a representative station located on the corridor. The same approach is also applied to estimate the cost of overweight trucks on the entire network of New York state bridges. In this case, the methodology uses a statistically representative sample of bridges and a statistically representative sample of vehicles and extends the results to the entire state's population of trucks and bridges. The analysis also divided the results into cost for upstate and downstate counties.

The distribution of short to medium length bridges, with maximum span length less than 250 ft having simple and continuous span configurations that are under in the New York State Department of Transportation Database is equal to 8,977 bridges. Out of these 8,977 bridges, 8,482 (94.5%) are simple supported, while 495 (5.5%) are continuous bridges. According to the span distribution, 27 bridge configurations describe the entire simple supported population, while 28 bridge configurations describe 60% of the entire continuous bridge population. The effects of the remaining continuous bridges are inferred using statistical tools. This is done to reduce the prohibitive computational time required if the analysis is performed for all possible configurations. The effect of the long span bridges is not considered in this analysis because the loading of long span bridges is usually dominated by the dead weights and to a lesser extent by trains of trucks. Therefore, the effects of individual overweight trucks on long span bridges are not significant.

The bridge network cost allocation analysis performed in this Chapter is based on the following unit costs: \$8/ft^3 for concrete \$1/lb for steel rebars, \$2/lb for structural steel, \$50/ft^3 for prestressed concrete and \$1.5/lb for prestressing strands. Based on these unit costs, the total cost due to overweight trucks for upstate bridges is estimated to be \$58.5M per year. This cost accounts for the effects of all overweight vehicles including Divisible and Special Hauling permitted trucks as well as illegally overweight trucks. The effect of the divisible vehicles is equal to $9.0M per year, while the cost of the special hauling vehicles is equal to $1.5M. The major contribution to the safety margin utilization cost is due to the illegal vehicles that account for $48.0M. In downstate counties, the cost of divisible vehicles is equal to $2.0M per year while the effect of the special hauling trucks is as low as $0.5M. The cost due to the illegal overweight vehicles is equal to $34.0M. The total cost effect of the overweight trucks in downstate counties is equal to $36.5M per year. The total cost for the entire New York state bridge network is equal to $95M per year. The average cost is $5.29 per truck crossing of a typical upstate bridge and $3.15 per truck crossing of a downstate bridge.

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Chapter 4

Modeling the Effects of Overweight Trucks on New York State Pavements

4.1 Introduction

4.1.1 Background

The most commonly used method to allocate the costs of damage to pavements due to overweight trucks is based on the incremental cost approach. The incremental approach estimates the cost to upgrade the pavement thickness that would be required to accommodate higher truck weights while keeping the design life constant. This approach requires a comparison between two scenarios: the cost of assessing the pavements under the effect of current loads including overweight trucks compared to the cost for an idealized scenario where no overweight trucks are traveling over the pavement.

The incremental approach requires the evaluation of the damage caused by each vehicle and the accumulation of damage over the design life. Damage to pavements has traditionally been evaluated using the Equivalent Single Axle Load (ESAL) method which is an empirical model developed in the 1950's during the American Association of State Highway Officials (AASHO) road tests conducted at the University of Illinois (AASHO, 1961). The ESAL method has been found to ignore a number of parameters that may limit the applicability of the model. For this reason, researchers have recently proposed various versions of Mechanistic-Empirical Pavement Design Guide (MEPDG) methods that are based on a combination of theoretical evaluation of the stresses in the pavement and empirical assessment of the damage they cause (AASHTO, 2008).

The object of this study is to evaluate the cost to New York pavements due to the effect of overweight vehicles. Overweight vehicles can be categorized into the following groups: 1) trucks issued permits to carry divisible loads beyond those stipulated by the New York State weight regulations including the federally mandated 80,000 lbs Gross Vehicle Weight (GVW) limit and the Federal Bridge Formula-B (FBF) and other limits set by the New York legislature; 2) special hauling vehicles carrying permits to exceed the weight limits; 3) Illegally overweight trucks. Task 2.c report described different ESAL-based methods for estimating the pavement cost due to overweight trucks. These ESAL-based methods are adaptations of the studies by the Ohio DOT, which utilized a design-oriented approach Campbell et al (2009), and the Louisiana DOTD, which adopted a maintenance-oriented approach (Saber and Roberts, 2008). Task 2.c report applied these approaches using two different idealized no-overweight scenarios. One idealized no-overweight scenario is similar to that adopted by Ohio DOT which simply removed from the traffic stream all overweight trucks. The other idealized no-overweight scenario assumes that the total cargo being carried by the overweight trucks will have to be carried by no-overweight trucks. The second scenario will require an increase in the number of no-overweight trucks which is an approach similar to that adopted in the LADOTD study. In both scenarios, representative Weigh-In-Motion (WIM) data was assembled and categorized based on truck axle configurations using the Federal Highway Administration (FHWA) vehicle classification system and truck overweight category.

The analysis of the results obtained in Task 2.c highlighted the following limitations in the Louisiana DOTD and Ohio DOT approaches in achieving the objectives of this study:

1. The Ohio DOT approach emphasized the effect of heavy vehicles on the design of the pavement and assumed that pavement maintenance was not related to presence of overweight trucks.
2. The Louisiana DOTD approach aimed to study the differences in the structural overlay thickness required during regular maintenance cycles when additional overweight trucks are allowed to travel over existing pavements in their current conditions.
3. Both approaches relied on the ESAL method, for pavement design in the case of Ohio DOT and for the determination of the structural overlay thickness in the case of the Louisiana DOTD approach. Furthermore, the LADOTD approach utilized variations on the ESAL method that are not used by NYSDOT.
4. While the ESAL method may be appropriate for the design of new pavements, it is not suitable for estimating the damage to existing pavements and the determination of the time at which corrective maintenance is required.

To overcome the limitations of the Louisiana DOTD approach while supplementing that of the Ohio DOT with the inclusion of the effects of overweight trucks on maintenance schedules, Task 2.d Report proposed a hybrid approach, herein labeled the NYSDOT approach. The proposed NYSDOT approach uses the basic concept applied in the Ohio DOT approach which states that the presence of overweight trucks will require different pavement design thicknesses that can be determined using the ESAL method, augmented by the consideration of the accumulated damage caused by the overweight trucks as done during the LADOTD study. Like the LADOTD method, the idealized scenario in the proposed NYSDOT approach is based on assuming that the cargo currently transported by overweight trucks will have to be re-distributed to non-overweight trucks. However, unlike the LADOTD approach, this report uses the MEPDG method for evaluating the damage to pavement and the identification of the times required for corrective maintenance.

The traffic load used in the analysis is obtained from Weigh-In-Motion truck data. In the idealized scenario, the number of trucks that are not overweight is increased to carry the cargo currently being transported by overweigh trucks.

The basic concept of the proposed NYSDOT approach is presented in Figure 1. The approach consists of the following steps:

1. Identify the pavement section to be analyzed and its characteristic parameters including Average Daily Truck Traffic (ADTT), number of lanes, section length and width.
2. Design pavement section to current (actual) truck weights using the ESAL method
    a. Determine current ESAL truck load that pavement must endure during its design life
    b. Design pavement section using the ESAL method
3. Determine required number of maintenance cycles under current (actual) truck traffic using the MEPDG method
    a. Determine truck configurations based on Weigh-in-Motion Data
    b. Determine MEPDG input parameters
    c. Use MEPDG analysis to determine the time at which a corrective maintenance is required
4. Repeat step 2 for idealized truck traffic with no overweight trucks
    a. Develop the idealized truck traffic scenario described in Task 2.c report, which assumes that the total cargo on a given highway remains constant and the weight being carried by the overweight trucks is shifted to trucks within the legal limits
    b. Determine ESAL truck load for idealized scenario
    c. Design pavement section
5. Repeat step 3 for idealized truck traffic scenario with no overweight trucks travelling over the pavement designed in step 4
6. Calculate incremental cost
    a. Define material cost for design
    b. Define maintenance cost
    c. Determine the difference in the costs associated with steps 2 and 3 as compared to the costs associated with steps 4 and 5

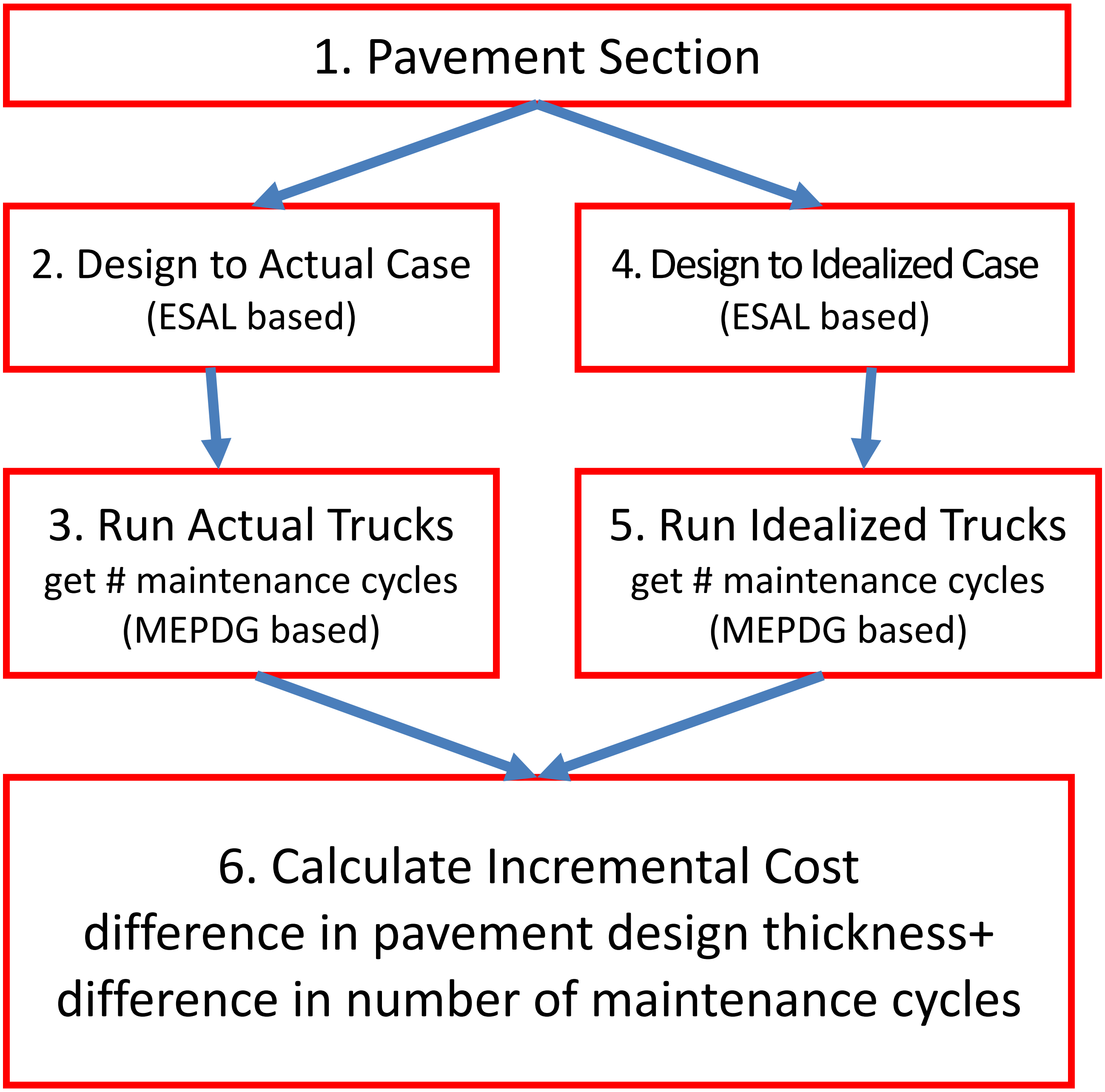


Figure 4.1- Flow chart of Proposed NYSDOT Pavement Cost Analysis Procedure

4.1.2 Chapter Outline

The following sections of this Chapter provide the details of the steps outlined above. The analysis approach is illustrated by applying it to the I-88 Corridor between Binghamton and Schenectady. Subsequently, the approach is applied to the entire highway network under the jurisdiction of the NYSDOT. Specifically, the Chapter consists of the following sections:

4.1. Introduction
4.2. I-88 Corridor description
4.3. NYS typical pavements types
4.4. Pavement life-cycle
4.5. Pavement models for design and performance evaluation
4.6. ESAL method for pavement design
4.7. MEPDG approach for pavement assessment
4.8. MEPDG-based corrective maintenance procedures
4.9. Implementation for I-88 Corridor
4.10. Implementation to NYSDOT highway network
4.11. Conclusions

4.2 I-88 Corridor Description

4.2.1 Site Information

Interstate I-88 in New York State was selected in the Task 1.e report to describe the implementation of the infrastructure damage allocation algorithm on the corridor level.

Interstate I-88 is a 117.70-mile long highway that begins just outside Binghamton in Broome County and passes through Chenango, Otsego, Delaware, and Schoharie counties to end just outside Schenectady with a total of 25 exits/entrances along the route plus endpoints. I-88 closely parallels state route NY-7 that was once the main connector route from the Capital District to Binghamton until the completion of I-88 in 1989. The construction of I-88 began in 1968 and the first section opened in the early 1970's. Figure 2 highlights the location of the I-88 corridor in the New York highway network.

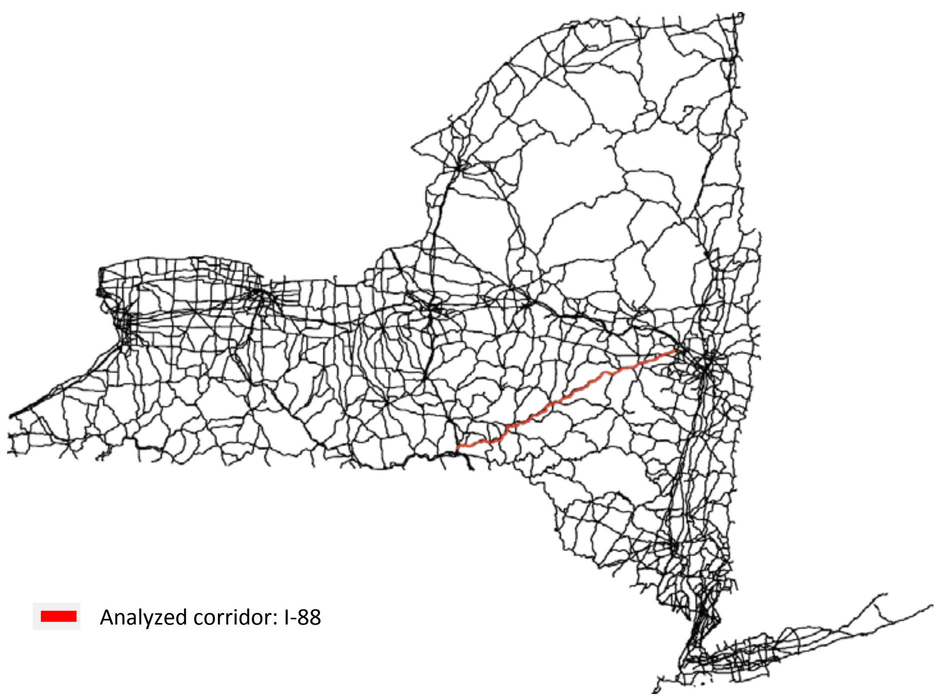


Figure 4.2 - I-88 Corridor within New York State Highway Network

4.2.2 Source of Truck Data

The truck traffic and weight data for the I-88 corridor is taken from Weigh-In-Motion (WIM) station 9580 located near the North-East end of I-88. This WIM station is located near Exit 23, 20 miles away from the end of the highway near Schenectady as shown in Figure 4.3. The 2011 data from this WIM station are analyzed in subsequent sections of this Chapter and extrapolated as representative of the traffic for the entire corridor. The data is sorted based on the FHWA classification and the truck gross weight data and axle weights are used for the analysis presented in this report.

Figure 4.3 - New York State WIM Stations with Highlighted Station 9580 on Interstate I-88

4.2.3 Pavement Information

The 117.70-mile long I-88 highway has all typical types of pavements used throughout the New York State (NYS) highway infrastructure. The I-88 Corridor includes 53.94 miles of concrete pavements, 50.59 miles of overlay, and 13.17 miles of asphalt. Pavement type distribution along the I-88 corridor is presented in Figure 4.4.

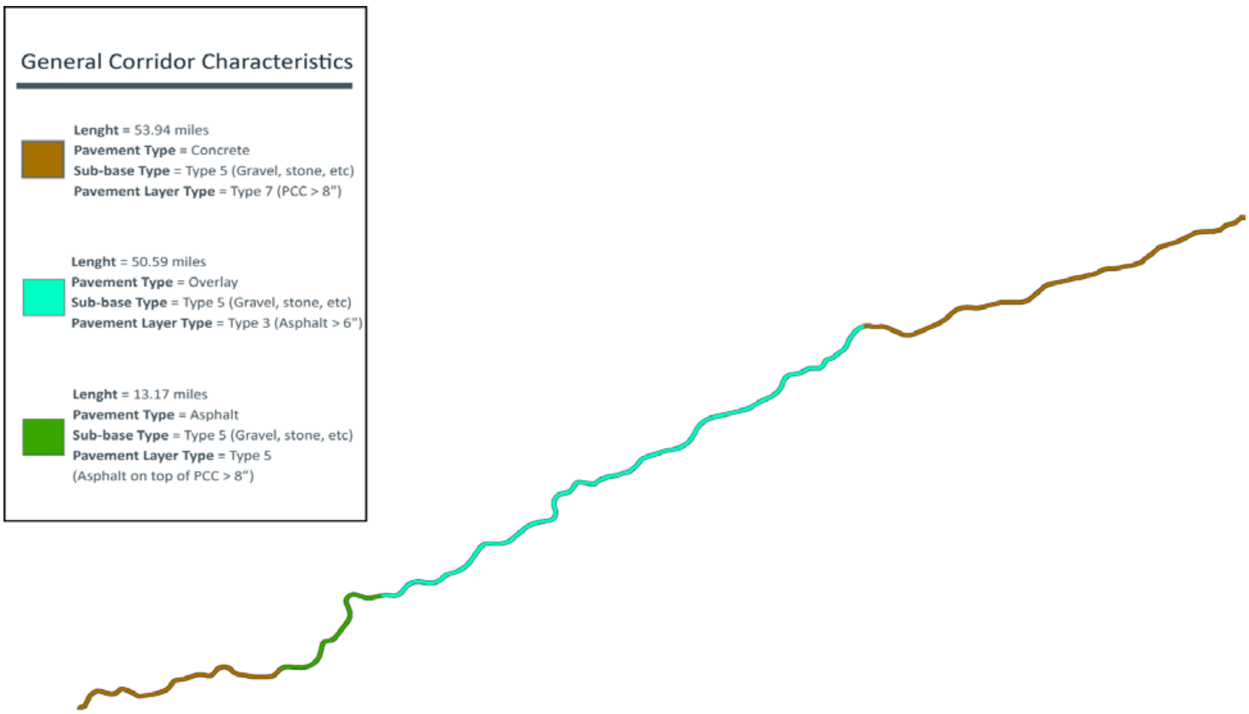


Figure 4.4 - Types of Pavements on I-88 Corridor (Binghamton on lower left corner)

4.3 NYS Pavement Types

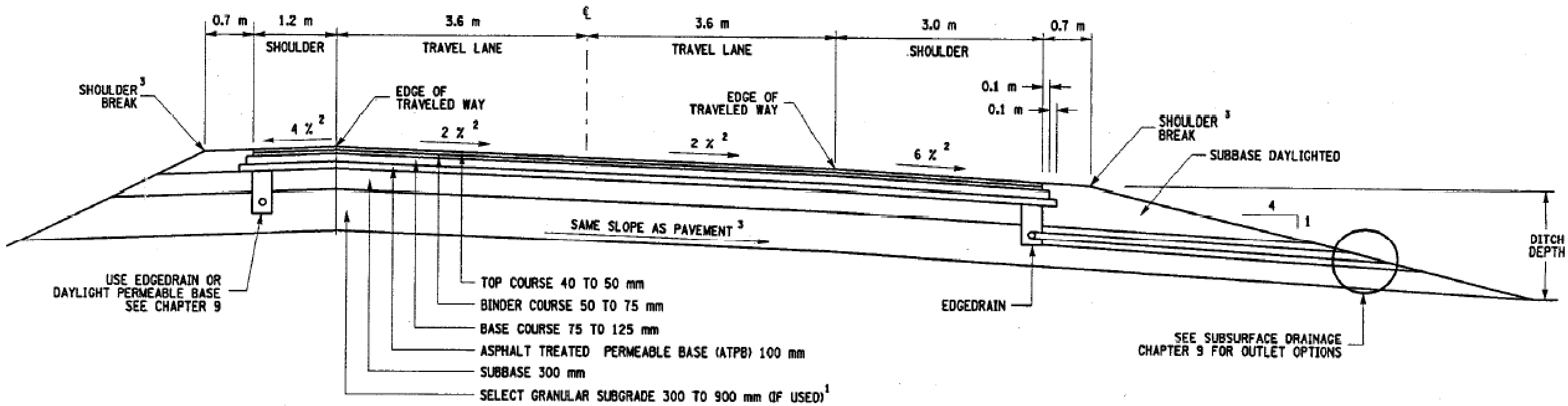
The NYS pavement database including common pavement types was presented in the report of Task 1.c. However, the database does not provide details about the layer thicknesses. For the purposes of this study, the pavement types are consolidated into two types: flexible and rigid as typically done by other researchers.

A flexible pavement structure is typically composed of several layers of material with better quality materials on top where the intensity of stress from traffic loads is high and lower quality materials at the bottom where the stress intensity is low. A typical flexible pavement structure consists of the surface course and underlying base and subbase courses. Each of these layers contributes to structural support and drainage.

A rigid pavement structure is composed of a hydraulic cement concrete surface course and underlying base course; a sub-base course is also often used. NYS rigid pavement is typically jointed plain concrete pavement (JPCP). JPCP controls cracks by dividing the pavement into individual slabs separated by contraction joints. Each inside lane slab is typically 3.6 m (12-ft) wide and 3.6 m (12-ft) to 6.1m (20-ft) long. Outside lanes are normally wider at 4.2 m (14-ft). JPCP pavements do not include any reinforcing steel but use dowel bars and tie bars between adjacent slabs. Jointed reinforced concrete pavement (JRCP) and continuously reinforced concrete pavement (CRCP) are other types of rigid pavements that are commonly used but not in New York State.

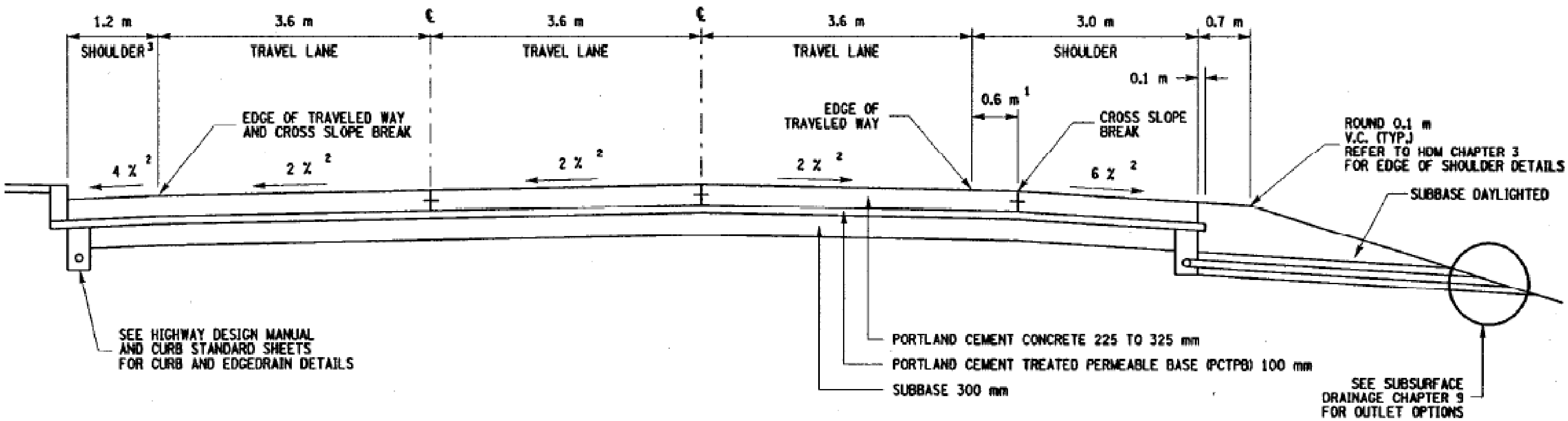
A composite pavement consists of hot mix asphalt (HMA) on top of hydraulic cement concrete. The HMA overlay may be placed at the final stage of a new construction, or as part of a rehabilitation or safety treatment. Composite pavement behavior under traffic loading is usually analyzed using the same approach as rigid pavement although the pavement should be considered as semi-rigid if the HMA is placed after cracking the concrete slab which is one of the rigid pavement rehabilitation techniques.

Typical cross-sections for NYS flexible and rigid pavements are taken from the NYSDOT Comprehensive Pavement Design Manual (NYSDOT, 2014) and shown in Figures 4.5 and 4.6.



THE DIMENSIONS SHOWN ARE TYPICAL ONLY AND THE DETAILS ARE SHOWN TO GIVE THE DESIGNER AN IDEA OF WHAT OVERALL SECTION LOOKS LIKE.

Figure 4.5 - Typical Hot Mix Asphalt (HMA) Pavement Section (Flexible Pavement Section)



THE DIMENSIONS SHOWN ARE TYPICAL ONLY AND THE DETAILS ARE SHOWN TO GIVE THE DESIGNER AN IDEA OF WHAT THE OVERALL SECTION LOOKS LIKE.

1. THE DRIVING LANE SLAB WIDTH IS 4.2 m, OF WHICH 3.6 m IS THE TRAVEL LANE AND 0.6 m IS PART OF THE SHOULDER.

Figure 4.6 - Typical Portland Cement Concrete Pavement Section (Rigid Pavement Section)

4.4 Pavement Life-cycle

According to NYS pavement design manual (NYSDOT, 2014 Section 6.2.1), the design life of any pavement type is 50 years. This design life is called analysis period. The lower layers of a pavement including sub-base and under-drain system are designed to last the entire design period. However, the top layers have a much shorter design life. The structure of the top layers is designed for a performance period of 20 years because a 50-year design life for the top layers is not practical from the cost of construction and maintenance perspective.

To keep the pavement in proper condition, four general procedures are used:

1. Preventive treatment which may consist of joint filling, crack sealing, and other treatments
2. Corrective maintenance which may consist of mill and fill, application of a single course overlay on flexible pavements, or deep joint and spall repair for rigid pavements. It has an expected service life of 8 to 20 years
3. Pavement rehabilitation such as applying a multicourse overlay for flexible and rigid pavements
4. Reconstruction

In principle, pavement treatment and corrective maintenance as well as rehabilitation are repeated as often as needed until the end of the 50-year design life at which time the pavement is completely reconstructed on existing alignment. However, the current NYS goal is to have an infinite design life for pavements by performing corrective maintenance on a regular basis. A major difference between preventive treatment and corrective maintenance is that preventive treatment does not lead to improvement in pavement condition but only helps to slow down the rate of deterioration, while corrective maintenance shows an initial improvement in pavement condition. Pavement rehabilitation involves replacing the top pavement layers allowing for an increase of the performance period.

Several factors such as traffic, environment, material and design considerations affect pavement damage over time. However, traffic load is a primary driving factor in deterioration of pavement. This deterioration can take several forms of distress, such as fatigue and alligator cracking, and rutting for flexible pavements, and transverse cracking and faulting for rigid pavements.

The most common mode of failure for flexible pavements is when the pavement top layers suffer fatigue cracking which propagates through the full thickness to the surface of the permeable base. The cracks in failed pavements allow water to enter the underlying crushed stone material causing saturation leading to the hydraulic displacement of fines (pumping) when subjected to heavy traffic loads. The consequence of pumping is degradation of the structure’s layers and pothole development. Such pavement would require major rehabilitation which could be avoided with timely proper corrective maintenance interventions.

Concrete pavements typically develop transverse cracks over time from repetitive traffic loads (fatigue cracking), thermal effects (curling and warping) that amplify the stress as traffic moves over the pavement, and drying shrinkage. Over time, these transverse cracks will widen degrading the pavement's ability to transfer load through aggregate interlock. Eventually, each side of the crack will move independently (no load transfer), which will result in increased slab deflections. A similar phenomenon may develop at transverse joints between two adjacent slabs. Large slab deflections, combined with the intrusion of water into the sub-layers, can lead to pumping. Pumping occurs when the slab deflects vertically and ejects a mixture of water and fine soil particles from the underlying base layers. As a result of the ejection, there is a loss of soil volume underneath the concrete slab, creating a void between the base layer and the slab. Thus, the pavement will lose vertical elevation, and ultimately cause the crack face to fault FHWA (2006).

Faulting can severely reduce the ride quality of rigid pavements, producing unwanted roughness. In order to prevent the crack or slab edge from faulting corrective maintenance should be applied to retain sufficient load transfer.

In addition to the combined effect of traffic and environment which lead to pumping, due to traffic and moisture, and rutting of flexible pavements due to traffic and temperature, the environment by itself contributes to pavement distress. Environmental damage to pavement is due to moisture, temperature, and frost. Moisture damage causes swells, which can be reduced with proper construction. Temperature acts on pavements in two principal ways: first, temperature extremes can affect asphalt binder rheology; second, temperature variations can cause pavement to expand and contract. Pavements, like all other materials, will expand with a rise in temperature and contract as the temperature falls. Frost may have three detrimental effects on a pavement structure:

1. The upper pavement layer or "wearing course" freezes to pieces; this may occur when the water in the pores of the wearing course has not enough space to expand when freezing. This phenomenon can be prevented through a good mix composition (such as dense asphalt concrete, cement concrete with air entraining admixture).
2. Heave damage: this means that the total pavement structure is pushed upward through the accumulation of water in the form of ice lenses that grow at the freezing plane (32^{o}F isotherm) in the subsoil (or frost-susceptible pavement layers). In the case of a uniform heave, this phenomenon is not detrimental in itself. Through the heave, the density of the grain skeleton however decreases, resulting in a lower bearing capacity.
3. Thaw damage: this is the most serious type of frost damage. As thawing begins, the ice melts primarily from the top down. This excess melt water may be trapped between the pavement structure above and the still frozen soil beneath. The pavement structure is then resting on a subsoil (or pavement layer) with a very low bearing capacity and the (heavy) traffic may easily cause serious structural damage to the pavement.

Another type of damage to rigid pavements includes spalling from the intrusion of incompressible materials into joints and cracks.

4.5 Pavement Models for Design and Performance Evaluation

Historically, pavement design consisted of defining thicknesses of layered materials that would provide strength to a weak subgrade. Pavements were designed against subgrade shear failure. The process was totally based on experience with successful projects. Overtime, engineers developed alternative pavement design methods based on subgrade shear strength. Subsequently, the strength criteria were supplemented with other pavement performance criteria such as ride quality and assessment of surface distress that may increase the rate of deterioration of pavement structures. Eventually, performance became the focal point of pavement design processes. However, the lack of sound scientific theories and means of analysis for the performance evaluation of complex pavement structures necessitated the development of empirical performance models based on experiments or experience. (Schwartz & Carvalho, 2007)

Observations were used to establish correlations between the loads, pavement material, geometric characteristics, and ride quality. The first empirical pavement design methods were based on pavement serviceability indexes extracted from test track experiments. Since the 1960’s and up to this day, the mostly widely used pavement design equations remain those developed by AASHO (1961). The AASHO (1961) empirical equations were based on test track experiments conducted near Chicago in the 1950’s. The AASHO Road Test was a seminal experiment from which the 1993 AASHTO Design Guide eventually evolved. The current AASTHO design equation is a regression relationship between the number of load cycles, pavement structural capacity, and performance, measured in terms of serviceability. The concept of serviceability was introduced in the AASHTO method as an indirect measure of the pavement’s ride quality. The serviceability index is based on surface distresses commonly found in pavements. The AASHTO method has been adjusted several times over the years to incorporate extensive modifications based on theory and experience that expand the applicability of the design equation. On the other hand, the consideration of observed failure mechanisms, such as fatigue cracking and permanent deformation of asphalt concrete, and the introduction of new materials and construction practices have necessitated the development of new design approaches. (Schwartz & Carvalho, 2007)

The Asphalt Institute method and the Shell method for flexible pavements are examples of procedures based on asphalt concrete’s fatigue cracking and permanent deformation failure modes. These methods relate two mechanical responses of pavement to distress: the horizontal strain at the bottom of asphalt layer is related to fatigue cracking and vertical stain in the layers is related to rutting. The Shell Oil and Asphalt Institute methods were the first to use the theory of mechanics to compute structural responses in combination with empirical models to predict the number of loads to failure for flexible pavements. These first mechanistic attempts were based on linear-elastic models which may not be accurate for flexible pavements. Nonlinearities, time and temperature dependency, and anisotropy are some examples of complicated features often observed in pavement materials which necessitate the application of advanced mechanistic models to predict performance. (Garber and Hoel, 2009)

As is the case with flexible pavements, rigid pavement design and performance evaluation can be improved using the mechanistic-empirical approach to combine the theories of mechanics and relate pavement structural behavior and performance to traffic loading and environmental influences. Progress has been made in recent years on certain features of the mechanistic performance prediction problem, but the reality is that fully mechanistic methods are not yet available for practical pavement design. The current mechanistic-empirical approach recommended by AASHTO (2008) is a hybrid approach. Empirical models are used to fill in the gaps that exist between the theory of mechanics and the performance of pavement structures. Simple mechanistic responses are easy to compute with assumptions and simplifications such as homogeneous material, small strain analysis, static loading as typically assumed. But, the mechanics models by themselves cannot be used to predict performance directly; some type of empirical model is required to make the appropriate correlation. Mechanistic-empirical methods are considered an intermediate step between empirical and fully mechanistic methods.

Current NYSDOT pavement design procedures are still based on an adaptation of the ESAL method in the AASHTO 1993 guide (Chen, Bendana, & McAuliffe, 1995). This empirical procedure is described in Section 4.6 of this Report. However, these procedures cannot be used to evaluate the performance of pavement to determine the maintenance needs for the pavement. For this purpose, the recently developed AASHTO MEPDG procedure is used. This procedure is described in Section 4.7.

4.6 ESAL Method for Pavement Design

A pavement is a complex structure made of several different materials. It is subjected to many diverse combinations of loading and environmental conditions throughout its life-cycle, which cause it to deteriorate and eventually fail. Using the AASHTO (1993) empirical model, the impact of trucks on pavement is usually measured by the number of Equivalent Single Axle Loads (ESALs). ESALs were developed as part of the AASHO (1961) road test in the 1950's as a way to normalize the effect of trucks with different axle weights. One ESAL is defined as the damage caused to the pavement by the passing of one 18,000 lb single axle. The use of ESALs for pavement damage estimation allows for the comparison between different types and weights of trucks. Because asphalt and concrete pavements incur different types of damage, ESALs for asphalt and concrete pavements are different.

Pavement condition is assessed using a Pavement Condition Index (PCI) or a Present Serviceability Index (PSI). PSI as defined by the American Association of State Highway and Transportation Officials (AASHTO, 1993) is used in this Report because it can be easily related to the axle weights through the ESAL method and is the most commonly used index. PSI is closely related to the riding comfort/quality of the pavement where PSI can range between 0 for impassible roads and 5 for perfect roads.

For design purposes, it is necessary to select both an initial and terminal pavement serviceability index. Values of initial serviceability established during the AASHO Road Test conditions were 4.2 for flexible pavements and 4.5 for rigid pavements. The terminal serviceability index is the lowest tolerated level before resurfacing or reconstruction becomes necessary for the particular class of highway. An index of 2.5 or 3.0 is often suggested for major highways, and 2.0 for highways with lower classification. For relatively minor highways, where economic considerations dictate that initial expenditures be kept low, a terminal serviceability index as low 1.5 may be used. Actual values are sometimes established by public survey of acceptance.

In this Report, initial pavement serviceability indexes of 4.2 and 4.5 are selected for newly constructed/reconstructed flexible and rigid pavements respectively. A serviceability index of 3.8 is selected to determine the time at which a corrective maintenance intervention should be executed. A terminal serviceability index equal to 2.5 is selected to indicate the end of useful pavement life.

4.6.1. Truck Load Conversion to ESAL

Each pavement section must withstand a certain loading expressed in equivalent single 18kip axle passes over the pavement (ESAL).

ESAL for Flexible Pavements

For flexible pavements, the ESAL from one axle having a load L_x is calculated based on the following formula:

$$\frac{W_x}{W_{18}} = \left[\frac{L_{18}+L_{2s}}{L_x+L_{2x}}\right]^{4.79} \cdot \left[\frac{10^{G/\beta_x}}{10^{G/\beta_{18}}}\right] \cdot [L_{2x}]^{4.33} \tag{4.1}$$

where: W_x = axle applications inverse of equivalency factors

L_x = axle load being evaluated (kips)

L_{18}= 18 (standard axle load in kips)

L_{2x}= code for axle configuration, which is:

= 1 for single axle

= 2 for tandem axle

= 3 for triple axle (added in the 1986 AASHTO Guide)

L_{2s}= 1 for standard axle equal to 1 (single axle)

$G = log\left(\frac{4.2-p_t}{4.2-1.5}\right)$ = a function of the ratio of loss in serviceability at time, t, to the potential loss taken at a point where p_t=1.5

p_t= "Terminal" serviceability index (point at which the pavement is considered to be at the end of its useful life

$b = 0.4 + \left(\frac{0.081\cdot(L_x+L_{2x})^{3.23}}{(SN+1)^{5.19}\cdot L_{2x}^{3.23}}\right)$ = a function which determines the relationship between serviceability and axle load applications

SN = Structural Number

For flexible pavement calculations, the following assumptions are made:

- $SN = 6$ Structural number is equal to six based on the calculations for NYS DOT typical flexible pavement section as shown in Figure 4 with average drainage coefficients and 10.5” HMA thickness.
- $p_t = 2.5$ According to AASHTO this is an average terminal serviceability index value

ESAL for Rigid Pavements

For rigid pavements, the ESAL is calculated based on the following formula:

$$\frac{W_x}{W_{18}} = \left[\frac{L_{18}+L_{2s}}{L_x+L_{2x}}\right]^{4.62} \cdot \left[\frac{10^{G/\beta_x}}{10^{G/\beta_{18}}}\right] \cdot [L_{2x}]^{3.28} \qquad (4.2)$$

where: W_x = axle applications inverse of equivalency factors

L_x = axle load being evaluated (kips)

L_{18}= 18 (standard axle load in kips)

L_{2x}= code for axle configuration, which is:

= 1 for single axle

= 2 for tandem

= 3 for triple (added in the 1986 AASHTO Guide)

L_{2s}= 1 for standard axle equal to 1 (single axle)

$G = log\left(\frac{4.5-p_t}{4.5-1.5}\right)$ = function of the ratio of loss in serviceability at time, t, to the potential loss taken at a point where p_t=1.5

p_t = “Terminal” serviceability index (point at which the pavement is considered to be at the end of its useful life

$b = 1.00 + \left(\frac{3.63\cdot(L_x+L_{2x})^{5.20}}{(D+1)^{8.46}\cdot L_{2x}^{3.52}}\right)$ = function which determines the relationship between serviceability and axle load applications

D = slab depth in inches

For rigid pavement ESAL calculations, the following assumptions are made:

- $D = 9"$ Nine inch slab depth is assumed based on the data from I-88 Corridor.
- $p_t = 2.5$ According to AASHTO this is an average terminal serviceability index value

Implicitly, in the ESAL equation it is assumed that the initial serviceability of flexible pavement is 4.2 and the value for rigid pavements is 4.5.

A sensitivity analysis showed no significant impact of structural number or slab depth on final ESAL calculations.

In this study, ESALs are calculated based on the representative WIM data for the highway being analyzed taking into consideration the pavement type, the Average Daily Truck Traffic (ADTT) on each pavement segment as provided in the NYSDOT pavement database. The example designs implemented in this Report for the I-88 Corridor are based on the WIM data for the year 2011 collected at WIM Station 9580. These data are assumed to be representative of the truck traffic along the entire corridor and the number of trucks is adjusted to reflect the ADTT for each segment.

Simplified ESAL Estimation

When no detailed data are available on truck axle weights, the New York State DOT Comprehensive Pavement Design Manual (2014) proposes that a weighted average ESAL value be assigned for the trucks. For New York State, the average truck equivalency factors for flexible and rigid pavements are shown in Table 4.1. Calculations based on Table 4.1 are only used to quickly verify the results obtained from the WIM ESAL calculations.

Table 4.1 - Truck Equivalency Factor for ESAL Calculation

FHWA Vehicle Classifications	Flexible Pavement (HMA)	Rigid Pavement (PCC)
4 - 13	1.35	1.85

4.6.2. ESAL-Based Design of Pavements

Instead of using the detailed AASHTO formulas, New York State DOT developed tables to extract the critical layer thicknesses for rigid and flexible pavements.

Design of Rigid Pavements

For the design of rigid pavements, the typical NYSDOT cross section is set as shown in Figure 4.6. All layers usually have standard thicknesses except for the slab thickness which would vary based on the site's ESALs. Table 4.2 gives the variation of the slab thickness as a function of the ESAL and slab width. It is herein assumed that pavement slabs are designed to have a width of 4.2 m (14 ft). To obtain a better gradation of the thickness, the data in Table 4.2 are interpolated to produce the design curve shown in Figure 4.7.

Table 4.2 - PCC Slab Thickness (NYSDOT, 2014)

80-kN ESALs	**PCC Slab Thickness 4.2 m driving lane slab width**	**PCC Slab Thickness 3.6 m driving lane slab width**
millions	**mm**	**mm**
ESALs ≤ **22**	225	225
22 < ESALs ≤ **36**	225	250
36 < ESALs ≤ **65**	225	275
65 < ESALs ≤ **100**	250	300
100 < ESALs ≤ **165**	275	325
165 < ESALs ≤ **250**	300	325[1]
250 < ESALs ≤ **400**	325	325[1]

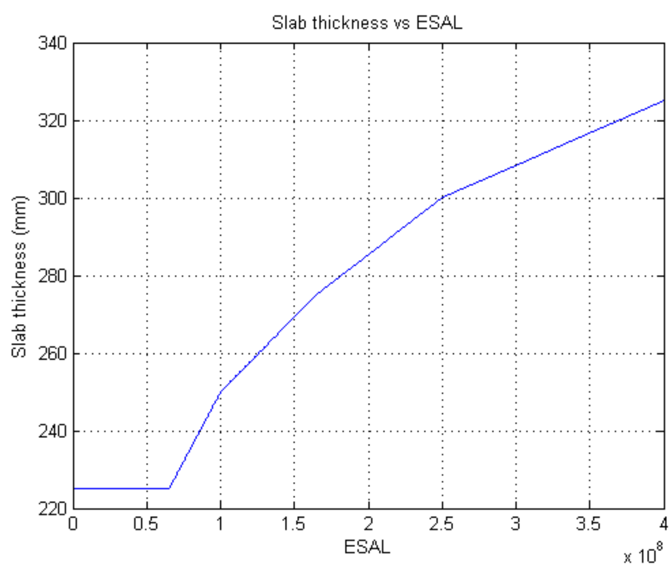


Figure 4.7 - Interpolated Design Curve for PCC Slab Thickness

Design of Flexible Pavements

For the design of flexible pavements, the typical NYSDOT cross section is set as shown in Figure 4.5. All layers usually have standard thicknesses except for the Hot Mix Asphalt (HMA) and the granular subgrade thicknesses which vary based on the site's ESALs and soil modulus of resilience. Table 4.3 gives the variation of HMA and subgrade thicknesses as a function of ESAL for a modulus resilience factor M_r=28 MPa which is the recommended value by NYSDOT (2014). To obtain a better gradation of the thicknesses, the data in Table 4.3 are interpolated to produce the design curves shown in Figure 4.8.

Table 4.3 - Flexible Pavement Design for M_r=28 MPa (NYSDOT, 2014)

M_r = 28 MPa		
80 kN ESALs over Design Life	**Total HMA Thickness**	**Select Granular Subgrade Thickness**
millions	**mm**	**mm**
ESALs ≤ **2**	165	0
2 < ESALs ≤ **4**	175	0
4 < ESALs ≤ **8**	200	0
8 < ESALs ≤ **13**	225	0
13 < ESALs ≤ **23**	250	0
23 < ESALs ≤ **45**	250	150
45 < ESALs ≤ **80**	250	300
80 < ESALs ≤ **140**	250	450
140 < ESALs ≤ **300**	250	600

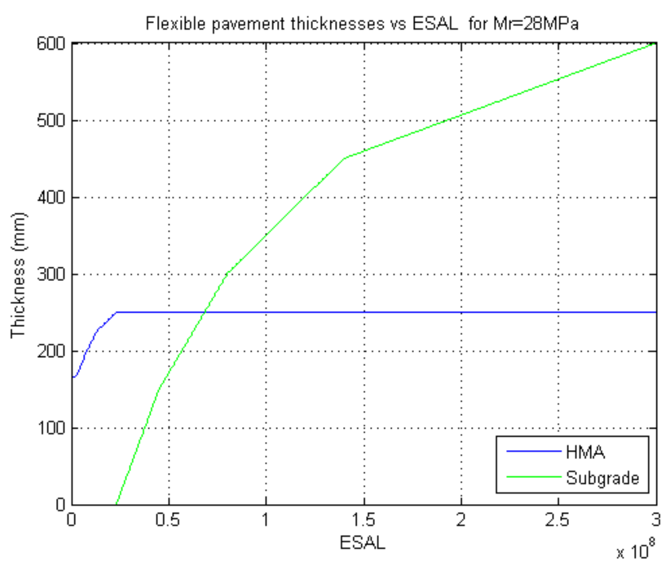


Figure 4.8 - Interpolated Design Curve for Flexible Pavement Design (M_r=28 MPa)

4.6 MEPDG Approach for Pavement Assessment

As discussed earlier, the Mechanistic-Empirical Pavement Design Guide (MEPDG) approach provides an improved method to estimate the condition of a pavement over time due to the applied vehicular load and the environment. The AASHTO (2008) MEPDG approach is used in this report to study how pavement performance changes over time and to estimate the number of corrective maintenance interventions needed to maintain the serviceability of both flexible and rigid pavements.

In engineering practice, it is common to use the Present Serviceability Rating (PSR) as a criterion to assess the in-service condition of flexible or rigid pavements and determine the need for corrective maintenance. In practice, PSR is defined as the mean value of the independent ratings obtained by different pavement inspectors who are evaluating the present serviceability condition of a specific roadway section (Huang, 2004). The PSR rating assigns a number between 0 and 5 to a pavement segment with 5 being the optimum rating. A rating of 0 indicates that the pavement segment is completely damaged and a rating of 5 indicates a perfect condition. Generally, a PSR rating in the neighborhood of 2.5 indicates that the pavement surface is becoming rough. Although PSR is usually determined through field inspection with the help of various non-destructive instruments, researchers have developed empirical models that correlate field-inferred PSR values to analytical parameters obtained after applying the stress analysis procedure inherent to the MEPDG method.

An alternative method to assess the condition of pavement is the International Roughness Index (IRI) which provides a ride quality statistic used in conjunction with the MEPDG approach (Mallick, 2009). IRI is usually calculated by measuring the longitudinal profile of the pavement in each wheel path, then entering that profile into a computer algorithm to simulate the response of the suspension of a typical sedan car traveling at 50 mph. The resulting accumulated vertical bounce of the vehicle is the IRI, reported in inches per mile. The higher the IRI number, the rougher is the ride quality. An IRI value less than 60 is indicative of a very smooth ride; a value between 61 and 120 indicates a smooth ride. A fair ride is associated with $121<IRI<171$. The ride quality is rough when IRI is between 171 and 220 and the ride is very rough when IRI is greater than 220 (NYSDOT, 2014). Researchers have developed models to correlate PSR and IRI values for a given pavement segment. One such model reported by (Huang, 2004) is given as:

$$PSR = 5e^{-0.0041IRI} \tag{4.3}$$

The AASHTO MEPDG manual (2008) provides a set of empirical equations that were calibrated to correlate the pavement condition expressed in terms of IRI and the mechanistic-empirical parameters that describe the effect of loads and environment on the pavement. These empirical equations were developed in order to provide analytical models to estimate the damage to pavements over time for application in pavement design and planning. Specifically, MEPDG equations can be used to obtain estimates of IRI based on pavement types and properties, vehicular loads, and environmental conditions. The IRI provides a measure of pavement distress that helps predict maintenance needs over time. Two sets of equations are provided in AASHTO' MEPDG manual (2008): one set is applicable to flexible pavements, and the other is applicable to rigid pavements.

4.7.1. MEPDG Method for Flexible pavements

According to the MEPDG approach, the International Roughness Index, IRI, of a given flexible pavement section can be estimated by equation (5-15a) in the MEPDG guide (AASHTO, 2008):

$$IRI = IRI_0 + 0.015(SF) + 0.400(FC_{TOTAL}) + 0.0080(TC) + 40.0(RD) \tag{4.4}$$

where: IRI_0 = Initial IRI of a given section right after construction or rehabilitation (in/mile)

SF = Site factor

$FC_{TOTAL} =$ Area of fatigue cracking (combined alligator, longitudinal and reflection cracking in the wheel path) as a percent of total lane area. All load related cracks are combined on an area basis; length of cracks is multiplied by 1 foot to convert crack per unit length into crack per unit area.

$TC =$ Length of transverse cracking (including the reflection of transverse cracks in exiting Hot Mix Asphalt, HMA, pavements) (feet/mile)

$RD =$ Average rut depth (in)

As can be seen in Eq. (4.4), IRI of a given flexible pavement section is calculated using five different input parameters. IRI_0 is the estimate when the section is new or immediately after maintenance is performed, SF is calculated once every year of the design life and the three remaining parameters (FC_{TOTAL}, TC and RD) are the accumulation of damage obtained from the crossing of each truck. Details on the calculation of each parameter are given below.

Initial and Terminal IRI

For a flexible pavement section, an initial serviceability index PSR= 4.2 is usually used. The design of a pavement is usually performed with the goal of reaching a terminal PSR=2.5. On the other hand, a PSR=3.8 would correspond to the pavement showing very early signs of reduction in performance. Executing corrective maintenance at that stage would ensure that the pavement remains smooth and increase its durability to keep it serviceable. Using Eq. (4.3), the initial and corrective maintenance IRI for flexible pavement sections can be calculated as:

$$IRI_0 = -\frac{1}{0.0041}\ln\left(\frac{4.2}{5.0}\right) = 42.52$$

$$IRI_{corrective\ maintenance} = -\frac{1}{0.0041}\ln\left(\frac{3.8}{5.0}\right) = 67$$

Note that initial IRI_o is defined for a new pavement when it is first opened for traffic and the same initial IRI_o is used after each corrective maintenance cycle.

Site Factor (SF)

The Site Factor in Eq. (4.4) can be calculated using equation (5-15b) of the AASHTO MEPDG procedures (AASHTO, 2008):

$$SF = Age[0.0203(PI + 1) + 0.007947(Precip + 1) + 0.000636(FI + 1)] \quad (4.5)$$

where: Age = Pavement age in years
PI = Percent Plasticity index of the soil
Precip = Average annual precipitation or rainfall in inches
FI = Average annual freezing index in ºF days

In this report, the design of flexible pavements is based on a soil resilience modulus equal to 28 MPa that corresponds to fine grained soil with plasticity index in the range of 4%-7%. A value for PI=5% is used as an average value. The freezing index for New York was taken as 753^{o}F days.

Data on annual precipitation is available through the National Oceanic and Atmospheric Administration (NOAA) website at https://www.ncdc.noaa.gov/cdo-web/. As an example, for Interstate I-88 between Binghamton and Schenectady, the average annual precipitation is equal to 41.22 in. This is based on the average monthly precipitation for Binghamton and Schenectady as presented in Table 4.4.

The contribution of SF to the IRI for the I-88 Corridor data shows that the site factor, SF, has little influence on IRI.

Table 4.4- Average Monthly Precipitation (in) for Binghamton and Schenectady

Month	Binghamton	Schenectady
Jan	2.45	2.67
Feb	2.31	2.14
Mar	2.99	3.06
Apr	3.43	3.32
May	3.57	3.61
Jun	4.31	3.81
Jul	3.7	3.07
Aug	3.45	3.36
Sept	3.63	3.06
Oct	3.33	3.01
Nov	3.3	2.99
Dec	2.83	2.71
Sum	42.58	39.88
Average for 2 cities	41.22	

Total Area of Fatigue Cracking (FC_{Total})

The total area of fatigue cracking FC_{Total} as a percent of the total lane area to be entered in Eq. (4.4) is calculated as the sum of alligator, longitudinal and reflection cracking. However, reflection cracking is calculated only for rigid pavements with Hot Mix Asphalt (HMA) overlays and is not calculated for flexible sections.

The MEPDG approach provides a method to calculate alligator and longitudinal cracking based on the cyclic fatigue Damage Index (DI). The damage caused by a single crossing of an axle group i, DI_i, is defined as the reciprocal of the number of applications of axle group load, i, that would cause the total damage of the pavement; or as expressed in Equation (5-5) of the MEPDG guide (AASHTO, 2008):

$$DI_i = \frac{1}{N_{f-HMA}} \qquad (4.6)$$

Where: N_{f-HMA} which is the hypothetical number of crossings of axle group i that would cause pavement failure is obtained from equation (5-4a) of MEPDG guide (AASHTO, 2008):

$$N_{f-HMA} = k_{f1}(C)(C_H)\beta_{f1}(\varepsilon_t)^{k_{f2}\beta_{f2}}(E_{HMA})^{k_{f3}\beta_{f3}} \qquad (4.7)$$

where: $N_{f-HMA} =$ Allowable number of axle-group load applications for the pavement

$\varepsilon_t =$ Tensile strain at critical pavement location (bottom of asphalt layer for bottom up alligator cracking damage, or top of asphalt for top down fatigue damage) due to the crossing of axle group i. The tensile strain is determined using a mechanistic analysis program or a finite element program. In this study, the mechanistic analysis of flexible pavements is executed using the program KENLAYER (Huang, 2004)

$E_{HMA} =$ Dynamic modulus of the Hot Mix Asphalt (HMA) layer measured in compression, psi

$k_{f1}, k_{f2}, k_{f3} =$ Global field calibration parameters which based on NCHRP Report 1-40D (2009) are set at 0.007566, -3.91492 and -1.281 respectively.

$\beta_{f1}, \beta_{f2}, \beta_{f3} =$ Local or mixture specific field calibration constants; for the global calibration effort, these constants were set to 1.0 (AASHTO, 2008).

In Eq. (4.7), the constant C is calculated using Equation (5-4b) of MEPDG guide (AASHTO, 2008):

$$C = 10^M \qquad (4.8)$$

And M is found using Equation (5-4c) of MEPDG guide (AASHTO, 2008):

$$M = 4.84\left(\frac{V_{be}}{V_a + V_{be}} - 0.69\right) \qquad (4.9)$$

where: $V_{be} =$ Effective asphalt content by volume, taken as 11 % (Raul Velasquez, 2009)

$V_a =$ Percent air voids in the HMA mixture, taken to be 8.5 % (Raul Velasquez, 2009)
$C_H =$ Thickness correction term, depending on type of cracking in question

For alligator cracking, Equation (5-4d) of MEPDG guide (AASHTO, 2008) is used to find the thickness correction term:

$$C_H = \frac{1}{0.00398 + \frac{0.003602}{1+e^{(11.02-3.49H_{HMA})}}} \quad (4.10)$$

For longitudinal cracking, equation (5-4e) of MEPDG guide (AASHTO, 2008) is used:

$$C_H = \frac{1}{0.01 + \frac{12}{1+e^{(15.676-2.8186H_{HMA})}}} \quad (4.11)$$

where: H_{HMA} = Total HMA thickness in inches

The dynamic modulus of HMA (E_{HMA}) in Eq. (4.7) can only be determined from extensive laboratory testing as it depends on many factors such as loading frequency, temperature and other environmental factors. For the purposes of this study, estimates of the dynamic modulus of asphalt at various temperatures are extracted from data plotted in Figure 4.9 as extracted from the report by (Yousefdoost, N.A). Figure 4.9 compares the normalized measured dynamic modulus of typical HMA at different temperatures. The basis for normalization is the modulus measured at a temperature T=70°F.

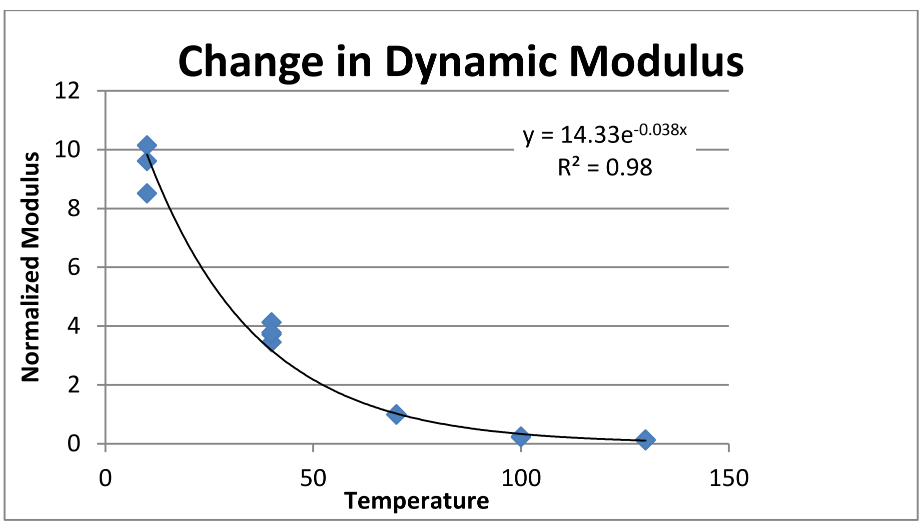


Figure 4.9 - Change in Dynamic Modulus vs. Temperature

Accordingly, the dynamic modulus of HMA can be approximated using following equation:

$$E_{HMA} = 400{,}000 \times 14.331e^{-0.038T} \tag{4.12}$$

where: E_{HMA} = Dynamic modulus of HMA in psi
T = HMA temperature in Fahrenheit

Eq. (4.12) is used in this Report to relate the dynamic modulus to the temperature of the pavement when a truck crosses the pavement section being analyzed.

The damage index DI_i for each axle group crossing the pavement as calculated in Eq. (4.6) is then summed for all axle groups of one truck and for all trucks that cross the pavement section being analyzed during the service period to obtain the cumulative damage.

$$DI = \sum_{k=1}^{N_{truck}} \sum_{i=1}^{n_{axle\ group}} (DI_i)_k \tag{4.13}$$

Two types of cumulative damages are calculated: Bottom up alligator damage, DI_{Bottom}, which is obtained from the application of Eq. (4.6) with the tensile strain calculated at the bottom of HMA layer; and top down fatigue damage DI_{Top} which is obtained from Eq. (4.6) with the tensile strain at the top of the HMA pavement.

Area of Alligator Cracking (FC_{Bottom})

The area of alligator cracking as a percent of total lane area is calculated using the following equation (AASHTO, 2008):

$$FC_{Bottom} = \frac{1}{60}\left(\frac{C_4}{1+e^{\left(C_1C_1^* + C_2C_2^* Log(DI_{Bottom}*100)\right)}}\right) \tag{4.14}$$

where: DI_{Bottom} = Cumulative damage index at the bottom of the HMA layer (from Eq. 4.13 and 4.6)
C_1, C_2, C_4 = Transfer function regression constants for alligator cracking; given by AASHTO (2008) as 1.00, 1.00 and 6,000 respectively
$C_1^* =$ $-2C_2^*$ (MEPDG Equation 5-6b)
$C_2^* =$ $-2.40874 - 39.748(1 + H_{HMA})^{-2.856}$ (MEPDG Equation 5-6c)
H_{HMA} = Thickness of HMA layer in inches

Length of Longitudinal fatigue cracks (FC_{Top})

Using the cumulative damage index calculated in Eq. (4.13) for top cracking, the length of longitudinal fatigue cracks is calculated using equation (5-8) of the MEPDG guide (AASHTO, 2008):

$$FC_{Top} = 10.56\left(\frac{C_4}{1+e^{\left(C_1 - C_2 Log(DI_{Top})\right)}}\right) \tag{4.15}$$

where: $DI_{Top} =$ Cumulative damage index near the top of the HMA surface (from Eq. 4.11 and 4.4)

$C_1, C_2, C_4 =$ Transfer function regression constants for longitudinal cracks; listed in AASHTO (2008) as 7.00, 3.5 and 1,000 respectively

Ultimately, the analysis of the various pavement sections showed that, for the range of pavement profiles applicable to this study, no tensile strains are obtained on the top of the flexible pavement and FC_{Top} is equal to zero.

Non-Load Related Cracking - Transverse Cracking (TC)

The crack length induced by each thermal cooling cycle is predicted using Paris law of crack propagation, as given in Equation (5-11a) of MEPDG guide (AASHTO, 2008):

$$\Delta C = A(\Delta K)^n \tag{4.16}$$

where: ΔC = Change in crack length due to a cooling cycle

ΔK = Change in stress intensity factor due to a cooling cycle, and

A, n = Fracture parameters for HMA mixtures

The parameters A and n can be obtained from the indirect creep-compliance and tensile strength of HMA in accordance with Eq. 5-11b and 5-11c of MEPDG guide (AASHTO, 2008) where:

$$A = 10^{k_t \beta_t \left(4.389 - 2.52 Log(E_{HMA} \sigma_m n)\right)} \tag{4.17}$$

and,

$$\mathrm{n} = 0.8\left[1 + \frac{1}{\mathrm{m}}\right] \tag{4.18}$$

where: k_t = Coefficient determined through global calibration

E_{HMA} = HMA dynamic modulus (psi) calculated from Eq. 4.12

σ_m = HMA Mixture tensile strength (psi)

m = Parameter derived from the indirect tensile creep compliance curve measured in laboratory tests

β_t = Local or mixture calibration factor

Additionally, the stress intensity factor K, has been incorporated in the MEPDG through the use of a simplified equation developed from theoretical finite element studies as shown in equation (5-11d) of MEPDG guide (AASHTO, 2008):

$$K = \sigma_{tip}[0.45 + 1.99(C_0)^{0.56}] \tag{4.19}$$

where: σ_{tip} = Far field stress from pavement response model at depth of crack tip (psi). Calculated from mechanical analysis of pavement using KENLAYER

C_0 = Current crack length (ft)

Ultimately, the degree of cracking is predicted by the MEPDG using an assumed relationship between the probability distribution of the log of the crack depth to HMA-layer thickness ratio and the percent of cracking using MEPDG guide equation (5-11e);

$$TC = \beta_{t1}\mathrm{N}\left[\frac{1}{\sigma_d}\mathrm{Log}\left(\frac{C_d}{H_{HMA}}\right)\right] \tag{4.20}$$

where: TC = Observed amount of thermal cracking (ft/mi)

β_{t1} = Regression coefficient determined through global calibration (=400)

$\mathrm{N}[z]$ = Standard normal probability distribution evaluated at [z]

σ_d = Standard deviation of the log of the depth of cracks in the pavement (=0.769 in)

C_d = Crack depth (in) (calculated after each cycle from Eq. 4.16 $C_d = C_o + \Delta C$

H_{HMA} = Thickness of HMA layers (in)

After investigating the effect of TC on IRI, looking at typical values published in the literature and observing that the effect of TC on IRI is less than 10%, a typical value of 1000 feet/mile is used in this study to calculate the IRI in Eq. (4.4) for each rehabilitation cycle for each section based on the work presented by (Velasquez, 2009).

Average Rut Depth (RD)

The average rut depth is the sum of the permanent deformations of the HMA layer and all unbound pavement sublayers. Equations to find each deformation are obtained using the MEPDG models provided in AASHTO (2008).

Plastic deformation in HMA layer

The permanent or plastic vertical deformation in the HMA layer is obtained using Equation (5-1a) of MEPDG guide (AASHTO, 2008):

$$\Delta_{p(HMA)} = \beta_{1r} k_z \varepsilon_{r(HMA)} H_{HMA} 10^{k_{1r}} n^{k_{2r}\ \beta_{2r}} T^{k_{3r}\ \beta_{3r}} \tag{4.21}$$

where: $\varepsilon_{r(HMA)}$ = Resilient or elastic strain calculated at the mid-depth of each HMA layer

k_{1r}, k_{2r}, k_{3r} = Global field calibration parameters which based on NCHRP 1-40D are given as -3.35412, 0.4791 and 1.5605 respectively

$\beta_{1f}, \beta_{2f}, \beta_{3f}$ = Local or mixture field calibration constants; for the global calibration these constants were all set to 1.0 as per AASHTO MEPDG (2008)

T	= Mix or pavement temperature, ºF
n	= Number of axle load group repetitions, use n=1 since one axle group is analyzed at a time
k_z	=depth confinement factor
H_{HMA}	= Total HMA thickness in inches

The depth confinement factor (k_z) is calculated with MEPDG Equation (5-1b)

$$k_z = (C_1 + C_2 D) 0.328192^D \quad (4.22)$$

The constants C_1 and C_2 are found according to AASHTO (2008) MEPDG Equations (5-1c) and (5-1d) from:

$$C_1 = -0.1039(H_{HMA})^2 + 2.4868(H_{HMA}) - 17.342 \quad (4.23)$$

$$C_2 = 0.0172(H_{HMA})^2 - 1.7331(H_{HMA}) + 27.428 \quad (4.24)$$

where: D = Depth to the middle of the thickness= $H_{HMA}/2$ (Harold L.Von Quintus, 2012)
H_{HMA} = Total HMA thickness in inches

Plastic deformation in unbound layers

Plastic deformation of the unbound layers is essentially the plastic deformation of the soil supporting the HMA. It is calculated for each layer using Equation (5-2a) from MEPDG guide (AASHTO, 2008):

$$\Delta_{p(soil)} = \beta_{s1} k_{s1} \varepsilon_v h_{soil} \left(\frac{\varepsilon_0}{\varepsilon_r}\right) e^{-\left(\frac{\rho}{n}\right)^{\beta}} \quad (4.25)$$

where: β_{s1} = Calibration constant for the rutting in the unbound layers; set =1.0 (AASHTO, 2008)
k_{s1} = Global calibration coefficient set at 1.673 for granular base material and 1.350 for fine-grained materials of the subgrade.
h_{soil} = Thickness of unbound soil layer under the HMA, it varies with pavement section depending on section design of the section. Based on NYSDOT pavement design manual, h_{soil} is in the range of 300 mm (11.81 inches) to 900 mm (35.43 inches)
ε_v = Average vertical resilient or elastic strain calculated at the soil layer from mechanistic analysis of pavement section, in this report determined by KENLAYER program
n = number of cycles of the axle group with vertical elastic strain ε_v

The parameters, β, ρ, ε_o and ε_r are material properties of the soil where β is obtained from MEPDG guide Equation (5-2b):

$$Log(\beta) = -0.6119 - 0.017638(W_c) \quad (4.26)$$

where: W_c = Water content in soil, chosen to be 5%.

A sensitivity analysis shows that the effect of W_C on the results of equation (4.26) is negligible. MEPDG guide Equation (5-2c) gives the parameter ρ:

$$\rho = 10^9 \left(\frac{C_0}{1-(10^9)^\beta}\right)^{\frac{1}{\beta}} \tag{4.27}$$

where: C_0 $= Ln(0.0075)$

ε_0 = Intercept determined from the laboratory repeated load permanent deformation tests, in/in

ε_r = Resilient strain imposed in the laboratory test to obtain material properties

For the purposes of this study, typical values for both the intercept and resilient laboratory strain were determined from available sources. Specifically, a value $\varepsilon_r = 451.7 \times 10^{-6}$ is extracted from NCHRP REPORT 547 (Witczak, 2005).

The ratio $\frac{\varepsilon_0}{\varepsilon_r}$ can be calculated using the following equation (FHWA, 2006):

$$Log\left(\frac{\varepsilon_0}{\varepsilon_r}\right) = 0.74168 + 0.08109W_c - 0.000012157M_R \tag{4.28}$$

where: M_R = Resilience Modulus= 39,000 psi

Preliminary investigations of the results of Eq. (4.25) for the range of pavement layer thicknesses applicable to this study indicated that the permanent deformation of the soil layers is small compared to the HMA deformations. Furthermore, once the soil consolidates no additional permanent deformations are expected during subsequent corrective maintenance interventions. For these reasons, Eq. (4.25) was not implemented in the analyses performed in this study.

KENLAYER Program for Stress and Strain Analysis

As observed from the above explanations, an important component of the MEPDG approach for assessing the damage to flexible pavements is the determination of the strains at different critical locations of a pavement section due to the crossing of the different axle groups of heavy vehicles. Specifically, the MEPDG approach for analysis of flexible pavements requires the following strains:

- Horizontal tensile strain at the top of HMA layer to obtain fatigue cracking
- Horizontal tensile strain at bottom of HMA layer to obtain alligator cracking
- Vertical strain at mid-height of each layer to obtain rutting

The evaluation of these strains requires a mechanistic analysis or a finite element program. In this study, the strains induced by each axle load group of each crossing truck are calculated using the program KENPAVE developed by Huang (2004). The program KENPAVE has two components: KENLAYER for flexible pavements and KENSLAB for rigid pavements.

KENLAYER Input Variables

Because it is impossible to analyze the millions of trucks that are expected to travel over each possible pavement section, this study develops a library of strain data for each critical point within a typical pavement section induced by different types of axle group loads. Specifically, the strain library was assembled for the following four main input pavement and load parameters:

1. Axle group and Axle weight: The effect of a truck on a pavement section depends on the truck axle configuration and its axle weights. Truck tires and axles that are separated by relatively short spacings have combined effects on each pavement cross section. But, when the spacing between consecutive axles exceeds 9 ft, their effects can be analyzed separately. Therefore, the following axle groups are identified in this study for KENLAYER analysis:

    - Single-axle-single-tire (steering axle)
    - Single-axle-dual-tire
    - Tandem axles
    - Tridem axles

    It should be noted that since HMA is a viscoelastic material, which means that scaling the effects of an axle group based on load intensity is not be possible. Therefore, the required strains are calculated for different weights of each axle group.
2. Temperature: Because HMA is a viscoelastic material, its response to applied axle loads depends on its temperature. Therefore, the ambient temperature is used as an independent parameter for calculating the required strains in flexible pavements under the effect of each axle group type for different group weights. The weight ranges considered during the KENLAYER analysis are summarized in Table 4.5.
3. Hot Mix Asphalt (HMA) thickness: Based on NYSDOT flexible pavement design guidelines, HMA thickness could range between 6.5 inches (165 mm) to 9.84 inches (250 mm).
4. Subgrade thickness: Based on NYSDOT flexible pavement design guidelines, subgrade thickness of flexible pavements could range between 11.81 inches (300 mm) to 35.43 inches (900 mm).

Note: As explained in Section 4.6 of this Report, the design of a flexible pavement section must have a minimum HMA thickness equal to 6.5 in (165 mm) and a minimum subgrade thickness of 11.81 in (300 mm). If the section is not adequate with the minimum thickness of each layer for the calculated ESAL, the HMA thickness is increased while keeping the subgrade thickness constant at the minimum value. Once the HMA thickness has reached the maximum allowed value of 9.84 in (250 mm), the subgrade thickness is increased until the design requirements are satisfied. Thus, to consider all possible pavement profiles, it is not necessary to scan all combinations of HMA and subgrade thicknesses. The ranges for the HMA and subgrade thicknesses and the temperatures considered during the KENLAYER analysis are listed in Table 4.6.

Table 4.5 - Load Range for Each Axle Load Group

Load Group	Min (lbs)	Max (lb s)	Interval (lbs)
Single Axle	2000	72000	2000
Tandem Axle	2000	144000	2000
Tridem Axle	2000	204000	2000

Table 4.6 - Range for HMA thickness, Subgrade thickness and temperature

	Min	Max	Interval
HMA thickness	6.5"	10"	0.5"
Subgrade thickness	10"	36"	2"
Temperature	10 °F	90 °F	5 °F

KENLAYER Input Constants

In addition to the input variables identified in the previous paragraph, the analysis of each flexible pavement section requires the input of several material and geometric characteristics. Table 4.7 below summarizes all the input parameters and the values that have been used for each during the KENLAYER analysis of flexible pavements performed in this study.

Table 4.7 - Summary of KENLAYER Input parameters

Input Parameter	Value	Remarks/ Reference
Material Type **(MATL)**	3	1 = Linear 2 = Nonlinear 3 = Viscoelastic 4 = Combined MATL=3 because Hot Mix Asphalt is a viscoelastic material
Number of periods per year (NPY)	1	NPY=1 since each truck is analyzed only once
Number of Load groups **(NLG)**	1	Since we are analyzing one load group at a time, NLG=1. Type of load group is entered separately see below.
Tolerance for Numerical integration **(DEL)**	0.001	Lower values would unnecessarily increase program running time
Number of layers **(NL)**	5	Total number of layers; will be always 5 but thickness might vary
Number of Z coordinates for analysis **(NZ)**	6	"Z" axis is vertical axis. Strain is calculated at top and bottom of HMA layer, mid-height of HMA, base, subbase and subgrade.
Maximum cycles of numerical integration **(ICL)**	80	Higher values would unnecessarily increase program running time

Type of responses **(NSTD)**	9	1=displacement only 5=plus stresses 9=plus strains
Systems of units **(NUNIT)**	0	0=US units 1=SI units
Thickness for each layer **(TH)**	Varies	Depends on section design. Thicknesses of layers 1 and 4 vary. Thicknesses of layers 2 and 3 remain constant. Thickness of layer 5 (foundation soil) is not needed since default is unbounded thickness.
Poisson's ratio for each layer **(PR)**	0.35 0.35 0.35 0.35 0.35	Use same as Asphalt Treated Base, values based on: Page 186 (Masada et al, 2004) Page 186 (Masada et al, 2004) Page 186 (Masada et al, 2004) Page 204 (Masada et al, 2004)
All layer interfaces bonded **(NBOND)**	0	1=Yes 0 = No
Elastic Modulus of each layer **(E)**	0 Varies 39050 psi 39050 psi 34000 psi	Assign 0 for viscoelastic material Equation (4.12) of this report Page 204 (Masada et al, 2004) Page 199 (Masada et al, 2004) Page 211 (Masada et al, 2004) Page 201 (Masada et al, 2004)
Type of Load **(LOAD)**	Varies	0=Single-axle-single-tire 1=Single-axle-dual-tire 2=Tandem-axle 3=Tridem-axle
Contact radius **(CR)**	6 (in)	Radius of tire footprint
Contact Pressure **(CP)**	Varies	Axle Group Weight/Number of tires in group/area of tire
Number of radial coordinates to be analyzed **(NR)**	1	Always analyzing point underneath the load
Radial Distance **(RC)**	0	Always analyzing point underneath the load
Duration of the load **(DUR)**	0.1	DUR=0.1 for moving load (Huang, 2004)
Number of viscoelastic layer **(NVL)**	1	Layer 1 of HMA is the only viscoelastic layer
Number of time duration for creep compliance **(NTYME)**	11	Recommendation from (Huang, 2004)
Times at which creep compliance are specified **(TYME)**	See Table 4.8	
Creep compliance of viscoelastic material at specified reference temperature **(CREEP)**	See Table 4.8	
Reference temperature of each viscoelastic layer at	70°F	(Huang, 2004)

which creep compliance have been specified **(TEMPREF)**		
Temperature shift coefficient **(BETA)**	0.113	(Huang, 2004)
Pavement temperature of each viscoelastic material **(TEMP)**	Varies	Taken as ambient temperature for the month truck crosses the section
Number of layers for bottom tension **(NLBT)**	0	Damage analysis is not performed.
Number of layers for top compression **(NLTC)**	0	Damage analysis is not performed
Layer number for damage analysis of bottom tension **(LNBT)**	0	Tensile strains are obtained at mid-depth of each layer, individual coordinates have been specified
Layer number for damage analysis of bottom tension **(LNTC)**	0	No damage analysis performed
Total number of load repetitions **(TNLR)**	1	Each axle load group is repeated only once
Fatigue coefficients (**FT1,FT2,FT3**)	0.0796 3.291 0.854	From Huang (2004) page 98 (Not used in IRI calculation but minimum of 1 layer must be provided)
Permanent deformation coefficients (**FT4,FT5**)	1.364E-09 4.477	Huang (2004) page 98 (Not used in IRI calculation but minimum of 1 layer must be provided)

Creep compliances are used to define the viscoelastic material properties of HMA. The creep compliances and respective times for the reference temperature chosen in Table 4.7 are presented in Table 4.8 based on the data provided by Huang (2004).

Table 4.8 – Creep Compliances for HMA

Time (Seconds)	Creep Compliance
0.001	2.50E-06
0.003	3.51E-06
0.01	5.81E-06
0.03	9.80E-06
0.1	1.69E-05
0.3	2.70E-05
1	4.19E-05
3	5.81E-05
10	8.11E-05
30	1.08E-04
100	1.282E-05

4.7.2. MEPDG Method for Rigid pavements (JPCP)

Following the same approach described above for flexible pavements, the MEPDG method estimates the damage to rigid pavements using an empirically derived equation for IRI. Specifically, IRI for Jointed Plain Concrete Pavements (JPCP) can be estimated using Equation (5-32a) in the MEPDG guide (AASHTO, 2008)

$$IRI = IRI_0 + C1 * TCRACK + C2 * SPALL + C3 * TFAULT + C4 * SF \quad (4.29)$$

where: IRI = Predicted IRI, inch/mile

IRI_0 = Initial smoothness measured as IRI, inches/mile

$TCRACK$ = Percent slabs with transverse cracks (all severities)

$SPALL$ = Percentage of joints with spalling (medium and high severities)

$TFAULT$ = Total joint faulting accumulated per mile, in inches

SF = Site factor

$C1$ = 0.8203

$C2$ = 0.4417

$C3$ = 0.4929

$C4$ = 25.24

The IRI is calculated for JPCP rigid pavement sections using five different input parameters. IRI_0 is the estimate of smoothness of the section when the section is newly built or immediately after maintenance is performed, SF and $SPALL$ are related to the age of the pavement and are thus calculated once a year. The remaining parameters, CRK and $TFAULT$ are the accumulations of damage obtained from the crossing of each truck. Explanations regarding the equations for finding each parameter are given below.

Initial and Terminal IRI

For a rigid pavement section, an initial serviceability index PSR= 4.5 is usually used. The design of a pavement is usually performed with the goal of reaching a terminal PSR=2.5. As was the case with flexible pavement, executing corrective maintenance when the PSR reaches 3.8 will ensure a smooth ride and increases the durability of the pavement. Using Eq. (4.3), the initial and terminal IRI for rigid pavement sections can be calculated as:

$$IRI_0 = -\frac{1}{0.0041}\ln\left(\frac{4.5}{5.0}\right) = 25.69$$

$$IRI_{Corrective\ Maintenance} = -\frac{1}{0.0041}\ln\left(\frac{3.8}{5.0}\right) = 67$$

Similar to the flexible pavement procedure, the initial IRI_0 is defined for a new pavement when it is first opened for traffic and the same initial IRI_0 is used after each corrective maintenance cycle. It is also noted that the IRI criterion for corrective maintenance ($IRI_{Corrective\ maintenance}$ =67) corresponds to the upper limit for pavement conditions associated with very smooth rides as defined in NYSDOT (2014).

Site Factor (SF)

The Site Factor in Eq. (4.29) can be calculated using Equation (5-32b) in AASHTO MEPDG guide (AASHTO, 2008):

$$\mathrm{SF} = \mathrm{AGE}(1 + 0.5556 * \mathrm{FI})(1 + \mathrm{P}_{200}) * 10^{-6} \tag{4.30}$$

where: AGE = Pavement age in years

FI = Freezing index, ºF days and

P_{200} = Percent subgrade material passing No. 200 sieve

In this report, P_{200} was chosen to be 10% or 0.1. The freezing index for New York is taken as 753°F days.

Spalling (SPALL)

Percentage of joints with spalling can be calculated using Equation (5-33a) in AASHTO MEPDG guide (AASHTO, 2008):

$$\mathrm{SPALL} = \left(\frac{\mathrm{AGE}}{\mathrm{AGE}+0.01}\right)\left(\frac{100}{1+1.005^{-12*\mathrm{AGE}+\mathrm{SCF}}}\right) \tag{4.31}$$

where: SPALL = percentage joints spalled (medium- and high-severities)
AGE = pavement age since construction, years
SCF = scaling factor based on site, design and climate-related variables

The scaling factor, SCF, in Equation (4.31) above is calculated using Equation (5-33b) in MEPDG guide (AASHTO, 2008):

$$\mathrm{SCF} = \begin{matrix} -1400 + 350 * \mathrm{AC_{PCC}} * (0.5 + \mathrm{PREFORM}) + \ 3.5\,\mathrm{f'c} * 0.4 \\ -0.2\left(\mathrm{FT_{cycles}} * \mathrm{AGE}\right) + 43\,\mathrm{H_{PCC}} - 536\,\mathrm{WC_{PCC}} \end{matrix} \tag{4.32}$$

where: $\mathrm{AC_{PCC}}$ = PCC air content, percent
AGE =time since construction, years
$\mathrm{PREFORM}$ =1 if preformed sealant is present; 0 if not
$\mathrm{f'c}$ = PCC compressive strength, psi
$\mathrm{FT_{cycles}}$ = average annual number of freeze-thaw cycles
$\mathrm{H_{PCC}}$ = PCC slab thickness, in
$\mathrm{WC_{PCC}}$ = PCC water/cement ratio

The average $\mathrm{AC_{PCC}}$ is 6 percent (AASHTO, 2007). The PCC compressive strength specified in NYSDOT manual is f'_c=3,000 psi. The average number of freeze thaw cycles in New York is 39 cycles (WorldNow, 2014). The PCC water to cement ratio is typically 0.45 (AASHTO, 2007). For the parameter PREFORM, a value of 1 is used since sealant is always present between slabs.

Faulting (TFAULT)

Faulting at joints is the difference of elevation across a joint between slabs. It is caused in part by a buildup of loose materials due to pumping or a depression under the slabs. The AASHTO MEPDG guide Equation (5-20a) measures mean joint faulting at the end of month m, $Fault_m$ from the equation:

$$\mathrm{Fault_m} = \sum_{i=1}^{m} \Delta\mathrm{Fault_i} \tag{4.33}$$

where: $Fault_m$= mean joint faulting at the end of month m, in inches
$\Delta Fault_i$= incremental change (monthly) in mean transverse joint faulting during month i, inches

The Differential Energy (DE) between the displacements at the corners of adjacent slabs at the crossing of an axle load is a primary factor for evaluating faulting. DE is calculated using AASHTO MEPDG guide Equation (5-23a):

$$\mathrm{DE} = \frac{\mathrm{K}}{2}(\mathrm{DL}^2 - \mathrm{DU}^2) \tag{4.34}$$

where: K = subgrade reaction
DL = deflection of corner of loaded slab
DU = deflection of corner of unloaded slab

The incremental fault is obtained using equation (5-20b) of AASHTO MEPDG guide:

$$\Delta\mathrm{Fault_i} = \mathrm{C_{34}}(\mathrm{Faultmax_{i-1}} - \mathrm{Fault_{i-1}})^2(\mathrm{DE_i}) \tag{4.35}$$

where: $\Delta Fault_i$ = incremental change (monthly) in mean transverse joint faulting during month i, in inches
$DE_{\ i}$ = differential density of energy of subgrade deformation accumulated during month i
$Fault_{i-1}$ = mean joint faulting at the end of the previous month (=month i-1), in inches
$FAULTMAX_i$ = maximum mean transverse joint faulting for month i, in inches which can be calculated using AASHTO MEPDG equation (5-20c) as given below

$$\mathrm{Faultmax_i} = \mathrm{Faultmax_0} + \mathrm{C_7}\left[\sum_{j=1}^{m}\mathrm{DE_j}\left(\log\left(1 + 5.0^{\mathrm{EROD}}\mathrm{C_5}\right)^{\mathrm{C_6}}\right)\right] \tag{4.36}$$

where: $FAULTMAX_i$ = maximum mean transverse joint faulting for month i, inches
$FAULTMAX_0$ = initial maximum mean transverse joint faulting, inches
$DE_{\ j}$ = maximum differential density of energy of subgrade deformation during month j. It is found by considering maximum deflection of critical truck for the month j.
EROD = base/subbase erodibility factor (equal to 3)

Initial maximum mean transverse joint faulting in Eq. (4.37) above is given by AASHTO MEPDG Equation (5-20d):

$$\mathrm{Faultmax_0} = \delta_{\mathrm{curling}}\mathrm{C_{12}}\left[\log\left(1 + 5.0^{\mathrm{EROD}}(\mathrm{C_5})\right)\log\left(\frac{\mathrm{WetDays}(\mathrm{P_{200}})}{\mathrm{P_s}}\right)\right]^{\mathrm{C_6}} \tag{4.37}$$

Where: EROD = base/subbase erodibility factor (index of 3)
$\delta_{curling}$ = maximum mean monthly slab corner upward deflection PCC due to temperature curling and moisture warping
P_s = overburden on subgrade, psi where $\mathrm{P_s} = \sum \mathrm{H} \times \gamma$ with H being the thickness of each layer supported by the pavement subgrade and γ is the specific weight
P_{200} = percent subgrade material passing No. 200 sieve (10 % typical in NYS)
WetDays = average annual number of wet days (greater than 0.1 inch rainfall) (121 days)

The global calibration constants recommended by AASHTO MEPDG guide (AASHTO, 2008) C_1, C_2, C_3, C_4, C_5, C_6,C_7, C_{12}, C_{34} are listed in Table 4.9. AASHTO recommends that these constants be calibrated based on local and state-specific calibration process. But, lacking such data, the values given by AASHTO (2008) are used in this study.

Table 4.9 - Calibration Constants for Faulting Equations

Global Calibration Constant	Value provided in AASHTO MEPDG manual
C_1	1.29
C_2	1.1
C_3	0.001725
C_4	0.0008
C_5	250
C_6	0.4
C_7	1.2
C_{12}	See equation (4.39)
C_{34}	See equation (40)

Two constants, C_{12} and C_{34} in Table 4.9 are calculated using AASHTO MEPDG Equations (5-20e) and (5-20f) respectively:

$$C_{12} = C_1 + C_2 * FR^{0.25} \tag{4.38}$$

$$C_{34} = C_3 + C_4 * FR^{0.25} \tag{4.39}$$

where: FR = Base freezing index defined as percentage of time the top base temperature is below freezing temperature.

The environmental impact on faulting is accounted for through delta curling, which is the upward corner deflection due to moisture and temperature. KENSLAB is used to find curling.

The total faulting per mile is obtained using:

$$TFAULT = \frac{Fault_m}{slab\ length\ (\text{in miles})} \tag{4.40}$$

Cracking (TCRACK)

The rigid slab may exhibit bottom up and top down cracking due to the tensile stresses incurred due to the passing of heavy trucks. The development of such cracks is due to cyclic fatigue from the application of large numbers of loading cycles. The fatigue Damage Index, DI_i, due to a single tensile stress cycle, i, is obtained using Equation (5-17a) of AASHTO MEPDG guide (AASHTO, 2008):

$$DI_i = \frac{1}{N_{f-conc}} \tag{4.41}$$

where: N_{f-conc} = hypothetical number of stress cycle having the same intensity, i, that would cause pavement failure in concrete rigid pavements.

N_{f-conc} is calculated using equation (5-17b) of AASHTO MEPDG guide (AASHTO, 2008):

$$\log(N_{f-conc}) = C_1 \left(\frac{\mathrm{MR}}{\sigma}\right)^{C_2} \tag{4.42}$$

where: $\mathrm{N_{f-conc}}$ = allowable number of stress cycle applications for the concrete pavement

σ = applied stress in psi

C_1 = 2.0

C_2 = 1.22

MR = Modulus of rupture of concrete= $7.5\sqrt{\mathrm{f'_c}}$ =$7.5\sqrt{3000}$ = 410.79 psi

The damage index, DI_i, for each stress cycle is then summed for all stress cycles caused by one truck and for all trucks that cross the pavement section being analyzed during the service period to obtain the cumulative damage:

$$\mathrm{DI} = \sum_{k=1}^{N_{truck}} \sum_{i=1}^{n_{stress\ cycles\ for\ truck\ k}} (DI_i)_k \tag{4.43}$$

The contribution of DI to cracking is found using Equation (5-16) of AASHTO MEPDG guide (AASHTO, 2008):

$$\mathrm{CRK} = \frac{1}{1+0.6\ \mathrm{DI_F}^{-2.05}} \tag{4.44}$$

Two types of cumulative damages are calculated: bottom up and top down cracking. The contributions to cracking from top-down and bottom up cracking can be calculated as a percent using Equation (5-18) of AASHTO MEPDG guide (AASHTO, 2008) as shown below:

$$\mathrm{TCRACK} = (\mathrm{CRK_{Bottom-up}} + \mathrm{CRK_{Top-down}} - \mathrm{CRK_{Bottom-up}} * \mathrm{CRK_{Top-down}}) * 100 \tag{4.45}$$

KENSLAB Program for Stress and Deflection Analysis

As observed from the above explanations, an important component of the MEPDG approach for assessing the damage to rigid pavements is the determination of the stresses and deflections at different critical locations of a pavement due to the crossing of heavy vehicles and the curling due to temperature changes. Specifically, the MEPDG approach for analysis of rigid pavements requires the following stresses:

- Horizontal tensile strain at the top of rigid slabs to obtain top down fatigue cracking.
- Horizontal tensile strain at bottom of rigid slabs to obtain bottom up cracking
- Vertical deflections at corners of adjacent slabs to obtain differential energy
- Curling of slabs due to temperature differentials

The evaluation of these stresses and deflections requires a mechanistic analysis or a finite element program. In this study, the stresses and deflections induced by each axle load group of a crossing truck are calculated using the program KENPAVE developed by Huang (2004). The program KENPAVE has two components: KENLAYER for flexible pavements and KENSLAB for rigid pavements. Because rigid pavements are treated as linear elastic materials, the principle of superposition is valid and is applied in this study. Therefore, the analysis of stresses due to truck crossings can be performed simply by analyzing single and dual tire axles having unit load intensities which is used to develop an influence line to find the maximum stress at a particular point of a pavement cross section. Using the principle of superposition the influence line is scaled for different axle load values. The principle of superposition uses the scaled up influence lines obtained for each axle to calculate the total stress due to a given truck. An example set of influence lines used to find the stress at the top of the slab at its midlength are provided in Figure 4.10. Figure 4.11 shows how the influence lines can be superposed to provide the time history of the stress at the top of the slab at midlength due to the crossing of a semi-trailer truck with a split rear tandem. The plot in Figure 4.10 shows 5 compressive stress cycles at the top of the slab and 4 tensile stress cycles where the crossing of the zero stress line defines a cycle. Thus, top down cracking is evaluated using the four tensile stress cycles because compressive stresses do not cause fatigue fracture. It is noted that due to symmetry in unbonded rigid slab sections, the same stress history shown in Figure 4.11 is valid with opposite signs at the bottom of the slab. Thus, bottom up cracking will be evaluated using 5 stress cycles.

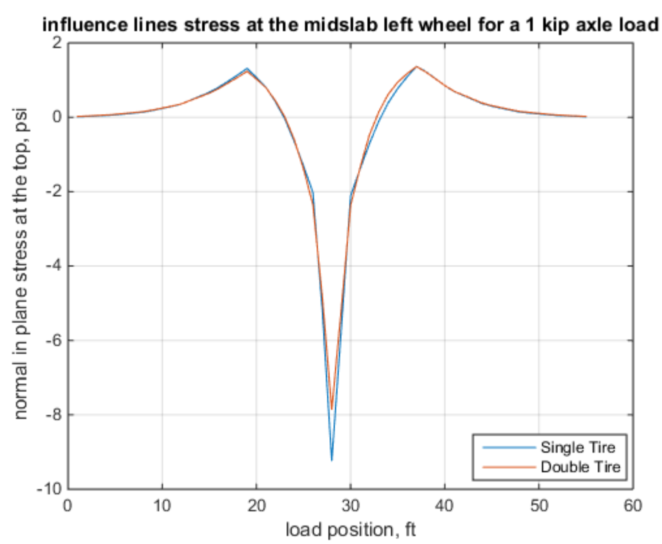


Figure 4.10 - Example Influence line for maximum stress at top of slab at midlength

Figure 4.11 - Example Influence line for maximum stress at top of slab at midlength

KENSLAB Input Variables

Because it is impossible to analyze the millions of trucks that are expected to travel over each possible pavement section, this study uses KENSLAB to develop influence lines for each critical point within a typical pavement section induced by different types of axle groups. Specifically, the stress influence lines are assembled for single-axle-single-tire (steering axle) and single-axle-dual-tire (all other axles) for different slab thicknesses. Also, deflections at the corners of adjacent slabs were obtained from KENSLAB for unit loads of the different types of axle groups including single-axle-single-tire, single-axle-dual-tire, tandem axles and tridem axles.

KENSLAB Input Constants

The calculations of the influence lines as well as the deflections for each typical rigid pavement section are executed using the program KENSLAB which requires the input of several material and geometric characteristics. Table 4.10 summarizes all the input parameters and the values that have been used for each KENSLAB analysis.

Table 4.10 - Summary of KENSLAB Input parameters

Input Parameter	Value	Remarks/ Reference
Type of Foundation **(NFOUND)**	**1**	0=Liquid or Winkler foundation 1=Solid foundation 2=Burmister's foundation
Damage Analysis **(NDAMA)**	0	0=No damage analysis 1=Damage analysis based on PCA fatigue cracking criteria 2=Damage analysis based on user specified fatigue coefficients (We are using MEPDG method, we only need stresses at certain critical locations, we perform our own damage analysis)
Number of period per year **(NPY)**	1	Since each trucks/ load group is analyzed once, number of period is always 1
Number of Load group **(NLG)**	1	Since we are analyzing one load group at a time, number of load group is 1.
Total number of slab **(NSLAB)**	6	A six-slab model is chosen for analysis to account for interaction of loaded slab with adjacent slabs (see Figure 4.12).
Total number of Joints **(NJOINT)**	7	When 6 slabs are arranged in 2 rows and 3 columns, 7 joints are needed to connect slabs to each other, see Figure 4.12
Number of X-coordinate for each slab (NX)		depends on mesh size
Number of Y-coordinate for each slab (NY)		depends on mesh size
Total number of layer **(NLAYER)**	2	Two layers are analyzed, concrete slab and cement treated subbase
Joint number **(JONO1,JONO2,JONO3,JONO4)**		See Figure 4.12 in this report.
Nodal number used to check convergence **(NNCK)**		Node used for calculating stresses was used as check node
Number of nodes NOT in contact **(NOTCON)**	0	NOTCON is always equal to zero when NCYCLE = 1.
Number of nodes with initial gaps **(NGAP)**	0	Assign 0 when NCYCLE = 1
Number of nodes for stress printout **(NPRINT)**	1	We are interested in stresses at a given location, at midlength of slab 3 , right underneath one of the loads
Maximum number of cycles for checking subgrade contact **(NCYCLE)**	1	USE 1 for full contact
Input of gaps from previous problem **(INPUT)**	0	0=No 1 = Yes
Bond between two slab layers **(NBOND)**	0	0=Unbonded 1=Bonded
Temperature Curling **(NTEMP)**	0 or 1	0=No, 1=Yes. This variable decides if or not curling is to be considered.

		When analyzing the truck, this is set to 0 (No) but when calculating curling of the slab, this is set to 1 (Yes)
Weights of the Slab **(NWT)**	0 or 1	0=No – stress cycle analysis does not require slab weight 1=Yes – curling analysis requires slab weight
Number of nodes with different thicknesses of slab layer 1 **(NAT1)**	0	All 6 slabs will have same thickness
Number of nodes with different thicknesses of slab layer 2 **(NAT2)**	0	All 6 slabs will have same thickness
Number of nodes on X axis of symmetry **(NSX)**	0	We are looking at entire slab, no need to use any symmetry
Number of nodes on Y axis of symmetry **(NSY)**	0	We are looking at entire slab, no need to use any symmetry
More detail printout **(MDPO)**	0	0=No, 1=Yes Detail print out is not required as long as we get our output
Systems of units **(NUNIT)**	0	0=English 1=SI
Uniform Load **(UL)**	0	No uniform load is considered except truck load
Temperature Gradient **(TC)**	0	No temperature gradient was considered
Concentrated Load **(CL)**	2	Indicate, with anything other than 0 that there is a concentrated load
Temperature Differential between top and bottom **(TEMP)**	0 or Varies	For truck analysis, this is set to zero because effects of curling are not combined. When calculating curling deformation, it is calculated based on ambient temperature.
Unit weight of each layer **(GAMA)**	150 150	Same unit weight is used for slab and cement treated base.
Modulus of rupture of pavement slab for each layer **(PMR)**	0 0	Not required if no damage analysis is to be performed (NDAMA=0)
Poisson's ratio of each layer **(PR)**		
Coefficient of Thermal expansion **(CT)**	0.00055	Default per Huang (2004)
Tolerance for iteration **(DEL)**	0.001	Default per Huang(2004)
Maximum allowable deflection **(FMAX)**	1	
Fatigue properties for each layer **(F1,F2)**	0, 0 0, 0	Not required if no damage analysis is to be performed (NDAMA=0)
X-coordinate of each slab **(X)**		
Y-Coordinate of each slab **(Y)**		
Thickness of each slab layer **(T)**	Varies 3.94	Concrete slab thickness vary base thickness is 3.94 inches (100 mm)
Poisson's ratio of each layer **(PR)**	0.20 0.22	For concrete and cement treated base
Young Modulus **(YM)**	3122018 (psi) 1000000 (psi)	For slab E=57000*sqrt(3000) = 3122018 for concrete slab For cement treated base 1E+6 as per Huang (2004)
Number of uniformly distributed load for each load group **(NUDL)**	0	No uniform load is considered since UL = 0. Also, only 1 zero value because we are looking at one load group at a time to create out strain database.
Number of concentrate nodal force	2	Single-Axle-Single-tire

(NCNF)	4	Single-Axle-dual-tire (NOTE: This is HOW MANY concentrated force, actual force value is next and we are only creating stress database single and dual tire, rest all comes from influence line
Number of nodal moment in X-direction **(NNMX)**	0	No concentrated moment need to be applied except truck load
Number of nodal moment in Y-direction **(NNMY)**	0	No concentrated moment need to be applied except truck load
Nodal number at which load is applied **(NN)**	Varies	Different nodes along the truck load path are loaded as the truck moves to create strain database. NOTE: total number of nodes here must be equal to NCNF specified.
Concentrated force applied at the given node **(NCNF)**	1000	1kip=1000 lbs is used for influence lines.
Nodal number at which stresses are computed and printed **(NP)**	167	This point is on the load path as well as is the mid-point of the slab
Foundation Seasonal Adjustment factor **(FSAF)**	1	No adjustment need to be made
Young Modulus of subgrade **(YMS)**	30,000 psi	
Poisson's ratio of subgrade **(PRS)**	0.25	Pending reference
Young Modulus of steel bars **(YMSB)**	29000000	ACI
Poisson's ratio of steel bars **(PRSB)**	0.3	ACI
Spring constant for shear transfer **(SPCON1)**	0	load transfer between slabs is due to dowels at joints
Spring constant for moment transfer **(SPCON2)**	0	load transfer between slabs is due to dowels at joints
Modulus of dowel support **(SCKV)**	1500000	Minimum value suggested by KENSLAB
Dowel bar diameter **(BD)**	1.64 in	Bar No. 14 is assumed
Dowel bar spacing **(BS)**	12	Common practice
Width of joint **(WJ)**	0	Not critical
Gap between concrete and dowel **(GDC)**	0	Dowel assumed fully bonded to the slab

A six-slab model was used for the analysis of rigid pavements when using KENSLAB. Dimensions, slab numbers and joint numbers for each slab are shown in Figure 4.12. The analysis of stresses was performed for the section at midlength of Slab 2 under the path of the internal wheel. The truck is assumed to travel in the outer lane at 3 ft from the internal edge of the slab.

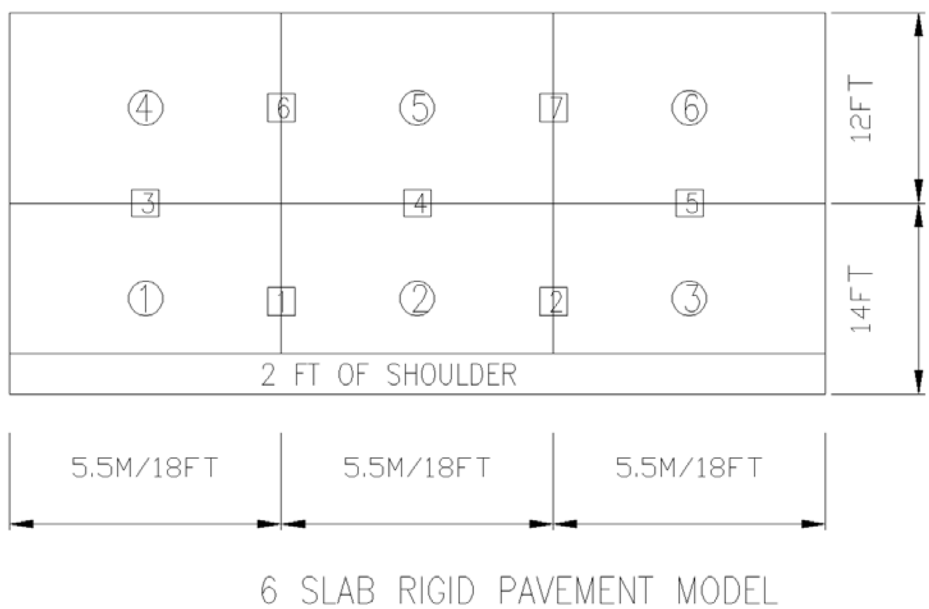


Figure 4.12 - Six-slab Model for Rigid Pavement Analysis

4.8 MEPDG-Based Corrective Maintenance Procedures

Highway pavements usually undergo corrective maintenance on a regular schedule which in NY is typically at 8-yr to 13-yr cycles although in some instances, pavement repairs may be necessary after longer increments of time. As shown in Figure 4.13, performing corrective maintenance when the pavement condition is at the edge of being classified as good rather than very good will prevent its potential rapid deterioration which would require more extensive and expensive repairs. It is estimated that this condition is associated with an IRI=67.

If properly maintained, the life cycle performance of the pavement may be represented as shown in Figure 4.14. The NYSDOT aims to keep its pavements in good condition with a goal of increasing their service life even beyond the typically specified 50-yr design life. This goal seems to be achievable as indicated by a NYSDOT (2004) report on the condition of NYS highways which shows that in the year 2004 about 25% of pavements were in need for crack sealing, almost 34% were in need of preventive maintenance paving, about 28% needed corrective maintenance, close to 7% needed rehabilitation, and only about 0.3%, representing 96 lane miles, needed to be reconstructed. However, there currently is some concern that the increasing numbers of overweight vehicles on the state's highway system may be causing a more rapid deterioration of the pavement conditions than anticipated. The object of this report is to provide some insight on the level of damage caused by overweight trucks. This information can eventually help the NYSDOT develop longer term management permit issuance and enforcement policies to control the numbers of overweight trucks and plan for future maintenance of the highway infrastructure. As mentioned earlier, the approach taken in this study to evaluate the effect of overweight trucks consists of comparing the costs of pavement construction and maintenance assuming that no overweight trucks are travelling over the highway system to the cost for constructing and maintaining the system with current numbers of overweight trucks. The construction designs are based on the ESAL method which is typically followed by the NYSDOT while the times at which corrective maintenance should be executed are estimated based on the MEPDG method.

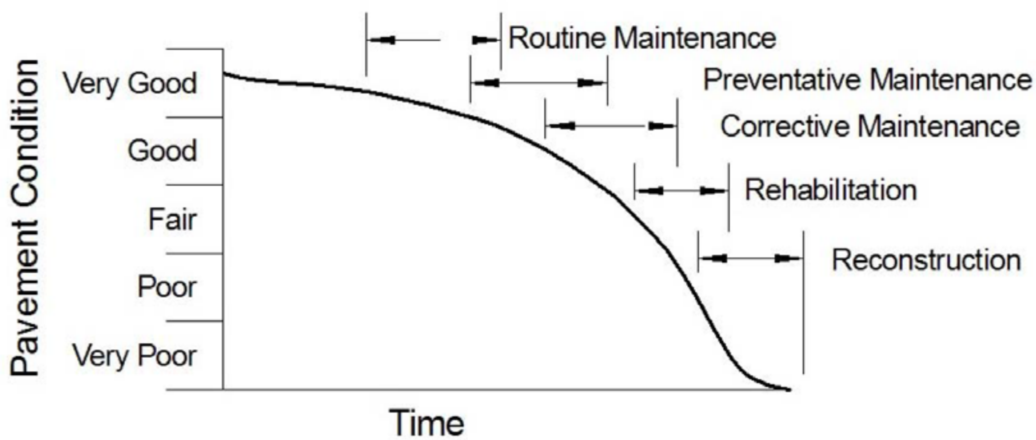


Figure 4.13 - Representation of pavement life-cycle

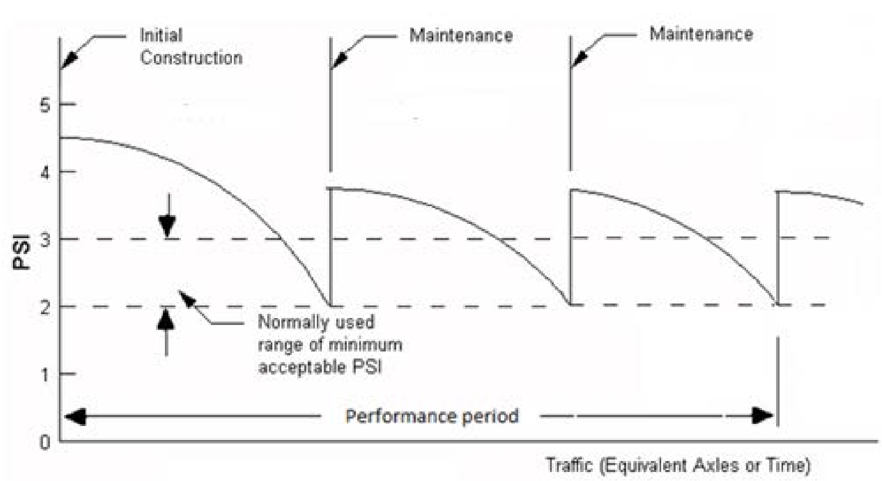


Figure 4.14 - Representation of pavement life-cycle

Applying the MEPDG approach was a challenging task because few reports and publications give sufficient details to illustrate the methodology in easy step-by-step procedures. Furthermore, the approach was calibrated on national data based on average performances of pavement throughout the US. The AASHTO MEPDG Manual (2004) recommends that the calibration be adjusted based on state, regional and local practice and conditions. Therefore, the results obtained in this report can best be used for comparative purposes rather than a clear indication of the existing pavement conditions.

For comparing the costs of constructing the pavements, it is sufficient to compare the cost of the materials as the costs of deployment and manpower do not vary significantly with minor changes in pavement sections. Specifically, the cost of concrete (C) is taken as $300 per ton. The cost for Hot Mix Asphalt (HMA) is $150 per ton. The Subgrade cost is $33.42 per cubic yard.

As per NYSDOT procedures, Corrective Maintenance applies to HMA and PCC pavements and includes for PCC: resealing joints, spall repair, grinding and isolated full-depth segment replacement; and for HMA: mill and fill, cold or hot in-place recycling with single course overlay. The unit cost for milling and recycled hot filling is about $12.00 per square yard according to the Means (2009) or about $100,000 per lane mile. A similar cost is also anticipated for corrective maintenance of concrete pavements.

The truck data used in the analysis presented in this Report was collected from the WIM station located on the I-88 Corridor. The analysis performed in this study compares the cost of pavement construction and maintenance due to the current truck weights including legal and overweight trucks to the cost that would be incurred in the case where the total cargo being carried by the overweight trucks is shifted to trucks within the normal weight limits. The redistribution of the cargo to not-overweight trucks is executed based on the ESALs. It is first noted that the ESALs vary even for trucks at the legal weight limits. For example, a Class 9 truck that meets the legal limits may produce 4.1 ESALs while another legal truck with the same GVW would produce a lower number of ESALs. Similarly, some of the overweight trucks may produce ESALs that are lower than those of legal trucks. This observation is illustrated as shown in Figure 4.15. Figure 4.15.a shows the entire distribution of ESALs for Class 9 vehicles including legal and overweight trucks from the I-88 WIM site. Figure 4.15.b separates the histogram of the legal trucks (in blue) from that of the overweight trucks (in red). The redistribution of the weights from the overweight trucks to the legal truck is executed for only the overweight vehicles that have ESALs higher than the highest ESAL of a typical legal truck. Figure 4.15.c shows the adjusted ESAL histogram for the Class 9 vehicles assuming that the cargo being carried by the overweight truck is reassigned to not-overweight trucks. The adjustment is executed for each vehicle class independently assuming that all the cargo of overweight trucks with ESALs higher than the maximum not-overweight truck ESAL are shifted to trucks of the same class. Table 4.11 shows that the maximum ESAL for a Class 9 legal (not overweight truck) is 4.1 for flexible pavements. The maximum value for rigid pavements is 5.0. Figure 4.15.c shows the concentration of the ESALs for flexible pavements at the maximum value of 4.1 after shifting the cargo of overweight trucks to not-overweight trucks. The shifting assumes that the tare weight for a typical Class 9 vehicle is 28,000 lbs. Table 4.11 lists the assumed tare weight for typical trucks in each vehicle class and the maximum gross vehicle weight that the not-overweight truck in each class can carry. The Table also lists the calculated maximum ESALs for not-overweigh trucks and the maximum ESALs for overweight trucks in each vehicle class.

Table 4.11 - Data Input for Redistribution of Weight from Overweight Trucks to Legal Trucks

Class	Assumed Truck Tare Weight (lb)	Maximum not overweight load (lb)	Maximum not Overweight Flexible ESAL	Maximum not overweight Rigid ESAL
5	24,000	45,000	4.4	4.9
6	27,000	57,100	3.5	4.6
7	32,000	71,000	3.0	4.0
8	26,000	74,000	4.8	5.4
9	28,000	80,000	4.1	5.0
10	29,000	80,000	3.6	4.2
11	28,000	80,000	4.9	5.4
12	32,000	80,000	3.2	3.4
13	36,000	80,000	2.1	3.3

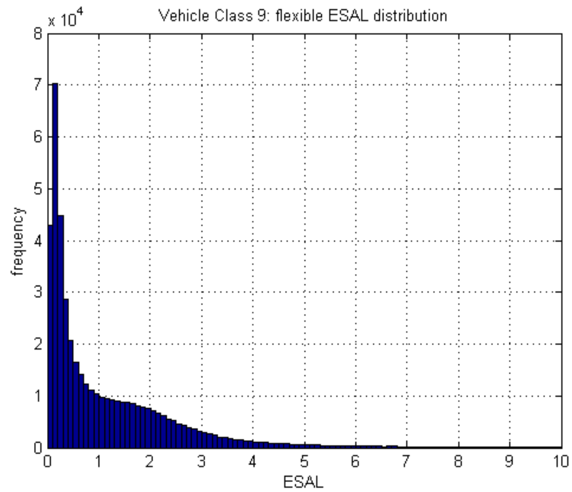


Figure 4.15.a – ESALs for all Class 9 Trucks

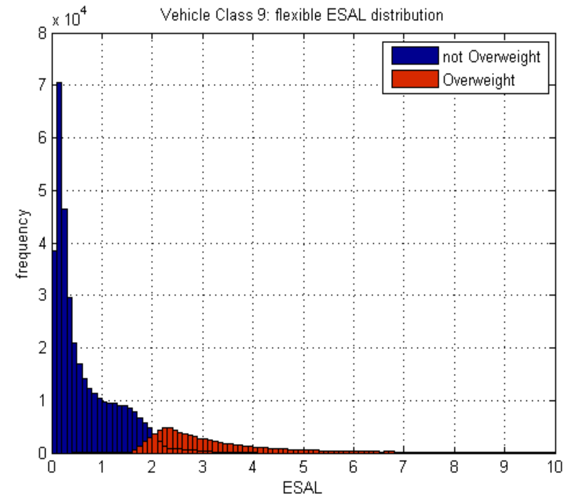


Figure 4.15.b ESAls for Legal (blue) and Overweight (red) Class 9 Trucks

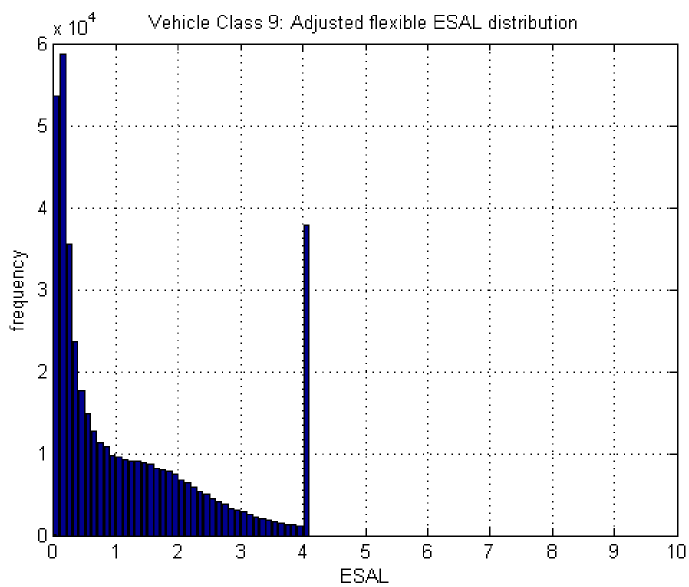


Figure 4.15.c: Histogram of adjusted ESALs to Carry all Current Class 9 Cargo on Legal Trucks Only

Because the MEPDG analysis is very computationally intensive, it is not possible to analyze every single truck that crosses over every pavement segment of the NYSDOT highway network. Even the analysis of a single corridor would require hours of computer time if it were to be analyzed for each truck crossing. For this reason, a sampling procedure is implemented under which the types of trucks are separated in terms of their FHWA Class type and their weight categories in terms of trucks that are not overweight (NO), trucks that have been issued permits to carry divisible loads (DV), those that were issued special hauling permits (SH) and those that are illegally overweight (IL). For example, Table 4.12 shows the distribution of trucks in percent in each class/weight category for the data collected at the I-88 WIM site. The trucks are further divided in terms of the travel month as some of the parameters in the MEPDG method are seasonally adjusted. Figure 4.16 compares the histogram of axle weights of the trucks in the I-88 WIM database to the histogram of the axles extracted from the trucks selected for analysis in this study. The sampling filled each class/weight category with 120 trucks. Figure 4.16 shows that using a more limited database would yield reasonably similar results to those obtained if the entire database were used.

Table 4.12 - Distribution of I-88 Trucks per Class/Weight Category

Class	5	6	7	8	9	10	11	12	13
NO	11.27%	8.59%	0.60%	3.81%	29.51%	2.21%	0.25%	0.10%	0.07%
DV	0.00%	0.76%	4.34%	0.06%	16.79%	4.55%	0.00%	0.00%	0.52%
SH	0.65%	0.03%	0.01%	0.13%	0.23%	0.24%	0.00%	0.00%	0.17%
IL	0.61%	2.69%	1.35%	0.37%	7.99%	1.70%	0.05%	0.02%	0.33%

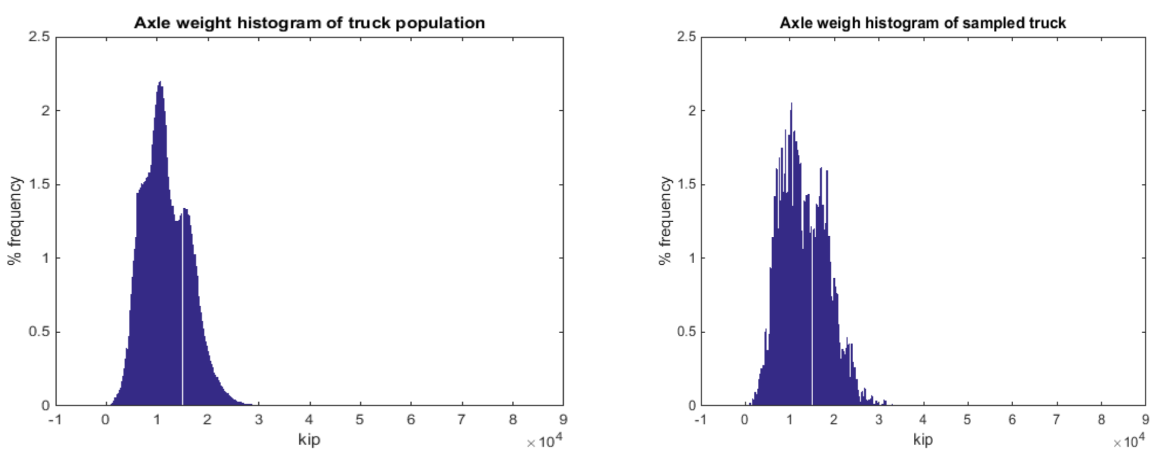


Figure 4.16 – Comparison of Original Axle Weight Histogram to the Sample Used for MEPDG Analysis

4.9 Implementation for I-88 Corridor

4.9.1 Effect of Overweight Trucks on Pavement Design

In this section, the steps followed for analyzing the pavement of the I-88 Corridor are described and the results of the analysis are presented. In the first step, Weigh-In-Motion (WIM) data from Station 9580 for the year 2011 were used to represent the truck classes and the truck axle weights for the entire I-88 Corridor. The location of WIM station 9580 and the I-88 corridor are shown in Figures 4.2 and 4.3.

The calculation of the ESALs for each truck in the WIM database is performed using Equations (4.1) and (4.2) for flexible and rigid pavements.

The truck data collected at the WIM station need to be adjusted to reflect the actual ADTT at each highway segment. The data has also to be adjusted for the number of lanes and extrapolated over the 50-year design life of the pavement. Accordingly, each section of the highway was associated with the ESALs calculated as:

$$ESAL_{SEC} = ESAL_{WIM} \cdot \frac{ADTT_{SEC}}{ADTT_{WIM}} \cdot D_F \cdot L_F \cdot \sum_{Y=1}^{Y=50} G_F \qquad (4.46)$$

$ESAL_{WIM}$ = Actual ESAL calculated from WIM data

$ADTT_{SEC}$ = Average Daily Truck Traffic for section

$ADTT_{WIM}$ = Average Daily Truck traffic from WIM station= 1552 for the I-88 WIM station.

$G_F =$ $(1 + anual\ growth)^\wedge(Y-1)$ = Compound growth factor, according to the NYS pavement design manual, one percent growth per year should be assumed.

Y = year (=50 for design life)

D_F = Directional Distribution Factor = 1.0 if WIM data for one-way traffic
= 0.5 if WIM data is for two-way traffic.

L_F = Lane Factor which converts one-direction truck numbers to trucks per main lane
= 1 for one-lane highways
= 0.85 for two-lane highways
= 0.7 for three-lane highways
= 0.6 for four or more lanes.
If the highway has different numbers of lanes in each direction, we assume that the larger value controls.

The results of the ESAL calculations for the WIM data are summarized in Table 4.13 in terms of the average ESALs per truck for rigid and flexible pavements. The data in Table 4.13 show that on average, each truck has 1.27 ESALs for flexible pavements, and 2.05 ESALs for rigid pavements. These values are comparable to the simplified ESAL values equal to 1.35 and 1.85 that have been proposed and used by NYSDOT as shown in Table 4.1. After redistributing the cargo of overweight trucks (labeled as Idealized) the ESALs drop slightly to 1.20 for flexible pavements and 1.80 for rigid pavements. The drop is not so severe because there will be more trucks on the highway system to carry the redistributed load.

Table 4.14 lists the total number of ESALs obtained for the entire WIM data divided by class/weight category. Four cases are shown: a) Flexible ESALs for original WIM data, b) Rigid ESALs for original WIM data, c) Flexible ESALs for idealized (redistributed) WIM data, and d) Rigid ESALs for idealized (redistributed) WIM data.

Table 4.13 Average ESALs per truck for I-88 Corridor

ESAL	Actual	Idealized
Flexible	1.27	1.20
Rigid	2.05	1.80

Table 4.15 gives the ESALs and the design pavement thicknesses for the I-88 segments for the actual and idealized (redistributed) load cases. The segments are classified in terms of the ESALs in the design (or main) lane. Thus, two categories of flexible pavements are observed those with 2000 trucks/day in the design lane, which consist of 16.54 miles and the 3.21 miles with 2500 trucks/day in the main lane. In both cases, the designs would lead to an HMA thickness of 9.84 in while the subgrade changes from 13.66 in to 15.66 in when the original WIM data is used. After load redistribution, the HMA thickness remains at 9.84 in and the subgrade thickness (SGSG) is slightly reduced to 13.19 in and 15.08 in for each group.

A larger number of main lane ADTT groups are observed for rigid pavements although the vast majority have design lane ADTTs that lie between 1500 to 2500 trucks per day. The design of rigid pavements changes only the slab thickness which varies between 8.86 in and 10.16 in. It is noted that the vast majority have a slab thickness equal to 8.86 in which is the smallest allowed thickness according to NYSDOT design standards.

Table 4.14 – Totals ESALs for I-88 Corridor per Class/Weight Category

	Vehicle Class								
a. ESAL Flexible Actual	5	6	7	8	9	10	11	12	13
NO	15780.9	8359.0	541.5	13855.4	228026.3	3441.6	11174.9	813.7	179.2
DV	0.0	1886.6	12269.7	218.7	208105.0	36505.7	0.0	3.1	8987.0
SH	2167.6	86.8	14.8	869.2	2946.8	3096.0	0.0	0.0	3872.2
IL	2000.2	4858.8	2865.0	3631.5	106410.1	17644.0	2823.4	740.6	5137.5

b. ESAL Rigid Actual	5	6	7	8	9	10	11	12	13
NO	15867.1	11377.3	919.1	14929.4	362314.7	5947.5	11209.5	905.8	285.2
DV	0.0	2904.5	24428.7	245.1	340669.6	72972.0	0.0	3.4	16853.3
SH	2266.8	136.7	26.5	1039.5	4627.2	6043.9	0.0	0.0	8482.2
IL	2117.6	7558.2	5672.8	4166.5	174144.5	34987.6	2888.5	810.3	9682.5

c. ESAL Flexible Adjusted	5	6	7	8	9	10	11	12	13
NO	15780.9	8359.0	541.5	13855.4	228026.3	3441.6	11174.9	813.7	179.2
DV	0.0	1246.6	11508.9	216.7	194269.2	36047.6	0.0	3.1	8201.5
SH	1851.6	71.1	12.1	866.6	2627.8	2631.4	0.0	0.0	3300.0
IL	1725.2	4336.1	2709.2	2609.5	89170.7	15456.6	2435.7	640.5	3854.2

d. ESAL Rigid Adjusted	5	6	7	8	9	10	11	12	13
NO	15867.1	11377.3	919.1	14929.4	362314.7	5947.5	11209.5	905.8	285.2
DV	0.0	1711.6	18127.8	237.9	292218.9	55483.6	0.0	3.4	13905.4
SH	1939.2	102.3	16.9	1032.8	3856.1	3449.3	0.0	0.0	5622.2
IL	1803.0	6290.6	4474.1	2814.4	133263.1	23007.2	2535.8	692.7	6453.8

Table 4.15 - ESALs and Designs of I-88 Segments for Actual and Idealized (Redistributed) Load Cases

Type	ADTT (lane)	Miles	ESAL (10^6) Actual	ESAL (10^6) Idealized	HMA or Slab (in) Actual	SGSG (in) Actual	HMA or Slab (in) Idealized	SGSG (in) Idealized
Flexible	2000	16.54	29.9	28.1	9.84	13.66	9.84	13.19
Flexible	2500	3.21	37.4	35.2	9.84	15.66	9.84	15.08
Rigid	1000	1.57	24.2	21.1	8.86	-	8.86	-
Rigid	1500	17.34	36.2	31.7	8.86	-	8.86	-
Rigid	2000	32.25	48.3	42.3	8.86	-	8.86	-
Rigid	2500	36.69	60.4	52.8	8.86	-	8.86	-
Rigid	3000	0.7	72.5	63.4	9.07	-	8.86	-
Rigid	3500	4.25	84.5	73.9	9.41	-	9.11	-
Rigid	4500	4.1	108.7	95.1	9.97	-	9.70	-
Rigid	5000	3.03	120.8	105.6	10.16	-	9.93	-

4.9.2 Pavement Damage Analysis and Maintenance Schedule

Each pavement section of the I-88 Corridor was designed for the two loading scenarios: a) actual WIM histogram and b) idealized with redistributed loads to obtain a no-overweight scenario, using the approach described in Section 4.9.1. The MEPDG approach was used to estimate the number of corrective maintenance cycles necessary to keep the IRI below 67. The analysis of flexible pavement segments was found to be sensitive to the effects of temperature for cracking but especially for rutting that is most severe in the summer months. The average monthly temperatures used are listed in Table 4.16. Also, it is noted that the rutting process is highly dependent on the load history. In the analysis, it is assumed that the pavement was opened to traffic on January 1st to establish a base starting point.

Table 4.16 – Average Monthly Temperatures

Month	Average temperature
January	22
February	25
March	33
April	49
May	56
June	65
July	69.5
August	68
September	60.5
October	49
November	39
December	26

The number of interventions necessary to maintain a pavement in very good condition varies between 2 to 9 interventions in the 50-year design life of the highway when the actual loads are applied. As shown in Table 4.17, the number of cycles depends on the type of pavement (rigid or flexible) and the ADTT in the main design lane. For the same ADTT, the rigid pavement shows a higher number of maintenance interventions than the flexible pavements. This is because rigid pavements are more sensitive to the effects of very high loads than flexible pavements while flexible pavements are more sensitive to the number of load cycles. When the overweight cargo is redistributed to no-overweight trucks, the number of maintenance interventions reduces down to 1 to 3 interventions over the 50-year design life. It is observed that for flexible pavements, the number of maintenance interventions is essentially the same as before redistributing the cargo. This is because by redistributing the loads, we increase the number of trucks on the highway system which compensates for the reduction of the heavy trucks by the increased number of loading cycles. On the other hand, the number of maintenance interventions in rigid pavements is vastly reduced. This is because by redistributing the cargo to not-overweight trucks the extreme loads are removed from the traffic stream eliminating their effects on rigid pavements. The associated increase in the number of truck cycles is not a very critical factor in rigid pavements. Finally, the number of maintenance interventions for rigid and flexible pavements is essentially the same if no overweight trucks are allowed.

The results listed in Table 4.17 are also presented in Figure 4.17 where the differences between the required numbers of corrective maintenance interventions are compared for the rigid pavement segments of the I-88 Corridor (on the left) and the flexible pavements segments on the right.

Table 4.17 – Results of MEPDG Analysis of I-88 Corridor

Type	ADTT (lane)	Miles	HMA or Slab (in) Actual	SGSG (in) Actual	HMA or Slab (in) Idealized	SGSG (in) Idealized	No. of Maint. in 50 yrs Actual	No. of Maint. in 50 yrs Idealized
Flexible	2000	16.54	9.84	13.66	9.84	13.19	2	2
Flexible	2500	3.21	9.84	15.66	9.84	15.08	3	3
Rigid	1000	1.57	8.86	-	8.86	-	4	1
Rigid	1500	17.34	8.86	-	8.86	-	5	2
Rigid	2000	32.25	8.86	-	8.86	-	5	2
Rigid	2500	36.69	8.86	-	8.86	-	5	2
Rigid	3000	0.7	9.07	-	8.86	-	6	2
Rigid	3500	4.25	9.41	-	9.11	-	7	2
Rigid	4500	4.1	9.97	-	9.70	-	8	3
Rigid	5000	3.03	10.16	-	9.93	-	9	3

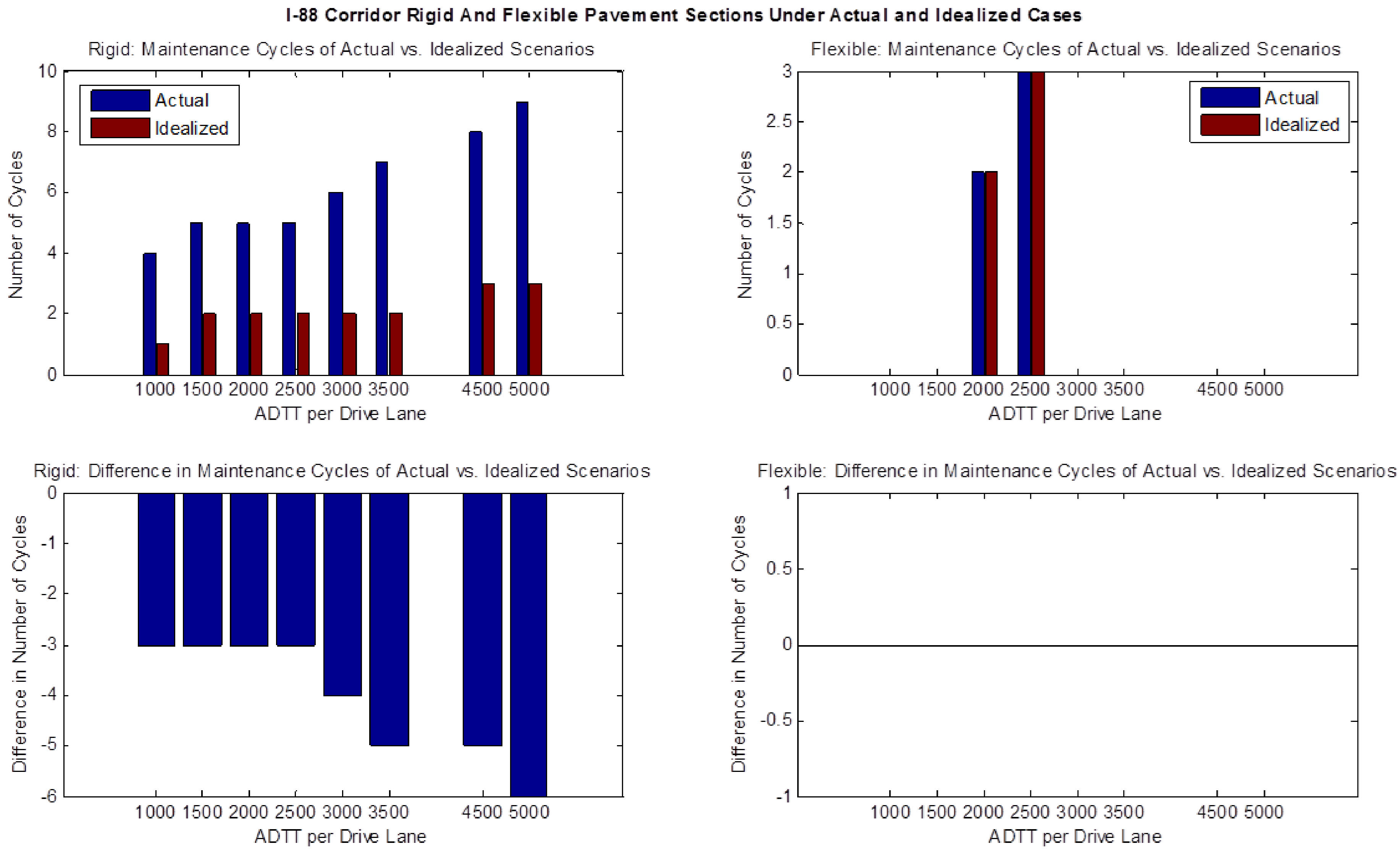


Figure 4.17 - Number of Maintenance Interventions for I-88 Corridor

In flexible pavements, the most important factors are rutting and fatigue fracture. Their contributions to the IRI over the 50-yr (600-month) pavement design life are depicted in Figure 4.18 for the actual truck WIM data. The results of the MEPDG analysis shows that corrective maintenance coincides with HMA rut depths of about 0.25 in and accumulated fatigue Damage Index DI=0.50. These values have been recommended as upper limits for these two damage phenomena in the pavement engineering literature (NCHRP, 2004). A shown in Figure 4.18, the contributions of rutting and fatigue to the total limiting IRI are respectively 10 and 12.5. The total accumulated IRI over the service months of the flexible pavement is depicted in Figure 4.19 which clearly retraces the theoretically expected pattern previously depicted in Figure 4.14 in terms of PSI. The figure illustrates how the time between interventions decreases over the years (months) as the service life of the pavement approaches the design life. This narrowing of the time between interventions is primarily due to the assumed growth in truck traffic over time.

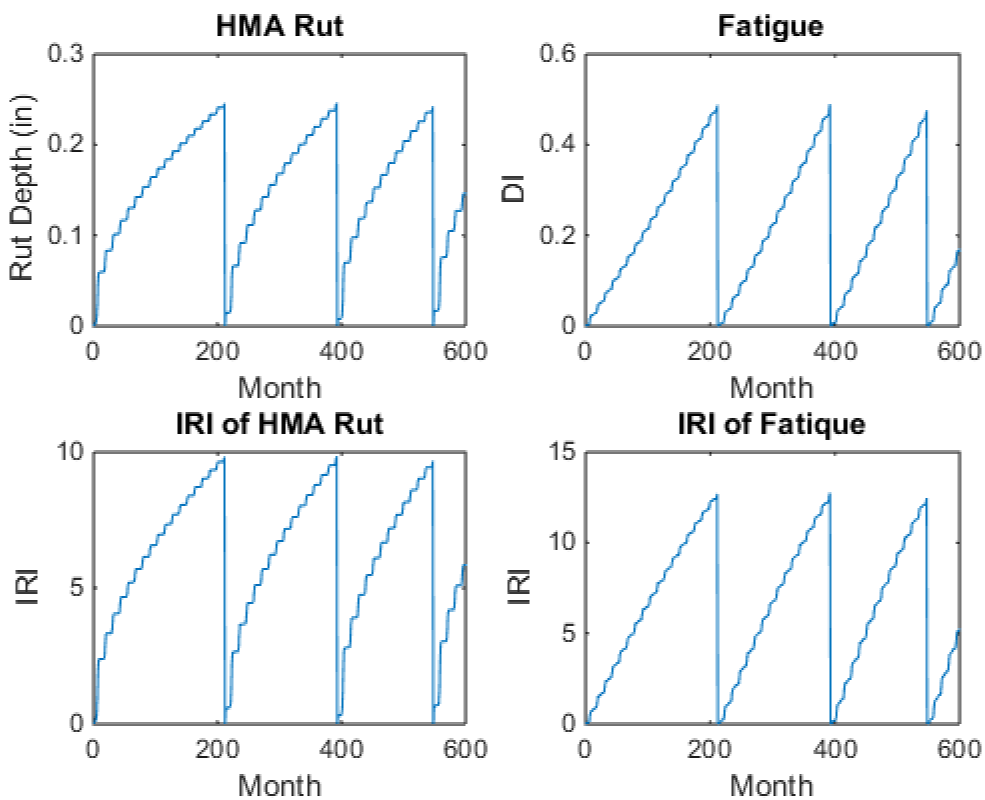


Figure 4.18 – Contributions of Rutting and Fatigue of Flexible Pavements to Overall Deterioration

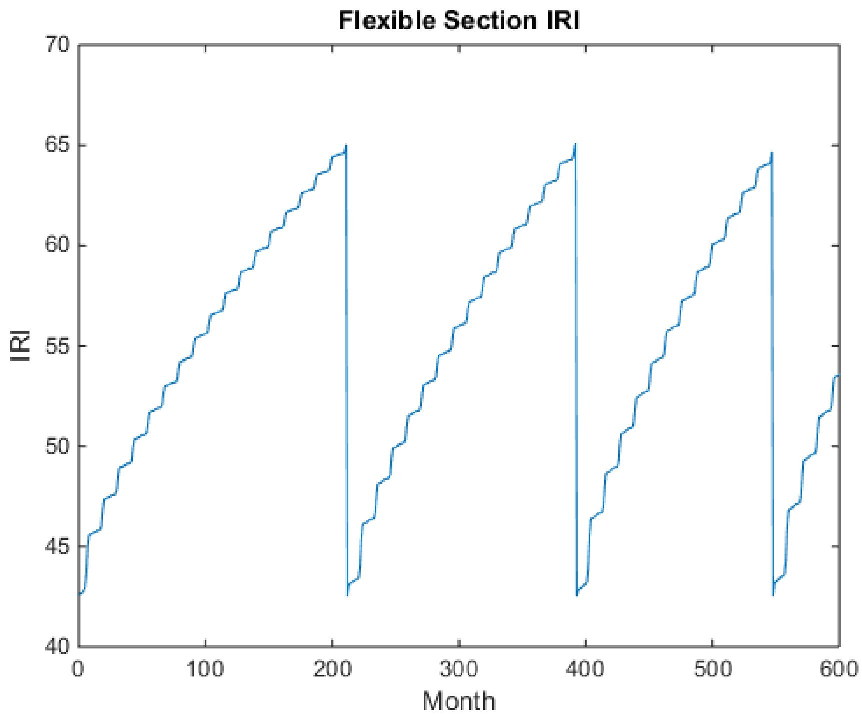


Figure 4.19 – Total IRI for Flexible Pavements on I-88 Corridor

For rigid pavements, the most important factors are faulting and fatigue fracture. Their contributions to the IRI over the 50-yr (600-month) pavement design life are depicted in Figure 4.20 for the idealized redistributed truck WIM data for a pavement section having a ADTT=1500 per main design lane. The results of the MEPDG analysis show that corrective maintenance coincides with faulting deflections on the order of 0.30 in and accumulated fatigue Damage Index DI=0.15. While the fatigue damage is relatively on the low side, the faulting value of 0.30 in has been recommended as an upper limit for this damage type in the pavement engineering literature (NCHRP, 2004). A shown in Figure 4.20, the contribution of faulting to the total limiting IRI is on the order of 45. The total accumulated IRI over the service months of this rigid pavement segment is depicted in Figure 4.21 which clearly retraces the theoretically expected pattern depicted in Figure 4.14 in terms of PSI. The figure illustrates how the time between interventions decreases as the service life of the pavement approaches the design life. This narrowing of the time between interventions is primarily due to the assumed growth in truck traffic over time.

It is also interesting to observe that the curves in Figure 4.19 for flexible pavement concave downward due to the consolidating nature of the rutting problem where rutting damage slows down over time, while the curves in Figure 4.20 for rigid pavement concave upward showing the acceleration of damage with time due to the faulting phenomenon.

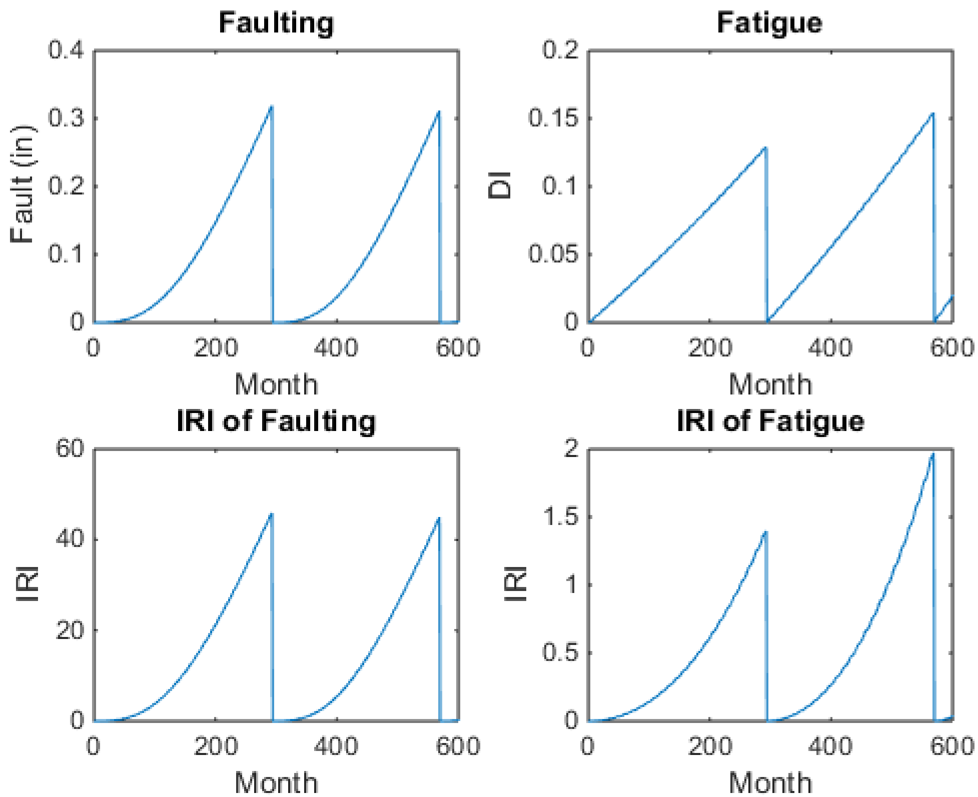


Figure 4.20 – Contributions of Faulting and Fatigue of a Typical Rigid Pavement Segment to Overall Deterioration Assuming Idealized Truck Spectrum on I-88 Corridor

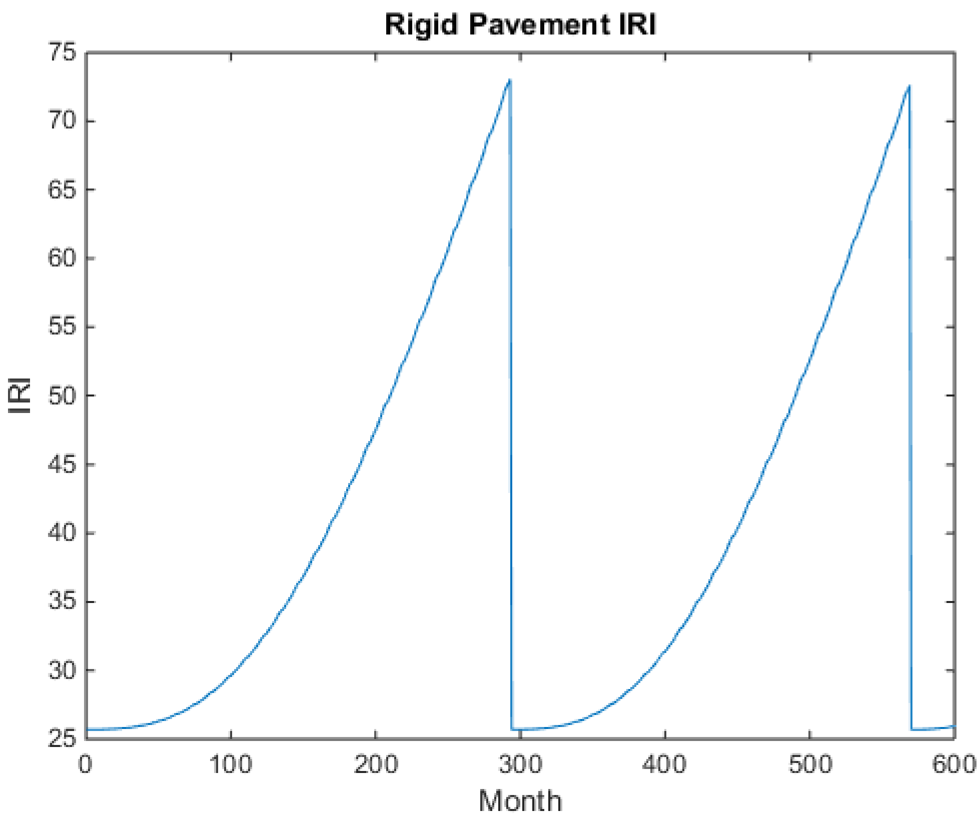


Figure 4.21 – Total IRI for Typical Rigid Pavement Segment under Idealized Truck Spectrum on I-88 Corridor

4.9.3 Pavement Cost

The differences in the costs for designing and maintaining the flexible and rigid pavements of the I-88 Corridor are presented in Figure 4.22 as a function of the ADTT in the main design lane. The costs in Figure 4.22 are in dollars per main design lane mile.

The analysis assumes that the costs of deployment and manpower for design do not vary significantly with changes in pavement section design. Therefore, the difference in the construction cost between the two scenarios (designs under actual truck weights versus designs required after redistribution of the overweight to not-overweight trucks) is only due to the difference in the costs of the materials. During the analysis, the cost of concrete (C) is taken as \$300 per ton. The cost for Hot Mix Asphalt (HMA) is \$150 per ton. The Subgrade cost is \$33.42 per cubic yard.

To account for the difference in the maintenance costs, the unit cost for each flexible or rigid pavement maintenance intervention is assumed to be \$100,000 per drive lane mile.

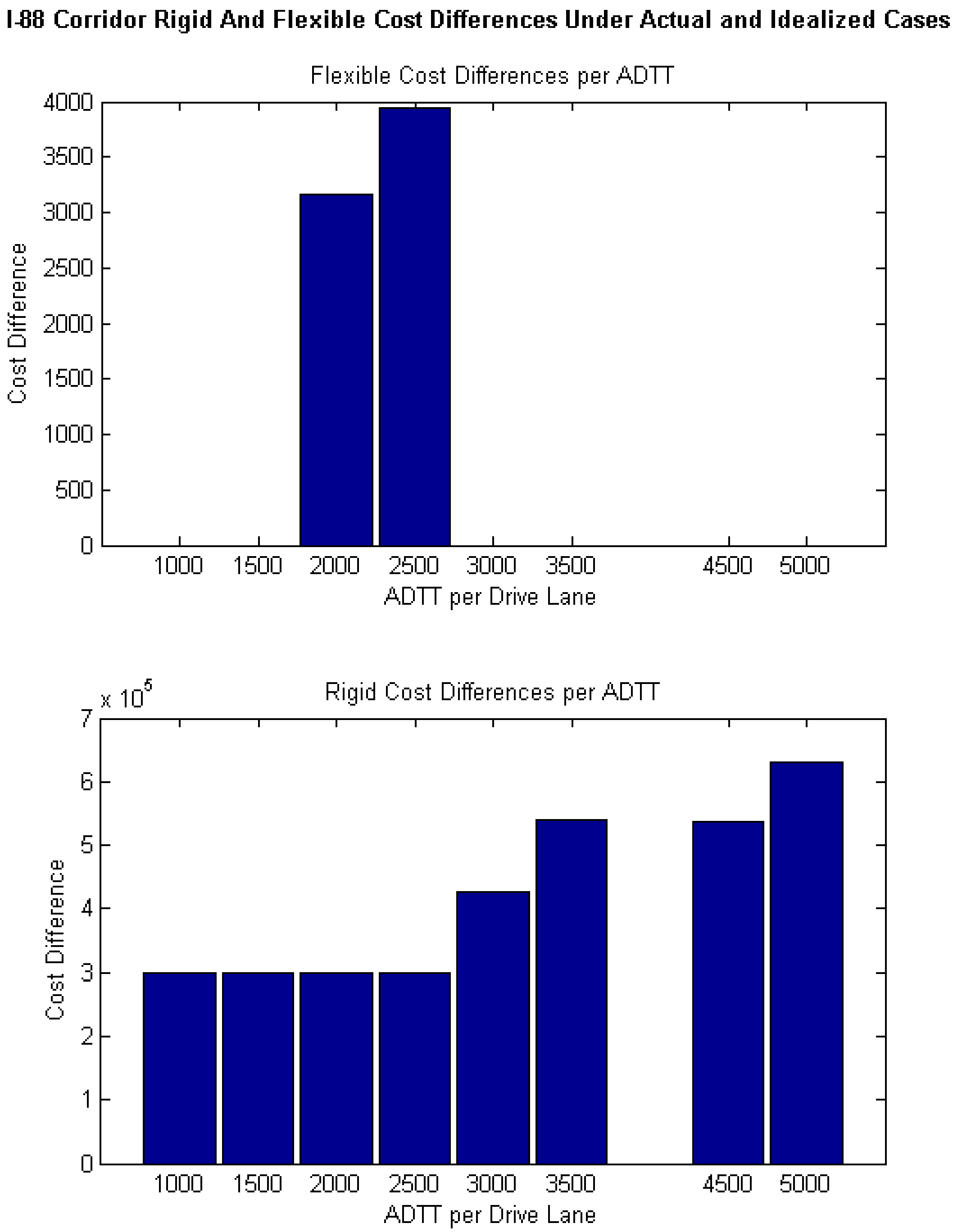


Figure 4.22 – Cost of Overweight Trucks on I-88 Rigid Pavement and Flexible Segments
(in dollars per mile per design lane ADTT)

The total cost of the I-88 Corridor due to overweight trucks is calculated based on the material costs and the additional numbers of corrective maintenance interventions. The allocation of the cost to Divisible (DV) permit trucks, Special Hauling (SH) permit trucks and Illegally (IL) overweight trucks is executed based on the percentage of the additional overweight ESALs in each class/weight category. Table 4.18 gives the percentage additional ESALs in each class/weight category for the flexible I-88 pavements and Table 4.19 gives those percentages for the rigid pavements. By additional ESALs, it is meant the ESALs above those of the most severe legal truck in each Class category as shown in Table 4.11.

The costs for each class/weight category are presented in Table 4.20 which shows that the total cost for the I-88 Corridor is 2,827,779 $/year. This total cost is divided into 1,589,583 $/yr for Divisible (DV) permits, 136,712$/yr for Special Hauling (SH) permit trucks, and 1,101,485$/yr for Illegally (IL) overweight trucks.

These results are specific to the pavement of the I-88 Corridor based on overweight truck distributions particular to the WIM data of Site 9580. The analysis must be repeated for any changes in the WIM data or pavement sections as the results are not scalable due to the nonlinearity of the damage accumulation process.

Table 4.18 – Percentages of Total ESALs per Class/Weight Category for I-88 Flexible Pavements

Flexible Allocation	5	6	7	8	9	10	11	12	13	Total
DV	0.00%	1.25%	1.96%	0.01%	37.80%	1.25%	0.00%	0.00%	2.15%	44.42%
SH	0.77%	0.04%	0.01%	0.01%	0.83%	1.27%	0.00%	0.00%	1.56%	4.49%
IL	0.57%	1.37%	0.35%	1.66%	37.50%	5.35%	0.95%	0.27%	3.09%	51.10%
Total	1.33%	2.66%	2.32%	1.67%	76.14%	7.87%	0.95%	0.27%	6.80%	100.00%

Table 4.19 – Percentages of Total ESALs per Class/Weight Category for I-88 Rigid Pavements

Rigid Allocation	5	6	7	8	9	10	11	12	13	Total
DV	0.00%	0.68%	4.59%	5.35E-05	35.84%	12.94%	0.00%	0.00%	2.18%	56.24%
SH	0.22%	0.03%	7.11E-05	4.98E-05	0.55%	1.92%	0.00%	0.00%	2.12%	4.84%
IL	0.17%	0.91%	0.85%	0.59%	25.22%	8.63%	0.23%	0.09%	2.24%	38.93%
Total	0.39%	1.62%	5.45%	0.60%	61.61%	23.48%	0.23%	0.09%	6.54%	100.00%

Table 4.20 – Total Cost per Year due to Overweight Trucks on I-88 Corridor Allocated to Each Class/Weight Category

Flexible Allocation	5	6	7	8	9	10	11	12	13	Total
DV		19,294	129,744	151	1,013,569	365,164			61,660	1,589,583
SH	6,125	719	201	141	15,496	54,236			59,794	136,712
IL	4,918	25,842	4,101	16,766	713,831	243,758	6,411	2,470	63,389	1,101,485
Total	11,043	45,856	154,045	17,058	1,742,896	663,158	6,411	2,470	184,843	2,827,779

4.10 Implementation to NYSDOT Highway Network

The same procedure is implemented on the entire highway pavement network in the NYSDOT database. This network excludes the Thruways, parkways and New York City. The entire NYSDOT network consists of 16,354 center miles. The temperature profile used in this analysis is shown in Table 4.16 which is assumed to be representative of the average temperature profile throughout the state. Also, the WIM data used in this analysis are the data taken from Weigh-In-Motion (WIM) station 9580 near Schenectady which are assumed to be representative of the weights and configurations of the truck population throughout the state. The analysis was performed to cover all possible variations of pavement types and ADTT's to cover all the pavement segments of the highway network as provided in the New York State DOT database after removing the segments pertaining to Thruways, parkways and New York City. The analysis is divided into upstate and downstate regions to reflect the different percentages of overweight trucks including permit trucks in those two regions.

Figure 4.23 shows how the number of maintenance schedules would increase when current trucks are traveling over the rigid pavements of the upstate highway network as compared to the case when all overweight cargo is redistributed to not-overweight trucks as the ADTT of the main design lane increases. As observed from the I-88 results, the change in the number of maintenance interventions is due to the sensitivity of the rigid pavement to heavy overweight trucks. On the other hand, the maintenance schedule for flexible pavements remains essentially the same when the overweight cargo is redistributed to not-overweight trucks. This is because for flexible pavements the increase in traffic volume, when the total cargo is shifted to not-overweight trucks, compensates for the reduction in the truck weights. The same phenomena are observed for downstate highways as depicted in Figure 4.24.

Table 4.21 gives the cost allocation due to the effect of overweight trucks on the entire NYS pavement network given the above listed assumptions. The results show that the total cost is 144,622,963$/year of which 22,475,403$/year is allocated to divisible permit trucks, the amount allocated to special hauling trucks is 48,558,730$/year and illegally overweight trucks are responsible for 73,588,830$/year.

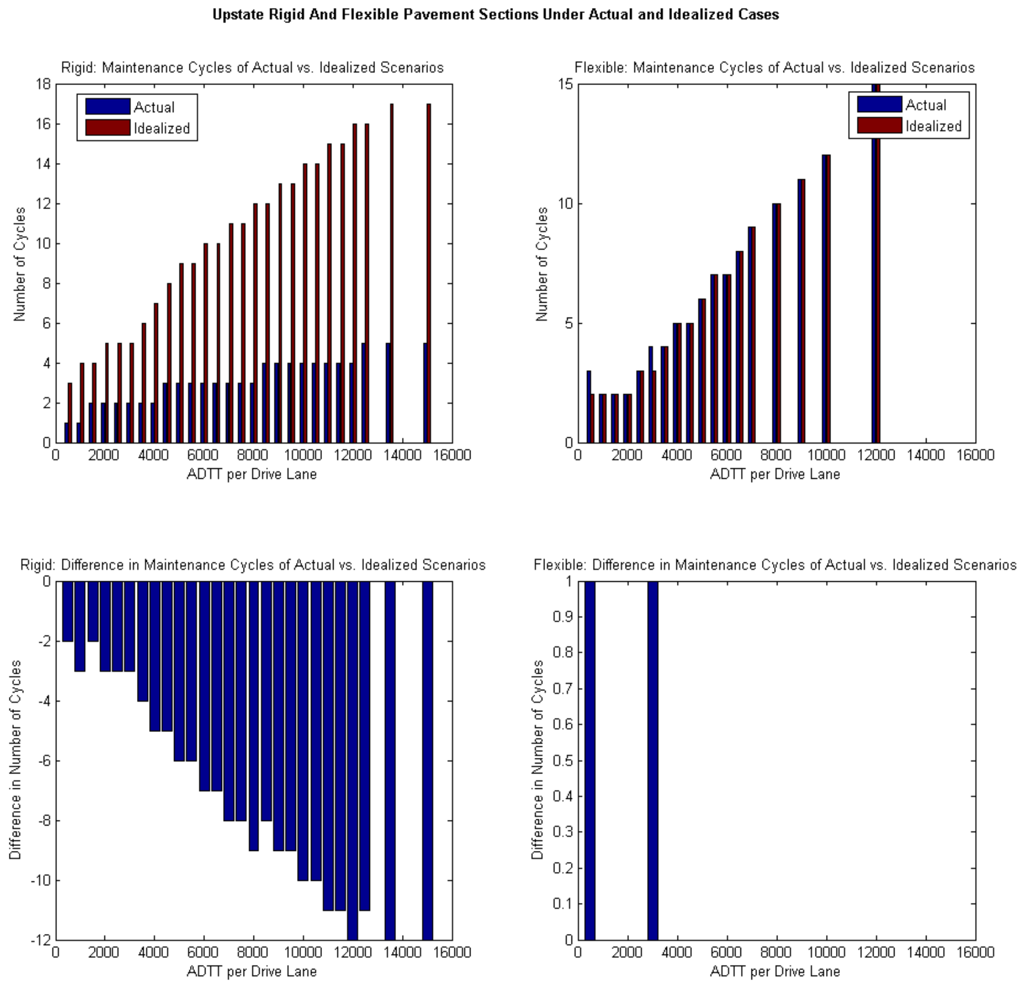


Figure 4.23 - Cost of Overweight Trucks on Upstate Rigid and Flexible Pavements
(in dollars per mile per design lane ADTT)

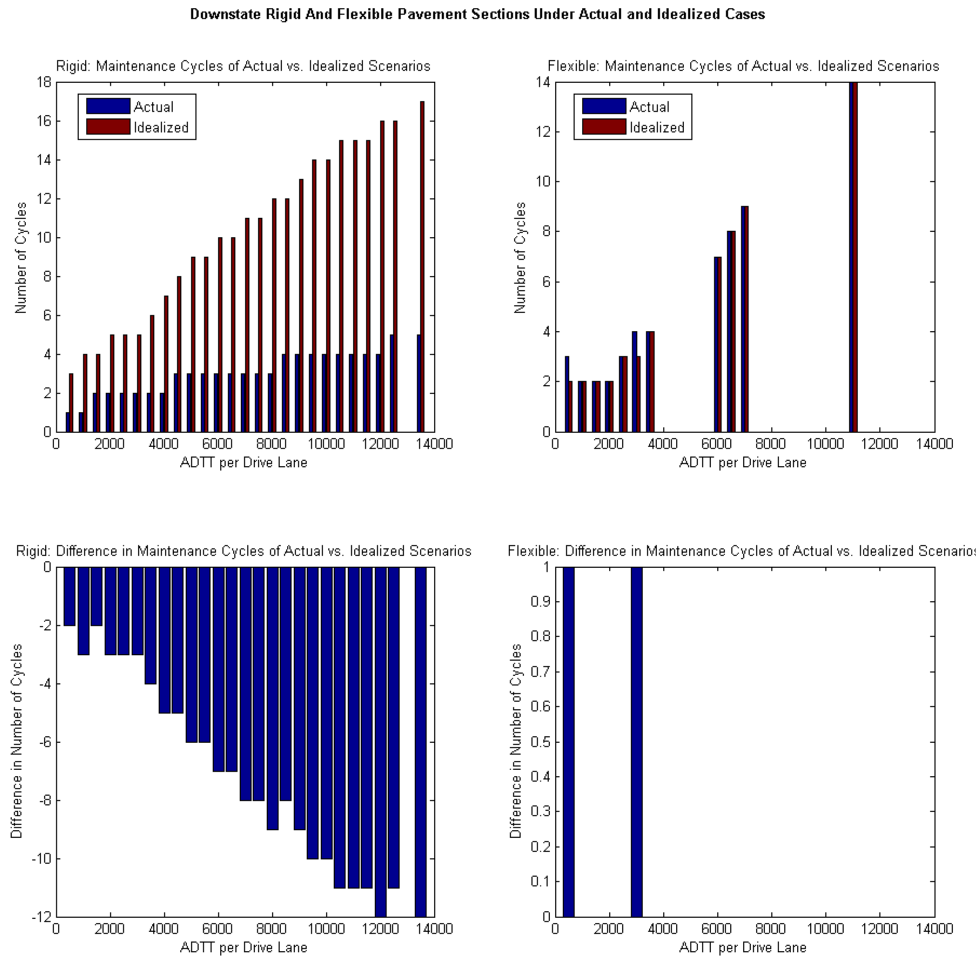


Figure 4.24 - Cost of Overweight Trucks on Rigid and Flexible Pavements
(in dollars per mile per design lane ADTT)

Table 4.21 - Cost Allocation for Entire NYS Pavement Network

UP/R	5	6	7	8	9	10	11	12	13	Total
DV	-	1,434,637	6,156,777	16,810	46,473,474	21,110,192	-	-	2,955,958	78,147,849
SH	423,220	48,218	32,145	8,156	572,894	2,027,918	-	-	3,508,617	6,621,169
IL	436,046	2,451,172	1,785,678	967,457	33,220,640	15,712,626	157,909	66,502	5,055,915	59,853,945
Total	859,266	3,934,027	7,974,601	992,424	80,267,008	38,850,736	157,909	66,502	11,520,491	144,622,963

4.11 Conclusions

This Chapter describes a method for evaluating the cost effects of overweight trucks on New York state DOT highway pavements. The method assumes that the cost of overweight trucks are due to an increase in the design thickness of the pavement layers and in a possible increase in maintenance schedules that would be required to keep a pavement system in very good condition. The approach compares the designs and maintenance schedules required under current traffic conditions to those that would be required if the cargo being carried by overweight trucks is redistributed to not-overweight trucks.

The design and maintenance cost analysis assumes that the design of pavement is based on the traditional ESAL method that is being currently used by NYSDOT. The deterioration model however, is based on the AASHTO MEPDG method.

The methodology is first implemented on the I-88 Corridor between Binghamton and Schenectady which shows that the cost due to overweight trucks for that corridor is on the order of $3 million dollars/year.

The implementation of the approach to the entire NYSDOT pavement network shows that the cost due to overweight trucks for the entire network is on the order of 145 Million $/year, with about $78M/yr allocated to divisible permit trucks, about $7M/yr allocated to Special Hauling permits and the remaining $60M/yr to illegally overweight trucks.

The values obtained in this report for the costs of pavement due to overweight trucks are on the same order of magnitude but lower than the values found by other researchers for other states. For example, the Indiana DOT study (Sinha et al, 2005) estimated an annual cost for pavement in the range of $354-397 million. The Arizona DOT study (Strauss and Semmens, 2006) estimated the cost of damage to their pavements from heavy vehicles to be around $210 million per year.

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Chapter 5

Summary of Findings

This Report developed models for assessing the effect and associated costs of overweight vehicles to New York State's highway pavements and bridges. The models are applied for assessing the cost of overweight trucks that have been issued permits by the New York State Department of Transportation (NYSDOT) to carry both divisible and non-divisible loads, as well as illegally overweight non-permit trucks that violate the legal limits and trucks that violate their permit limits.

In a first phase, this study developed a data mining procedure to estimate the numbers, types and legal status of heavy vehicles traveling over the New York State (NYS) highway network. The analysis of the data shows that the numbers of overweight trucks that exceed any of the NYSDOT criteria vary depending on location and vehicle types and classes. On the average, it is estimated that about 18% of the total truck population is overweight. A further breakdown into categories shows that about 11% of the trucks travelling on New York highways may be carrying divisible load permits, 1% may be carrying special hauling permits, while about 6% may be illegally overweight.

The analysis of the effect of overweight trucks on bridges is implemented on a representative sample of New York State bridges. The bridges analyzed include concrete slab bridges, concrete T-beam bridges, simple span and continuous steel girder bridges, prestressed concrete I-girder bridges, and prestressed adjacent multi-box girder bridges. The analysis excluded special bridge types, such as culverts, trusses, arches, movable, suspension bridges and long span bridges which are dominated by other factors than damage from individual overweight trucks. The selected bridge sample is considered to be representative of the effects of overweight trucks on 93% of the bridges in the New York State DOT database that are prone to damage by individual overweight trucks.

The allocation of the additional cost incurred by the NYS bridge network due to overweight trucks is estimated to be on the order of $95M per year, $50M of which are allocated to trucks issued divisible load permits, $2M are due to trucks with special hauling permits, and $43M per year are the additional costs due to illegally overweight trucks. The costs allocated to illegal trucks are considerably disproportional to their numbers because permit issuance policies ensure that the excessive weights are distributed on trucks having configurations that minimize the load effect on bridges and pavements. The bridge analysis considered cyclic fatigue damage to bridge members and decks as well as the increased risk caused by overstressing bridge members.

The MEPDG method is used to supplement the ESAL method to estimate the pavement cost of overweight trucks. The cost allocation study performed on the entire NYS pavement network consisting of over 16,000 miles shows that the overall cost to NYS pavements due to overweight trucks is on the order of \$145M/yr divided into \$78M/yr for divisible permit trucks, \$7M/y for special hauling trucks and \$60M/yr for illegally overweight trucks.

Overall, the effects of the overweight trucks on NYSDOT bridges and pavements add up to about \$240M per year. The cost estimates obtained in this Report are on the same order of magnitude found by other researchers for other states.

University Transportation Research Center - Region 2
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Region 2 - University Transportation
Research Center
The City College of New York
Marshak Hall, Suite 910
160 Convent Avenue
New York, NY 10031
Tel: (212) 650-8050
Fax: (212) 650-8374
Website: www.utrc2.org