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Monitoring and Control of Overweight Trucks for Smart Mobility and Safety of Freight Operations

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Monitoring and Control of Overweight Trucks for Smart Mobility and Safety of Freight Operations

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Disclaimer

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Executive Summary

Infrastructure systems constitute a major part of the national investment and are critical for the mobility of our society and its economic growth and prosperity. The United States has an estimated $25 trillion investment in civil infrastructure systems, including all installations that transport, transmit, and distribute people, goods, energy, resources, services, and information. Road pavements, bridges, and other infrastructure systems are considered assets that require maintenance and proper management to guarantee their functionality to the U.S. economy. Yet, the degree of deterioration due to the exposure to natural and manmade hazards such as overweight trucks is relatively high. In New York City (NYC), major decisions must be made to allocate limited funding for repair, maintenance, and rehabilitation of the infrastructure network. These decisions should be based on integrating various sources of information to control the infrastructure systems and their deterioration using structural models coupled to traffic modeling at the network level, to help perform economic forecasting and life-cycle cost analysis.

Considerable research has been done on the impact of the overweight vehicles on the infrastructure. While many researchers investigated the impact of overweight vehicles on pavement and bridges separately, not many studies employ an integrated approach for capturing combined effect of overweight vehicles on the infrastructure. Nassif et al. (2015) developed a model for analyzing the impact of overweight freights on the New Jersey (NJ) infrastructure. As part of this comprehensive approach, they developed data-driven pavement and bridge deterioration models using the data collected from the statewide weigh-in-motion (WIM) stations over a period of more than a decade. Then, the economic analysis involved the estimation of the life-cycle costs for the NJ infrastructure based on the predicted maintenance and replacement cycles from the deterioration models. Finally, the results were embedded in an app called ASSISTME-WIM for visualizing the results and predicting the future damage of overweight vehicles given their weight, axle configuration, and route.

New York (NY) and NJ are two contiguous states that share many interstate highways and have similar traffic composition due to their proximity. Therefore, their infrastructures are likely exposed to similar deterioration. Due to many similarities between these two regions, one might ask if a similar approach can be adopted for NYC. While there are many similarities, differences such as the percentage of trucks on the roadway segments, overweight loading condition, pavement structure, current condition of the pavement, and construction and maintenance costs should be identified so that the deterioration models and damage costs can be recalibrated for NYC. Recalibration of model parameters requires either detailed aforementioned data or sound assumptions if detailed data are not readily available.

This project aims to monitor the impact of overweight trucks on the bridges and pavement infrastructure under the jurisdiction of NYC Department of Transportation (NYCDOT). Unlike NJ, where large amount of truck traffic data are available from statewide WIM stations, data from only three

existing permanent fixed WIM stations were available at the time of this study: Alexander Hamilton Bridge (AHB), Van Dam Street (VDM), and Rockaway Boulevard (RKW). Moreover, the current condition and construction costs for pavement projects were not readily available, which required assumptions to be made for further analyses. Because of the limited data from NYCDOT, some reasonable assumptions were made. Pavement structure of NYC is similar to NJ, pavement deterioration and intervention of NYC is similar to NJ, and pavement rehabilitation/maintenance cost of NYC is higher than NJ. The deterioration of bridge superstructures is similar in both NYC and NJ. The preliminary economic impact of overweight vehicles on bridges was quantified as dollar per overweight-ton per deck area per trip for three case studies (AHB, VDM, and RKW). The research team found that the unit damage cost is constantly higher for NYC than for most NJ highways. The unit damage cost of overweight trucks on bridges near RKW for reinforced concrete bridge decks, steel multibeam girders, and steel girder-floorbeam girders is 146%, 327%, and 361% of the maximum damage cost found in NJ, respectively. Similarly, the impact on pavements due to overweight vehicles depends on the total number of vehicles and total mileage traveled by overweight vehicles per year. The results from the preliminary analysis on selected corridors in NYC estimate the impact on pavements in the range of $0.0345 and $0.0698 per equivalent single axle load (ESAL)-lane-mile on an interstate highway near AHB and between $0.117 and $0.648 per ESAL-lane-mile for local roads (VDM and RKW), which are approximately 27.6% to 34.2% higher than NJ. However, the estimates obtained in this study give only a preliminary estimate since the available data is currently limited. Hence, to better understand the effects of overweight trucks on NYC infrastructure, it is necessary to obtain bridge and pavement inspection report data for damage model, and truck traffic data in many more key locations in the city. For future study, the team will (i) investigate whether the local transportation agency has short truck counts in other locations, (ii) obtain the database behind the geographic information system map of NY-wide annual average daily traffic on the links (and later fuse this data with the truck data from the closest WIM station), and (iii) install WIM systems to monitor the truck traffic at key locations.

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Section 1 – Introduction

Transportation agencies are responsible for the expenditure of taxpayer dollars to preserve and maintain the highway infrastructure system in a state of good repair according to current standards. Understanding the effect of truck loads on the infrastructure network is important for upgrading and maintaining a state of good repair. Overweight trucks cause significant deterioration to pavements and bridges, which results in frequent maintenance and higher rehabilitation costs. Currently, each state tries to control the number of overweight trucks by issuing permits and imposing fees based on each state's regulatory policy. For example, in New Jersey (NJ), oversize vehicles, overweight vehicles, Code 23 registered trailers, and Annual Ocean Borne Container permits have been regulated by imposing an excess weight fee. The current fee structure is not intended to recover the damage cost induced by overweight trucks but to encourage truck owners to get permits so that the state can manage these vehicles (Nassif et al., 2015).

Regulating overweight truck operations and oversize trucks is necessary to ensure the safety of infrastructure and to minimize damage to pavements and bridges while promoting commerce and the movement of goods and services. The American Association of State Highway and Transportation Officials (AASHTO) conducted early studies on the impact of heavy truck loads on road pavements in the form of Road Tests in the 1950s. Test data have shown that the damage on pavements can be as large as to the fourth power of the loads.

The Connected Cities for Smart Mobility towards Accessible and Resilient Transportation (C2SMART) project (Monitoring and Control of Overweight Trucks for Smart Mobility and Safety of Freight Operations) conducted a study in order to develop a web-based geographic information system (GIS) application tool to evaluate the damage cost associated with overweight vehicles, to collect and process data that are essential for monitoring special hauling trucks and their movements on side routes in NJ and New York City (NYC), and to develop a proof of concept for an autonomous ticketing system for overweight trucks. A web-based GIS application was developed using 4 years of recent permit data (2013 to 2016) to estimate the cost of pavement damage of the NJ roadway system and to evaluate the pavement and bridge damage costs associated with overweight trucks. The tool will help monitor the movement of permit trucks and determine hotspots frequently crossed by permit vehicles. This tool will also help New Jersey Department of Transportation (NJDOT) engineers better plan maintenance and repair operations.

Weigh-in-motion (WIM) data from the entire set of WIM stations in NJ and three WIM stations in NYC were collected and processed to calculate bridge deterioration due to overweight trucks. The collection of truck weight data and axle configurations provides the basis to evaluate the impact of overweight trucks on the road infrastructure in NJ and NYC. Operating large trucks on side roads can adversely affect the performance of infrastructure and would disrupt traffic patterns.

Budget implications for maintaining safe roads and their integrity are obvious. Major decisions must be made to allocate the limited funds available for repair, rehabilitation, or replacement of the infrastructure system. Accordingly, transportation agencies spend significant resources to maintain and repair affected roadways and bridges. Factors that cause damage to roads, pavements, and bridges include overloading, fatigue, aggressive environment, and seasonal variation; however, this knowledge needs to be used to create reliable tools to provide recommendations for improved management of the infrastructure system. This report synthesizes the research effort for developing a methodology to estimate the actual infrastructure damage cost associated with overweight trucks in the NYC metropolitan area.

Subsection 1.1 Objectives

The main objectives of this project are to (1) develop the methodology to estimate the infrastructure damage cost associated with overweight vehicles on the road infrastructure in NYC, especially for pavements and bridges; (2) develop a web-based application to calculate the network-based damage cost to assess their impact; and (3) search the proof of concept of the WIM system for enforcement. The long-term objective of this project is to develop an integrated WIM system for screening trucks for autonomous enforcement of overweight trucks. Additionally, this research will develop a decision-support tool for the NYC Department of Transportation (NYCDOT) to collect data to foster future maintenance and preservation operations.

Section 2 - Literature Review

The most relevant literature related to impact of overweight vehicles on pavements and bridges and the mechanism of deterioration have been reviewed based on current practice, technical literature, and research findings from domestic and foreign sources. Moreover, a complete literature search of all related research work done by Federal Highway Administration (FHWA), National Cooperative Highway Research Program (NCHRP), Strategic Highway Research Program (SHRP), and other departments of transportation on the impact of heavy trucks on highway infrastructure was completed. Heavy trucks have significant effect on the highway bridges and pavements in terms of load carrying capacity, serviceability, and structure maintenance. In addition, the available WIM sensors in the markets have been reviewed to evaluate their performance, pros, and cons. The advanced and high-accuracy WIM systems were also reviewed, and some implementation case studies were reviewed for enforcement purposes.

Subsection 2.1 Effect of Overweight Trucks on Bridges and Pavements

Due to the increasing number of permits issued for overweight trucks, the impact of overweight trucks on highway infrastructure—mainly in bridges and pavements—is a major concern in North America.

New York State

New York State (NYS) analyzed the effect of overweight vehicles on the NYS bridge and pavement infrastructure network for bridges and pavements (Ghosn et al., 2015). The research team used overstress and fatigue to compute the response of bridges to overweight trucks and the mechanistic-empirical method to analyze pavements. The data indicate that about 11% of trucks traveling on NYS highways have divisible load permits, 1% have special hauling permits, and about 6% may be illegally overweight. The analysis also showed that these overweight trucks are increasing the risk to bridge failure by causing stresses above those specified in design specifications and by reducing bridge service (fatigue) lives through repetitive overloading. The safety margin utilization cost of the infrastructure for bridges per year was estimated at about \$50M, while about \$145M per year was the impact estimated for pavements. About 50% of these costs were attributed to illegal overweight trucks traveling on the NYS infrastructure network (Ghosn et al., 2015).

New Jersey State

NJDOT assessed the impact of overweight vehicles (both permitted and nonpermitted) on NJ’s infrastructures, specifically highway pavements and bridges. The analyses project the effect of deterioration due to overweight trucks over the life of the highway infrastructure using life-cycle cost analysis (LCCA). The estimated statewide average cost of moving 1 ton of overweight load 1 mile is about \$0.33, in which approximately 60% of the damage cost is attributed to pavement and 40% to bridges. Based on the current permit fee structure in NJ, it was estimated that the weight-based fee does not cover the damage cost for loads exceeding legal limit (Nassif et al., 2015).

Canada

One of the earliest studies on this subject was performed in Canada. This study focused on the fatigue effect of heavy-permit trucks on steel highway bridges. Both ultimate and cumulative effect of the overloads were investigated. It was found that the selected bridges had adequate ultimate capacity to accommodate the overweight vehicles. However, the cumulative fatigue damage was the concern for a large number of passing overweight trucks. Additionally, the author stated that the concept of infinite fatigue life cannot be applied to bridges due to the involvement of overweight trucks, and a reasonably large number of special permits could only cause a small reduction in fatigue life (Dicleli et al., 1995).

Connecticut State

A study on the behavior of selected steel bridges under specific superload permit trucks was performed in Connecticut. The effect of six specific superload trailer types on bridges were discussed based on bridge span length, lateral load distribution, and dynamic load allowance. Strain data from the testing were obtained and compared to the response from structural analyses. The results showed that a conventional line girder analysis can be used to analyze the effect of superload on highway bridges. Impact can be taken as zero for trucks crossing at walking speed (Culmo et al., 2004).

Indiana State

In 2006, a study on the fatigue of older bridges in Northern Indiana due to overweight and oversize loads was reported. Field measurements of truck axle load spectrum and bridge response were collected. Both two-dimensional and three-dimensional models were built to predict the structural response under identified truck loads from WIM data. New three-axle and four-axle fatigue trucks were developed using WIM data. Moreover, a statistical database of resistance parameters was built. Then fatigue evaluation in terms of remaining fatigue life for the selected bridge was obtained (Reisert et al., 2006).

Wisconsin State

The evaluation of bridge under overload vehicles was initiated in 2009 in Wisconsin and expanded in 2012. The first phase focused mainly on the structural analysis of bridges under overweight trucks. Finite element models of 118 multi-girder bridges were developed, and 16 load cases of overload vehicles for each multigirder bridge were performed. The girder distribution factor equations for multigirder bridges under overload vehicles were proposed afterward. Investigators determined that intermediate diaphragms under overload vehicles were not a concern. As an extension of the first phase, the authors aimed to use LCCA to evaluate the long-term cost impact of vehicles on bridges. Finally, researchers investigated the long-term behavior of concrete decks and steel-girder bridges and developed a means to assign cost to the overloads (Bae et al., 2009; Bae et al., 2012).

Many researchers have stated that bridge decks deteriorate as a result of different complex failure modes, such as corrosion, fatigue, and global or local flexural cracks. An investigation of the impact of overweight trucks on bridge deck deterioration was performed using laboratory tests and numerical simulations. Laboratory tests simulated the combined effect of mechanical stresses and freeze-thaw cycles on concrete cylinders. The results confirmed that the mechanical loading combined with freeze-thaw cycles significantly increased the permeability of air-entrained concrete and may accelerate the deterioration of concrete elements such as bridge decks. The numerical bridge deck simulation analyzed the stress level in both transverse and longitudinal direction. Additionally, empirical equations were proposed to predict the stress under heavy wheel load (Lin et al., 2012).

A review of current permitting practice from different states and their fee structures were studied in Wisconsin. The preliminary trends for overweight and oversize demand in the foreseeable future was also outlined. The research team also documented infrastructure impacts of oversize and overweight loads, including pavement, bridge, safety, congestion, and environment. Finally, a methodology was proposed to quantify the cost, but it was not validated with empirical data (Adams et al., 2013).

Subsection 2.2 Pavement Damage Cost Due to Overweight Trucks

In order to reasonably estimate the cost infrastructure damage by overweight trucks, different cost models for pavement damage have been developed considering the unique property of the infrastructure in each state. Several pivotal research attempts were introduced to determine the pavement damage cost (PDC) and to quantify the impact of overweight trucks on pavement damage.

New Jersey State

This study considered various aspects of pavement characteristics, including layer types, material types, and thickness of flexible and composite pavements by selecting representative roadways among networks of NJ. This study suggested using mechanistic-empirical analysis procedure to calculate the PDC. The recommended PDC ($/equivalent single axle load (ESAL)/lane/mile) in this study as approximately $0.027 to $0.052 for Interstate and U.S. highways, and approximately $0.092 to $0.0483 for state highways (Nassif et al., 2015).

California State

The study shows that heavy trucks with five or more axles have more impact on pavement maintenance than light trucks and passenger cars. The PDC in $/mile/year of heavy trucks in California from 1984 to

1987 was estimated to be $7.60, while the corresponding cost of passenger cars is approximately $3.7 (Gibby et al., 1990).

Southern Ontario

The type of the highway is crucial in calculating PDC. For example, the study proposed the PDC in $/ESAL/km/year for new pavement in southern Ontario could range from C$0.0025 for a freeway to C$0.597 for a local road (Hajek et al., 1998).

Indiana State

The marginal pavement cost for routine maintenance expenditures in Indiana was introduced, and PDC in $/ESAL/mile could range from $0.0143 to $0.024 (Li et al., 2002).

Subsection 2.3 Deterioration Model for Highway Bridges

Deterioration of Bridge Decks

The deck suffers more deterioration than any other bridge component because it is directly exposed to traffic loads, environmental conditions, and deicing salts. Therefore, modeling the deterioration of bridge decks is complex. Previous studies have demonstrated that transverse cracks and water penetration during service decrease both the ultimate punching shear and fatigue strength of concrete decks (Azad et al., 1986; Okada et al., 1978; Kato et al., 1978). However, the interaction between deck deterioration and overweight loading has not been quantified explicitly yet. The current AASHTO *Manual for Bridge Evaluation* indicates that deicing salts and high truck traffic volume affect the deck deterioration rate, but the evaluation of bridge decks is generally limited to a visual inspection only (AASHTO, 2017). In reality, a combination of mechanical loading and environmental factors lead to the end of the deck service life.

Previous studies indicate that both fatigue and overstressing are the two major problems caused by mechanical loading, and it was found that the fatigue and overstressing are two independent possible deterioration modes for decks (Batchelor et al., 1978; Fang et al., 1990; Petrou et al., 1994). Starting in the late 1970s, researchers performed laboratory tests to investigate the failure modes of reinforced concrete (RC) decks. The test results showed that RC deck fatigue is governed by the punching shear failure of concrete (Petrou et al., 1994; Perdikaris et al., 1993; Youn et al., 1998).

Experimental tests have also shown that both the intensity of axle loads and the characteristics of the boundary conditions are the two most important factors for a correct evaluation of the deterioration of decks (Okada et al., 1978; Kato et al., 1978; Fang et al., 1990; Petrou et al., 1994).

Highway bridges are vulnerable to damage from environmental attack such as corrosion, freeze-thaw, and alkali-silica reaction. RC deck corrosion leads to a reduction in the cross-sectional area of the reinforcing steel and a loss of bonding, which may further lead to a loss of strength, which will eventually cause the deck to be unserviceable. Various models for this type of failure have been developed based on the mechanism of chlorides diffusion through the protective concrete cover, showing that the corrosion will be initiated once the chloride concentration exceeds a specific threshold (Weyers et al., 1994; Liu et al., 1998, Stewart et al., 1998; Vu et al., 2000).

Deterioration of Bridge Girders

The increase in legal truck weight would shorten the time for repair or replacement of many bridges. Yoder et al. (1979) investigated the impact of a gross vehicle weight (GVW) limit increase for Indiana DOT, including those on bridges. The following cost impacts were included in this effort for bridges: (a) strength-related costs, (b) steel fatigue-related costs, and (c) deck deterioration costs. The strength-related costs refer to inadequate load carrying capacity of bridges under the new permissible load. The steel fatigue-related costs were also estimated to be negligible, based on the data available at the time. Impact costs associated with bridge deck deterioration were estimated using an assumption that cost increase is linearly related to the maximum permitted GVW. This study represents an early effort in this area.

In 1985, NCHRP Project 12-28(3), Fatigue Evaluation Procedures for Steel Bridges, was initiated (Moses et al., 1987). The goal of the principle investigators was to develop fatigue design procedures that more accurately reflect fatigue-loading conditions. Probabilistic techniques were employed to ensure consistent levels of reliability. There has been extensive field-testing of bridges to determine remaining fatigue life. For the most part, the investigators installed strain gages to key fatigue-prone detail locations on a bridge structure and monitored strain and stress levels for a given period of time. The cumulative damage was calculated using Miner’s rule and the average daily truck traffic (ADTT) for each location. The remaining fatigue life is the total life less the current service life of the structure.

NCHRP Report 495 proposed a recommended methodology for estimating the impact of changes in truck weight limits on bridge network costs (Fu et al., 2003). Step-by-step instructions for applying the methodology were included in the report along with a detailed application of the methodology. Four cost-impact categories are covered in the methodology: (1) Fatigue of existing steel bridges, (2) fatigue of existing RC decks, (3) deficiency due to overstress for existing bridges, and (4) deficiency due to overstress for new bridges. The fatigue life evaluation is the core part of the procedure.

Nowak, Nassif, and DeFrain (1993) published the findings of a fatigue evaluation for a steel bridge. This bridge under investigation was instrumented to determine the remaining fatigue life. Strain gages were installed to monitor the fatigue of critical members. Additionally, all girders in one span were instrumented to determine the load distribution. This was crucial to understanding the actual versus assumed load distribution. Analytical results showed high stress concentration in the exterior girders, making these members most prone to fatigue. However, the measured stresses were much less than the calculated stresses. Sensors indicated that the connection of the floorbeams to the exterior girders was behaving like a fixed moment connection. Furthermore, the floorbeam was responding as a fixed beam against rotation but undergoing a relative displacement between the supports.

Subsection 2.4 LCCA of Bridges and Pavements

LCCA is used in transportation infrastructure management and decision-making processes. Many professional societies publish literature covering this topic, each with its own objectives. For example, the FHWA and local transportation agencies are interested in promoting LCCA as an evaluation tool for its ability to achieve higher policy objectives. Highway construction companies use LCCA to prove improved long-term benefits of their products (e.g., rigid pavements). The review in this subsection goes over the major literature provided by the different stakeholders, with special emphasis on academic research.

NCHRP Synthesis 122 summarizes the use of LCCA in highway agencies in 1985 (Peterson, 1985). The extent of LCCA application, its complexity, and comprehensiveness during that period was limited, mostly because of the difficulty in performing intensive computational analyses in the absence of high-performance computing systems available today. The well-known FHWA report called "Life-Cycle Cost Analysis in Pavement Design: In Search of Better Investment" became a major LCCA keystone and is one of the most referenced documents in LCCA literature (Walls et al., 1998). This report is important because it provides an easy-to-follow step-by-step process on how to conduct LCCA, including numerical examples. The most important contributions are the work-zone user cost calculations and the incorporation of reliability concepts in LCCA application based on the Monte Carlo simulation method. More recently, in 2003, the FHWA Office of Asset Management released the Probabilistic LCCA software package that performs the LCCA according to the report described above. The FHWA LCCA guidelines, however, have minimized the significance of user costs during normal operations in the LCCA. It assumes that user costs are comparable across competing alternatives when the pavement serviceability reaches a certain level; consequently, excluding them will not affect the LCCA outcome.

A publication by the American Concrete Pavement Association (ACPA) titled *Life Cycle Cost Analysis: A Guide for Comparing Alternate Pavement Designs* explains all factors that should be considered in the LCCA and provides guidance on the selection of LCCA-sensitive parameters. It includes useful real-life case studies with detailed numerical calculation that better illustrate the LCCA process. However, it clearly focuses on showing the benefits of lower life-cycle cost of rigid pavement (ACPA, 2002).

Hawk (2003) wrote NCHRP Report 483, *Bridge Life-Cycle Cost Analysis*, for professionals to perform LCCA for bridges. The first part of the report establishes guidelines and standardizes procedures for conducting LCCA. The second part of the report is useful to all professionals who use LCCA, either for the repair of existing structures or to evaluate new bridge alternatives. The manual outlines the concept of LCCA, identifies sources for data, and explains the methodology by which life-cycle costing can be conducted. The report also includes bridge life-cycle cost analysis (BLCCA) software that allows professionals to apply the LCCA concepts and methodologies to the analysis of bridges.

Daigle and Lounis (2006) developed an approach for LCCA of RC bridges that considers all costs incurred by owners and users during construction, maintenance, rehabilitation, and replacement. This approach also provides an estimate for the environmental impacts associated with construction and replacement of bridge decks in terms of greenhouse gas (GHG) emissions and waste production. The analysis considers all the key stages in the life cycle, including extraction of raw materials, construction, maintenance, repair and rehabilitation, replacement, and disposal. The total life-cycle costs are evaluated by using the present value method.

Kendall et al. (2008) developed an integrated life-cycle assessment and LCCA model to enhance the sustainability of concrete bridge infrastructure. The objective of this model is to compare alternative bridge deck designs from a sustainability perspective that accounts for total life-cycle costs, including agency, user, and environmental costs. A conventional concrete bridge deck and an alternative engineered cementitious composite link slab design are examined. Despite higher initial costs and greater material-related environmental impacts on a per mass basis, the link slab design results in lower life-cycle costs and reduced environmental impacts when evaluated over the entire life cycle. Traffic delay caused by construction comprises 91% of total costs for both designs. Costs to the funding agency comprise less than 3% of total costs, and environmental costs are less than 0.5%. These results show life-cycle modeling is an important decision-making tool since initial costs and agency costs are not illustrative of total life-cycle costs. Additionally, accounting for construction-related traffic delay is vital to assessing the total economic cost and environmental impact of infrastructure design decisions.

More recently, Zhang (2013) developed a new network-level pavement asset management system (PAMS) using LCCA and optimization methods. Integrated life-cycle assessment and cost analysis expand the scope of the conventional network-level PAMS from raw material extraction to end-of-life management. To aid the decision-making process, the authors applied a life-cycle optimization model to determine the near-optimal preservation strategy for a pavement network. The authors used a GIS model to enhance the network-level PAMS by collecting, managing, and visualizing pavement information data. The network-level PAMS proposed in this paper allows decision makers to preserve a healthy pavement network and minimize life-cycle energy consumption, GHG emissions, or cost as a single objective and also meet budget constraints and other agency constraints within an analysis period. In a case study about a pavement network in Michigan, authors compared the near-optimal preservation strategy to the Michigan Department of Transportation’s current preservation practice. The results of the analysis showed that the optimal preservation strategy reduces life-cycle energy consumption, GHG emissions, and cost by 20%, 24%, and 10%, respectively.

Subsection 2.5 WIM Sensor and WIM System Technologies

There are several different WIM sensor technologies, including piezoelectric sensor, bending plate, and load cell. The three types of piezoelectric sensors are piezoceramic (a ceramic material surrounded between a solid core and an outer sheath of copper), piezopolymer or polyvinylidene difluoride (PVDF) (a piezoelectric polymer surrounded by a flat brass casing), and piezoquartz or quartz sensor (quartz-sensing material placed in an aluminum alloy extrusion and surrounded with elastic material).

PVDF Sensor

The piezoelectric sensor converts the mechanical force applied by the tire pressure to the electric charge, which is assumed to be proportional to the tire pressure or the vehicle weight. The PVDF sensor is the most widely used because of easy handling and installation and low price. However, PVDF sensors are not very accurate and do not have a long service life. PVDF sensors have a dynamic characteristic because the charges are generated only when forced; therefore, they cannot be used to estimate static vehicles at very low speed (less than 10 mph) like load cells can. PVDF sensors are also very susceptible to changes in environmental conditions, such as the pavement roughness, vehicle dynamic suspension, pavement materials, and pavement and ambient temperature. (Nassif et al., 2018). These limitations will produce inherent errors; however, they could be minimized by selecting the proper WIM sites (error due to roughness, vehicle suspension, and pavement material) and compensating the WIM data (error due to pavement and ambient temperature). The service life of PVDF sensor is generally 3 to 4 years depending on the severity of weather, though the majority sensors are ripped off when snow is plowed in the winter in the Northeast region. The estimated annual costs with discount rate of 10% over 20 years are reported to be approximately $3,092 to $5,000 (Mimbela et al., 2000; Bushman et al., 1998; Zhang, 2007).

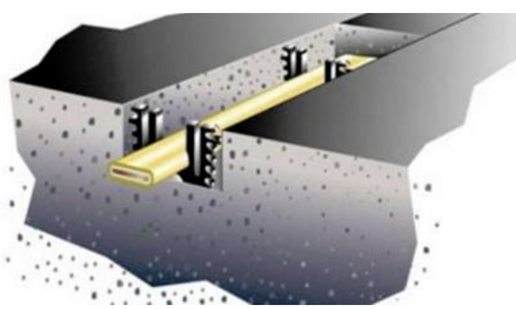


Figure 1: PVDF sensor (http://www.diamondtraffic.com).

Quartz Sensor

The piezoquartz or quartz sensor is an emerging type of piezo sensor. Unlike PVDF sensors, quartz sensors are insensitive to temperature and force direction and are therefore more accurate than PVDF sensors. The accuracy of quartz sensors is comparable to load cell accuracy. Quartz sensors are sensitive to forces in the vertical direction, but they are not susceptible to forces in the horizontal direction due to vehicle inertia, because they are encased in an aluminum alloy extruded profile. Although quartz sensors are more expensive, they provide better accuracy and are more reliable than PDVF sensors in estimating vehicle weights because they are insensitive to temperature variation and force direction. The service life of quartz sensors is not well defined because they were only recently developed, but manufacturers claim that they will last more than 15 years. Quartz sensors can provide an accuracy of ±10%/GVW with 100% confidence (Zhang, 2007).

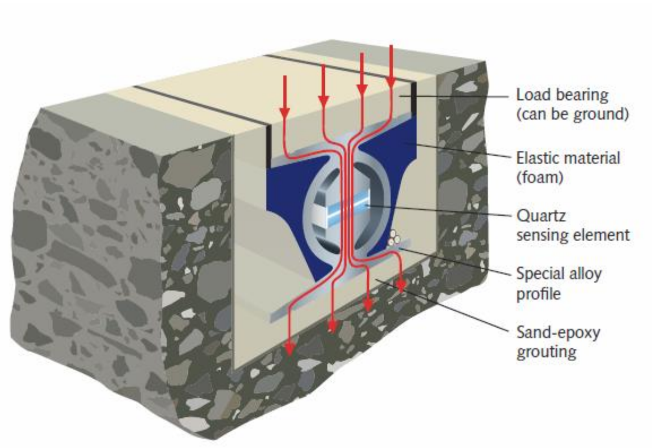


Figure 2: Quartz sensor (https://www.traffic-data-systems.net).

Load Cell Sensor

Single load cell sensors use a dynamic-rated load cell that is embedded in an enclosure covered by a rigid steel plate to measure the vehicle weight. The wheel force directly presses the single load cell that is proportional to the vehicle weight. The load cell does not have any inherent error due to temperature and pavement type because the load cell is encased in solid material. The enclosure provides good accuracy, and the load cell provides more accurate weight data than PVDF and bending plate sensors. The GVW tolerance of the single load cell is within ±6% of the actual vehicle weight. However, the load cell requires a huge cut of pavement and is therefore the most expensive type of sensor to install and maintain. The estimated annual cost is approximately $5,982 to $8,300 with a 10% discount rate over 20 years (Mimbela et al., 2000; Bushman et al., 1998; Zhang, 2007).

Figure 3: Load cell and installation example (https://www.irdinc.com).

Bending Plate Sensor

The bending plate is encased in solid material covered by a steel plate and, like the load cell, is not affected by temperature or other environmental conditions. The bending plate uses the strain gauge to measure the vehicle load attached to the underside of the steel plate. When the steel plate is bent by a vehicle, a strain signal is transmitted to the WIM system that is proportional to the vehicle load. This sensor is less expensive than the single load cell but provides equivalent accuracy within 10% tolerance. The accuracy and installation cost of the bending plate WIM system falls between that of PVDF sensor and load cell WIM system. The estimated annual cost is between $4,636 and $6,400 (Mimbela et al., 2000; Bushman et al., 1998; Zhang 2007).

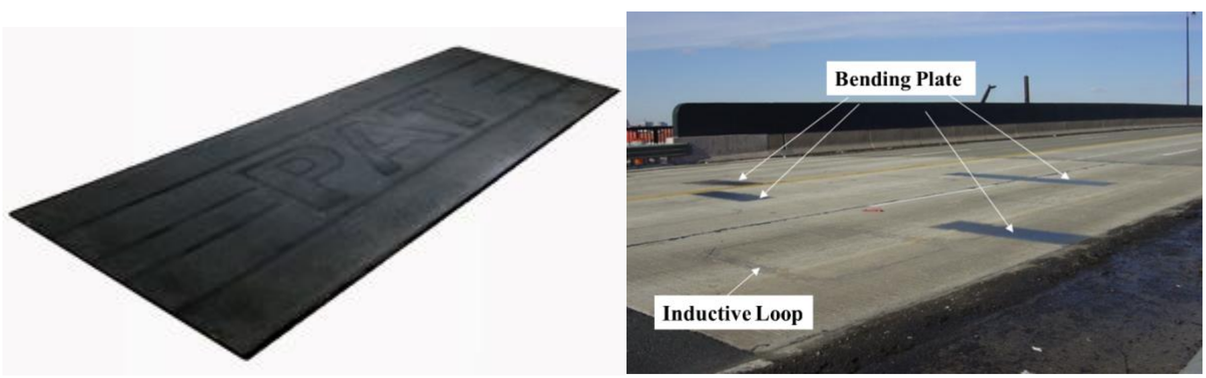


Figure 4: Bending plate and installation example (https://www.irdinc.com; Nassif et al., 2005).

Table 1 summarizes the comparison between different WIM sensors in terms of expected service life, annual cost, accuracy, weight enforcement, and reliability.

Sensor Technology	Expected Service Life	Annual cost	Accuracy (GVW, 95% confidence)	Weight Enforcement	Reliability
Bending Plate	5–6 yrs.	$4,636–$6,400	± 10%	Yes (limited)	Medium
Load Cell	5–12 yrs.	$5,982–$8,300	± 6%	Yes	High
Piezoelectric (PVDF)	3–4 yrs.	$3,092–$5,000	± 15%	No	Low
Piezoelectric (Lineas® Quartz)	15+ yrs.	High*	± 10% (GVW, 100% confidence)	Yes	Medium

* For Lineas® quartz, no specific annual cost is reported yet.

Table 1: Comparison of different WIM sensor technologies.

WIM Systems

Some foreign practices were reviewed, and data were compiled based on materials published from different sources, especially Europe and South Korea, where several high-accuracy WIM systems have been developed and are being implemented for future enforcement purposes. The majority of WIM systems use quartz sensors to comply with standards (ASTM Type I in the United States or COST323 in Europe). However, bending plates or load cells are not generally used in high-speed WIM systems except International Road Dynamics (IRD) Inc., probably due to the safety and installation issues. Table 2 summarizes the WIM systems that are used in Europe, Canada, and South Korea.

	Measurement Accuracy (Q = Quartz, P = Polymer Sensor)	Standard	System Implementation
Cross, Czech	GVW up to 5% (Q) ~ 20% (P)	COST323 A(5)~D+(20)	Europe, Asia
Q-Free, U.K.	GVW up to 5% (Q) ~ 15% (P)	COST323 A(5)~C(15)	Europe, Middle East
IRD, Canada	GVW up to 5% (Q) ~ 15% (P)	ASTM Type I, COST323 A(5)~C(15)	United States, America, Europe, Asia, Middle East
CAMEA, Czech Republic	GVW up to 5% (Q) ~ 15% (P)	COST323 A(5)~C(15)	Europe
NewConsTech, Korea	GVW up to 5% (Q) ~ 15% (P)	COST323 A(5)~D+(20)	Asia, South America

Table 2: Comparison of different WIM systems from vendors.

(a) Cross WIM, Czech (https://www.cross.cz)

(b) HI-TRAC TMU4, Q-Free WIM, Norway (https://www.q-free.com)

(c) iSINC WIM, IRD, USA (https://www.irdinc.com)

(d) UnicamWIM, CAMEA, Czech (https://www.camea.cz)

(e) ATS-WIM, NewConsTech, Korea

Figure 5: Different WIM systems.

Subsection 2.6 Advanced Calibration of WIM Systems

Several factors can affect the accuracy of WIM systems. Over the last several decades, a massive amount of WIM data have been collected across North America through the FHWA Long-Term Pavement Performance (LTPP) program. The LTPP program was initiated to promote extended pavement life through a comprehensive understanding of pavement performance and truck volume characteristics.

The LTPP data set represents one of the most comprehensive sources of information for scientific research on pavements with potential use for bridges. This data set is divided into six major categories and contains 26 distinct databases, as listed in Table 3. These data sets provide the base for developing more accurate procedures to supply high-quality vehicle data for decision makers and transportation planners.

Major Category	Subcategory	
Climate	Climate	
Traffic	Traffic Estimates	Monitored Traffic
Pavement Structure	Layer Thickness	Drainage
	Layer Type	Plain Cement Concrete (PCC) Reinforcement
	Geometry	PCC Joint
Pavement Monitoring	Falling-Weight Deflectometer	Friction
	Longitudinal Profile	Seasonal Effects
	Distress	Load Response
Material Characterization	Laboratory-Measured Modulus of Elasticity	
	Asphalt Concrete Creep Compliance	
	Back-calculated Elastic Modulus	Bound Base Strength
	Unbound Base and Subgrade Strength	Superpave Asphalt and Mixture Tests
	PCC Strength	PCC Thermal Coefficient
	AC Strength	Material Classification
Construction	Construction	

Table 3: Available parameters to improve WIM system accuracy (https://infopave.fhwa.dot.gov).

A statistical analysis of the parameters that affect WIM accuracy is necessary to understand how these factors are related. Several procedures can be implemented to evaluate the interdependency among these variables. The effects of the correlation between the aforementioned parameters will help to further reduce the number of major independent variables that affect the accuracy of WIM systems. Some of these procedures are listed below:

1) Grouping the parameters (e.g., temperature, International Roughness Index, pavement thickness, pavement type, and pavement material/composition) into bins of variability
2) Collecting reference histograms from historical data for axle weights, axle spacing, and gross weight for each class of trucks in the database
3) Assembling histograms for the range of variability of parameters under evaluation (experimental)
4) Fitting histograms using a smoothing technique (e.g., Gaussian mixture, kernel density estimator)
5) Performing statistical tests between the reference and experimental histograms to establish whether the two distributions are from the same population

These methods provide the steps to perform a virtual calibration of the WIM systems.

Subsection 2.7 Enforcement for Overweight Trucks

Enforcement and autonomous ticketing are key to reducing the number of overweight trucks on the infrastructure. NJDOT reported an annual truck count of 45 million in 2011 in NJ, and approximately 6.4% trucks are overweight. The truck count was collected from all WIM sites in NJ, and the trucks that violated the Federal Bridge Formula B were sorted as overweight trucks. NJDOT publishes an annual vehicle size and weight limit enforcement certification report (NJ Division of Highway Traffic Safety, 2009). This report summarizes the number of vehicles weighed or screened at four weighing stations in NJ (see Table 4) was 1,006,749 in 2009, and only 0.142% (1,430 trucks) of screened trucks were ticketed for being overweight. The overweight percentage screened at weighing stations is 0.142%, which is only 2.2% of actual overweight trucks. Table 4 shows the comparison of enforcement from 1998 to 2009. It shows that the number of trucks weighed at weighing stations has more than doubled in 12 years (487,103 in 1998 and 1,006,749 in 2009), but the number of citations issues to the trucks has decreased by 75% (5,873 in 1998 and 1,430 in 2009). Figure 6 shows the heat map of overweight trucks and the locations of static weighing stations in NJ. The majority of weighing stations are located where there is a high concentration of overweight trucks (on I-78, I-80 and I-278); however, there are also some stations away from the routes heavily used by overweight trucks (e.g., I-295). The results show that the static weighing stations are insufficient for the enforcement purposes.

Year	1998	1999	2000	2001	2002	2003	2004	2005	2006	2007	2008	2009
	Number of Vehicles Weighed											
Vehicle Weighed	487,103	499,319	470,244	512,323	511,954	510,421	471,200	417,240	297,412	589,145	1,096,480	1,006,749
Percentage Change	0%	-8%	+5%	+9%	-0.1%	-0.3%	-8%	-11%	-29%	+98%	+86%	-8%
	Number of Citations											
Number of Citations	5,873	5,061	3,334	3,565	3,286	2,652	1,798	1,235	910	2,283	2,020	1,430
Percentage Change	0%	-14%	-34%	+7%	-8%	-19%	-32%	-31%	-26%	+151%	-12%	-29%

Table 4: Comparison of enforcement effort from 1998 through 2009 (NJ Division of Highway Traffic Safety, 2009).

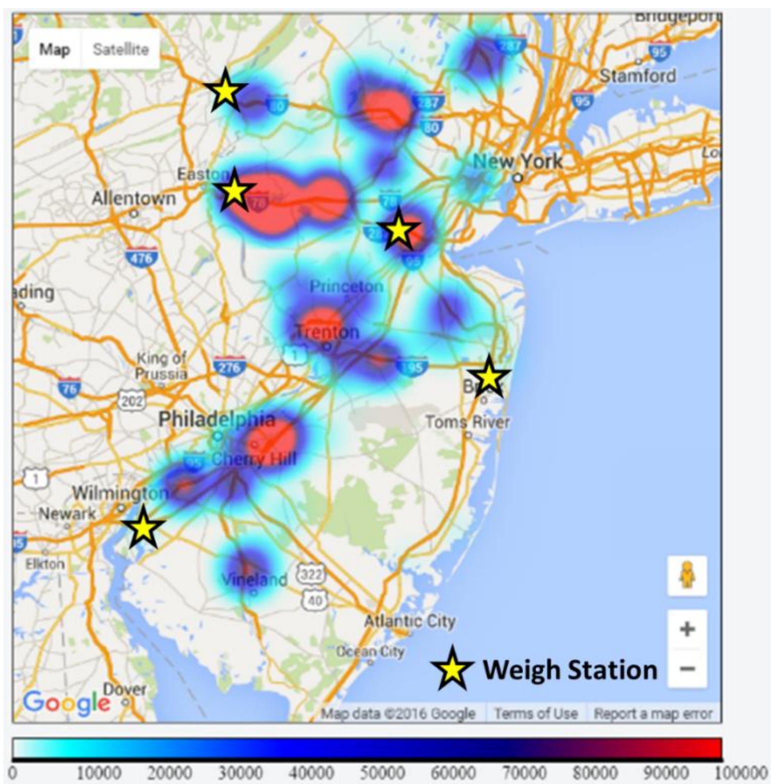


Figure 6: Overweight heat map and weigh station.

Major efforts have been made to enforce weight limits on overweight trucks using a high-speed weigh-in-motion (HS-WIM) system, which offers higher accuracy and reliability. The HS-WIM system has been used to monitor and collect traffic data from main highways. The HS-WIM system has relatively high accuracy at high speed and is integrated with an Automatic Number Plate Recognition (ANPR) system to enforce weight violations. Various HS-WIM systems were developed by Q-Free WIM Systems in the United Kingdom, CAMEA in Czech Republic, and Korea Express Highway in South Korea for this purpose. The first attempt to enforce overweight trucks was in Taiwan in 1998 using the HS-WIM with 30% GVW accuracy followed by higher accuracy of 10%. The Czech Republic government passed legislation in 2011 for enforcement using the HS-WIM with 5% GVW and 11% axle weight. Many countries, including South Korea, Netherlands, France, and Germany, implemented the HS-WIM systems with less than 5% GVW tolerance over a couple of years; however, they are currently used for prescreening overweight trucks because the legislation for the enforcement has not yet been established.

South Korea

The Korea Expressway Corporation (KEC) uses the HS-WIM system to enforce weight violation. This HS-WIM uses quartz sensors as the weighing medium with induced loops, temperature sensors, and wandering sensors. Temperature sensors are used to calibrate the accuracy of quartz sensors because the temperature change in pavement structure affects the measuring accuracy. The PVDF sensor is used for the wandering sensor, which is used to detect the location of each wheel on the pavement for improved accuracy. This sensor also detects vehicles that try to evade the lane. This system can measure the axle weight within 5% and GVW within 2% to 3% error. This HS-WIM is integrated with the ANPR system and the Variable Message Sign for violation enforcement. The KEC implemented this technology at three major highway sites in Korea. The results show a positive change in loading behavior. The number of overloaded trucks decreased from 13,035 per week to 9,598 per week over 16 weeks. For trucks over 100 kips, the number of overweight trucks decreased from 790 to 13 per week (Kwon et al., 2016).

The Netherlands

The Netherlands developed a WIM system with video called WIM+VID to monitor overweight trucks. This system uses quartz sensors, inductive loops, and cameras to monitor truck weights. Inductive loops and cameras are used on the shoulder lane to capture evading trucks. The reported margin of error for this system is 2% to 4% for GVW and 15% for axle weight (FHWA, 2007). This system is not used to enforce overweight trucks but to provide base information for the transportation engineer to focus on the corridors where more overweight violations exist. From the WIM and video information, the agency can identify the noncompliant companies that have many violations and communicate with them to prevent such violations.

Figure 7: Korean enforcement system (http://www.ctman.kr).

Figure 8: The Netherlands WIM-VID (FHWA, 2007).

Section 3 – Impact of Overweight Trucks on Pavements

To determine the impact of overweight trucks on pavements, this section first describes the methodology developed, application of methodology, and the analysis of Permit Fee System in NJ in Subsections 3.1 and 3.2. Then, the team expands the analysis to NYCDOT with a case study in Section 3.3 based on certain assumptions with limited data available. The impact costs were derived in a range estimate. The authors plan to extend this part of analysis in the future work and request more data from NYCDOT to obtain more accurate results.

Subsection 3.1 Methodology

A procedure for estimating the effect of overweight trucks on pavements was developed using overweight permit data for NJDOT (Nassif et al., 2015). The permit vehicle data were selected for this approach because they provide all necessary information such as overweight tonnage, axle configuration, and trip length. This procedure is schematically described in Figure 9. For each single overweight permit trip, the permit consists of a discrete number of links, while a road segment from one milepost to another milepost is defined. The PDC on each link was calculated using Equation 1. The ESAL in Equation 1 is computed using axle loads and axle spacings for each vehicle on the path. The data for miles and number of lanes in Equation 1 can be obtained based on the trip path of each vehicle. However, the unit pavement cost needs to be defined for each region because of the different maintenance regimes, labor and material costs, pavement characteristics, etc. Therefore, it is important to estimate the state-specific unit pavement cost in order to determine the cost of pavement damage from overweight trucks.

$$PDC = ESAL \times \text{Unit Pavement Cost} \times \text{Miles} \times \text{No. of Lanes} \tag{1}$$

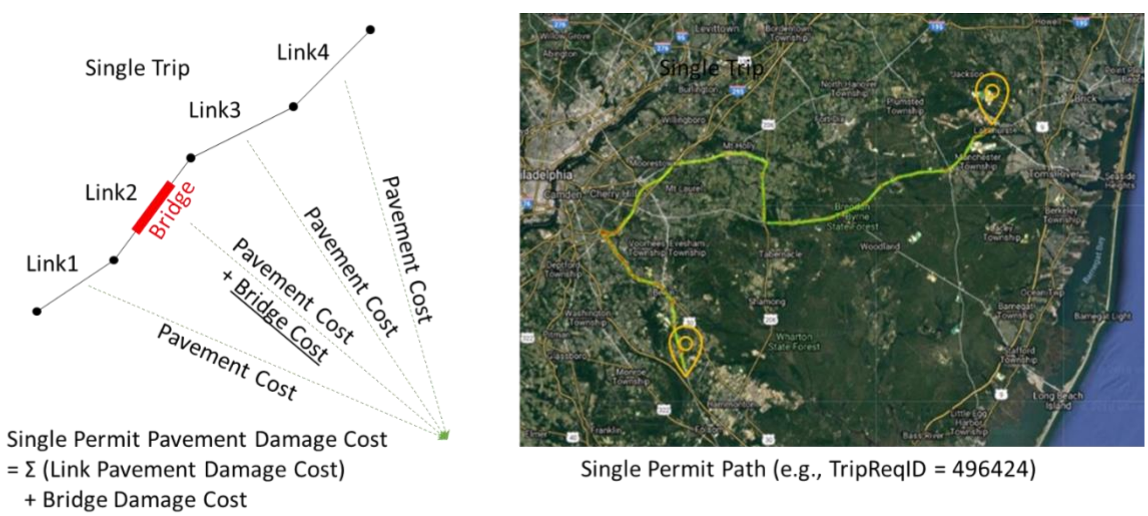


Figure 9: Pavement damage cost calculation procedure of single overweight permit trip.

In order to determine the pavement unit cost, it is important to select the representative road segments. Based on the previous research project performed by the research team (Nassif et al., 2015), two road types were selected for NJ, Interstate highways, and local roads. In this study, pavement profiles specified in Table 5 for two road types are used as the basis for developing deterioration models. The summary of this profile is obtained from the NJ study to reflect different types of pavements. In this report, these profiles are selected in order to preliminary quantify the effect of weight on the pavement damage cost. The construction history and the maintenance and rehabilitation cost data of the road segments were retrieved from the NJDOT construction database, where the cost data after 2004 were available, as summarized in Table 5. The most frequent maintenance action is the micromilling (M) and overlay of different depth of asphalt surfacing layers (O). For instance, M2.5/O2.5 indicate micromilling to a depth of 2.5 inches and overlaying with 2.5 inches of new asphalt material. The unit pavement cost per lane-mile ($/lane/mile) can be obtained by dividing the project costs by the total lane-miles, and the unit pavement costs per segments are summarized in the last column of Table 6. The project cost is the total project cost for a given maintenance project, and the pavement cost is a subset of the project cost for pavement only.

		Layer Thickness (inch)		
Road Type	**Pavement Type**	**Asphalt**	**Concrete**	**Base/Sub-base**
Interstate Highway	Composite	6	9	12
		4	9	12
		3	9	15
	Flexible	19.25	0	12
		12	0	15
		16	0	20
Local Road	Composite	3.5	7.5	12
		2.5	5.5	N/A
		5	7	N/A
	Flexible	4.5	0	20
		4	0	N/A
		4	0	N/A

Table 5: Selected pavement sites and pavement structure (Nassif et al., 2015).

Site	Lane Miles	Project (million)	Pavement (million)	Year	Treatment Type	Pavement Cost ($)	Unit Pavement Cost ($/lane/mile)
Interstate #1	48	$16.6	N/A	2008	M3/O3	N/A	0.35
	50.4	$24.95	$18.3	2008	M2/O6	0.36	0.5
Interstate #2	42.42	$19.6	$12.38	2006	M2/O4	0.29	0.46
	46.2	$11.7	$10.73	2006	M3/O3	0.23	0.25
Interstate #3	14.2	$5.0	N/A	2010	M3/O3	N/A	0.35
	51.44	$11.44	$8.0	2006	M2.5/O2.5	0.16	0.22
Local Road #1	10.28	$5.73	N/A	2009	M3.5/O3.5	N/A	0.56
	20.9	$3.1	$2.5	2006	M2/O4	0.12	0.15
Local Road #2	6	$3.17	$2.69	2006	M2/O2	0.22	0.26
Local Road #3	20.4	$3.47	$3.25	2004	M3/O4.5	0.16	0.17
Local Road #4	5.4	$3.71	$3.09	2005	M3/O3	0.2	0.24
Local Road #5	13.8	$3.1	$2.5	2006	M2/O4	0.18	0.22

Table 6: Treatment and cost data at selected sites (Nassif et al., 2015).

The pavement failure mechanism is expected to vary depending on structure, material, traffic loading, and environment. The failure criteria considered in the previous study for NJ mainly included load-related rutting and bottom-up fatigue cracking in the asphalt layer as part of the Mechanistic-Empirical Pavement Design Guide (MEPDG) (Nassif et al., 2015). In our MEPDG model, the environmental conditions are simulated by the Enhanced Integrated Climatic Model, and Newark, NJ, was selected as the climate station representative of the region.

After the unit pavement cost is obtained, an LCCA of the selected pavement is performed with the net present value (NPV) economic index to estimate the unit pavement damage cost. In the present study, the NPV is an exponential function of the number of ESALs and coefficients calibrated using the aforementioned MEPDG model (see "Marginal Pavement Damage Cost" in Nassif et al., 2015). The NPV is the sum of the present values of the individual cash flows and has been widely used in pavement LCCA. A conceptual diagram of LCCA is depicted in Figure 10.

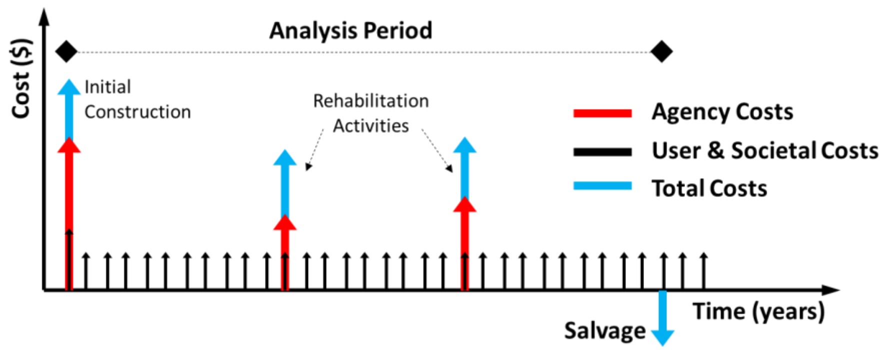


Figure 10: Conceptual cash flow diagram of a project (Kaan et al., 2004).

The NPV of the pavement can be calculated from Equation 2, which computes the discounted monetary value of the future costs and the salvage values at the end of the analysis period.

$$NPV = C + M_i\left(\frac{1}{1+r}\right)^{n_i} + \ldots + M_j\left(\frac{1}{1+r}\right)^{n_j} - S\left(\frac{1}{1+r}\right)^{N} \qquad (2)$$

Where

C = Present cost of initial rehabilitation activity;

M_i= Cost of the i-th maintenance and rehabilitation (M&R) alternative in terms of constant dollars;

r = Discount rate;

n_i = Number of years from the present to the i-th M&R activity;

N = Length of the analysis period in year;

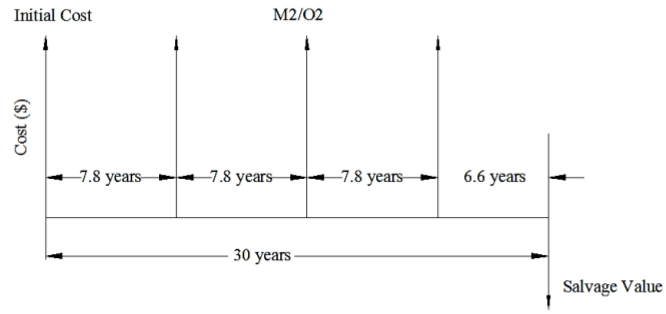
S = $\left(1 - \frac{L_A}{L_E}\right)C$ = Salvage value at the end of the analysis period;

L_A = Analysis life of the rehabilitation alternative in years;

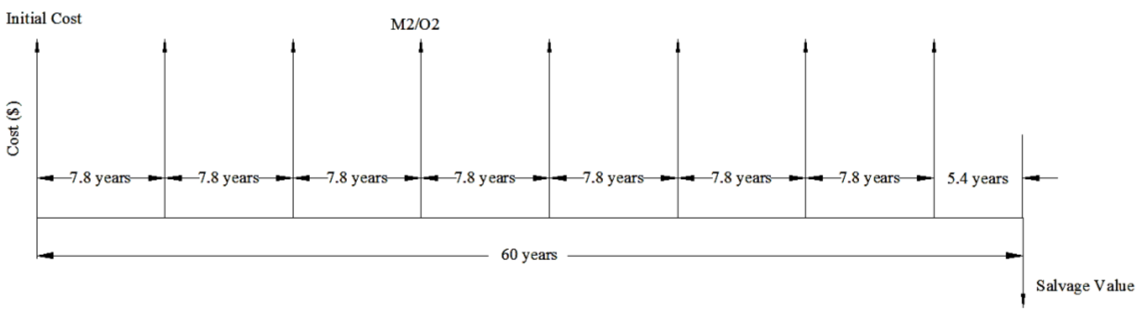
L_E = Expected life of the rehabilitation alternative; and

C = Cost of the rehabilitation alternative.

Equation 2 shows that the analysis period and discount rate are two major factors affecting the NPV. Two analysis periods (30 years and 60 years) were selected for the analysis, with a discount rate of 3%. It is assumed that successive overlays will be placed over the initial overlay, the service life of each overlay is equal, and the value of service life is 7.8 years, which is derived by Surface Distress Index analysis. Figure 11 shows how the 30-year and 60-year analyses are performed respectively for calculating NPV.



(a)



(b)

Figure 11: Activity flow with pavement service life of 7.8 years in (a) 30-year and (b) 60-year analysis period (Nassif et al., 2015).

Based on Vehicle Travel Information System (VTRIS) and WIM data, the ESAL for each year from 2010 to 2013 was calculated, and the estimated ESAL for 30 years and 60 years of the LCCA period was determined as summarized in Table 7. Therefore, the unit pavement damage cost can be obtained by dividing the NPV by the number of total ESALs for the selected road segments as presented in Equation 3. The pavement damage cost for two road types for 30-year LCCA period is summarized in Table 8. The pavement damage cost ranges from \$0.027 to \$0.052 and \$0.092 to \$0.482 per ESAL-lane-mile for each Interstate highway and local road, respectively, because of different composite type and layer thickness. The unit pavement damage cost in Eq. (3) represents the "marginal" damage cost. The pavement project costs associated with overweight trucks were isolated from various pavement project costs, so that only the pavement damage associated with overweight trucks can be accounted into the unit pavement damage cost. The team calculated the overall damage cost for two cases (case 1 includes all trucks including overweight trucks, while case 2 includes all legal trucks excluding overweight trucks) and obtained the difference in damage cost between the two cases which represents the "marginal" cost.

$$Unit\ Pavement\ Damage\ Cost\ = \frac{NPV}{ESAL_{total} \times miles \times lanes} \quad (3)$$

Route	Calculated ESAL per Year 2010	2011	2012	2013	Estimated ESAL 30 years	60 years	Source
Interstate #1	17,256	15,675	15,493	14,198	1.43E+08	2.73E+08	VTRIS
	2,816	3,176	2,775	2,524	3.05E+07	6.13E+07	VTRIS
Interstate #2	6,017	4,707	4,332	3,021	3.24E+07	5.66E+07	VTRIS
	3,363	3,022	1,509	N/A	1.77E+07	3.07E+07	VTRIS
	4,540	3,981	6,759	956	4.38E+07	8.63E+07	VTRIS
Interstate #3	N/A	1,074	1,846	1,619	3.46E+07	8.68E+07	VTRIS
Local Road #1	972	589	365	460	2.97E+06	4.62E+06	VTRIS
	383	363	356	1,125	7.15E+06	1.64E+07	VTRIS
Local Road #2	358	471	293	376	4.20E+06	8.53E+06	VTRIS
Local Road #3	514	514	514	514	8.48E+06	2.91E+07	WIM
Local Road #4	420	401	361	604	4.54E+06	9.20E+06	VTRIS
Local Road #6	241	198	183	194	1.64E+06	3.00E+06	VTRIS

Table 7: ESAL Calculation in 30-year and 60-year analysis period (Nassif et al., 2015).

Analysis Period	Road Category	Unit PDC ($/ESAL/lanes/miles) Average	Range
30 Years	Interstate highway	0.038	0.027–0.052
	State road	0.250	0.092–0.483

Table 8: Proposed PDC for NJ for 30 years of analysis period (Nassif et al., 2015).

Once the required parameter values are obtained in Equation 1, the pavement damage cost per truck is easily calculated. Table 9 shows the links for a given single permit truck (GVW = 136,440, axle count = 6, ESAL =18.9). The average unit pavement damage cost for Interstate and local roads in NJ were 0.038 and 0.250, respectively in Table 8. The number of lanes on each link can be retrieved from the NJ roadway network GIS database. The total pavement damage cost of single permit trip is obtained by summing the damage cost of each link is shown in Table 9. The pavement damage cost by single permit trucks is $73.91.

Links	Begin Milepost (MP)	End MP	No. of Lanes	Road Type	Per Vehicle Cost ($)
1	102.23	103.14	4	US Highway	2.61
2	103.14	107.46	2	US Highway	6.20
3	27.17	4.94	2	Interstate	31.93
4	61.47	67.19	2	US Highway	8.22
5	67.19	67.29	3	US Highway	0.22
6	67.29	69.45	2	US Highway	3.1
7	1.62	3.58	2	State Highway	18.52
8	15.48	15.15	2	State Highway	3.12
Summation					73.91

Table 9: Example of detailed links list of single permit trips.

Subsection 3.2 Use of the Application for Damage Cost Evaluation

With the cost equation addressed in the previous subsection, a web-based GIS application was developed to provide a useful tool to monitor the damage to NJ road infrastructure. The software architecture follows the concept of a general client (front)–server (back) model. The front user interface is designed to be user friendly to monitor the infrastructure easily using GIS maps. For the interaction between the client and the server, JavaScript and Extensible Markup Language are used to send and retrieve data from the server asynchronously without interfering with the display and behavior of the application. Amazon Web Services was used to set up the server, and the Uniform Resource Locator is used to access the application remotely. The programming languages used in the application are JavaScript, MySQL, MSSQL, PHP, and C++. The Google Maps application programming interface was also used to generate spatial maps. In this report, technical details about the software are omitted, but a flowchart showing how the damage cost of NY and NJ infrastructures is computed is presented.

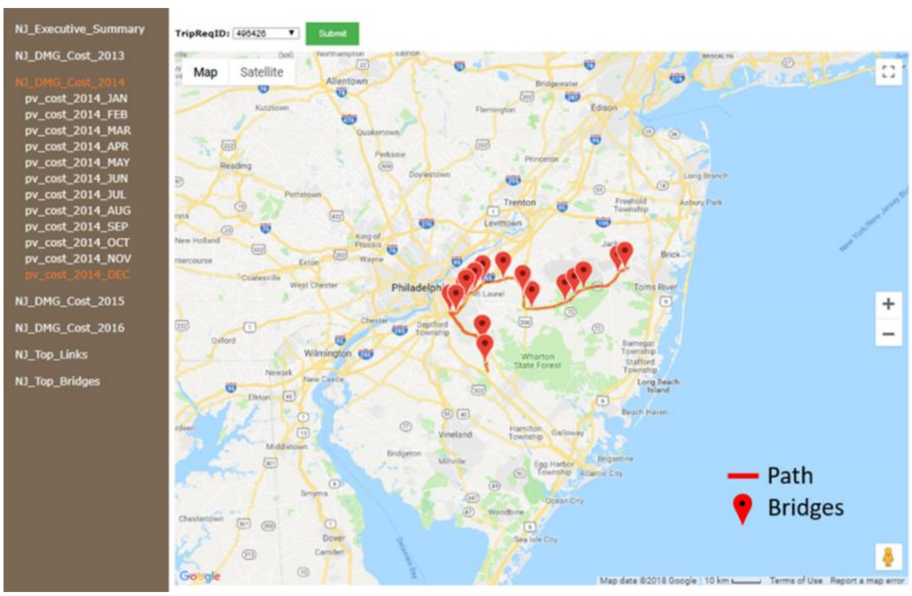


Figure 12: Graphical User Interface of Developed GIS Application for NJ Permit

The methodology to estimate the NJ pavement damage cost was applied to the entire permit trip database recorded between 2013 and 2016. The computational workload on these data sets is rather expensive, since the computation handles big data sets on the order of 100,000 trips per year (96,534 in 2013; 102,287 in 2014; 103,346 in 2015; and 108,420 in 2016). Therefore, the application was designed to be computationally efficient. The graphical user interface (GUI) for this application is shown in Figure 12. As mentioned earlier, the major feature of the GUI is to provide GIS technology and a summary of tabulated damage cost. With GIS technology, information for every permit truck can be retrieved and shown on the map at the user's request. This tool helps transportation engineers understand the effect of overweight truck trips on the roadway network.

Subsection 3.3 Extension of Developed Analysis Procedure to NYC

In this subsection, we discuss NYC-specific pavement damage cost. As described in Subsection 3.1, the unit pavement damage cost in Equation 3 should be derived, as a first step, with NYC pavement characteristics, construction history, cost, and ESAL information from various sources. Since the NYC database is limited, the team could not obtain the necessary data set to use Equation 3 to analyze the NYC case. Therefore, the team made some reasonable assumptions as below.

(1) Pavement structure is similar in both NYC and NJ. NJ and NYC have similar pavement construction and characteristics because they are in the same region and the same contractors construct the roadways. The vicinity of Alexander Hamilton Bridge (AHB) WIM station will have similar pavement structure as a NJ Interstate highway. Similarly, the vicinity of Van Dam Street (VDM) and Rockaway Boulevard (RKW) will have similar pavement structure as a NJ state road.

(2) Pavement deterioration and intervention in NYC are similar in NJ. It is well known that NJ and NYC have the heaviest truck traffic of all states in the United States. Since the pavement structure in both regions is similar, it is reasonable to assume that the pavement deterioration rate would be similar.

(3) Pavement rehabilitation/maintenance cost in NYC is higher than in NJ. RSMeans reports (RSMeans 2012a, b) summarize the material and installation (labor) cost index for NJ and NYC as presented in Table 10. According to Table 10, the weights of labor and installation cost index are approximately 56% and 44%, respectively. The average weighted material cost index is approximately 0.99 for NJ 1.02 for NYC, which is 3% higher than NJ. Similarly, the average weighted labor cost index is approximately 1.24 for NJ and 1.65 for NYC, which is 33% higher than NJ. And the average weighted total cost index is 1.10 for NJ and 1.30 for NYC, which is 18% higher than NJ. Similarly, the minimum weighted total cost index ratio between NYC and NJ is 1.16, and the maximum ratio is 1.22. Without further information, we also assume that administrative costs are higher in NYC than NJ by an amount equal to 10% of the unit pavement cost. Then, the estimated unit pavement cost in NYC is approximately 27.6% to 34.2% higher than NJ. Additionally, based on the variation of construction cost data within the five boroughs in NYC, the coefficient of variation of the construction cost is expected to be 6%.

Traffic data for NYC were obtained at the specific locations. Unfortunately, these data are currently limited because there are only three WIM stations in NYC. Please note that the WIM data do not represent all NYC traffic; therefore, this report may not provide a solid conclusion from this study. However, if the WIM network in NYC is expanded and additional overweight truck data are available, this approach will provide a reliable conclusion to quantify the cost of pavement damage cost from overweight trucks.

DIVISION		NEW JERSEY																				
		ATLANTIC CITY			CAMDEN			DOVER			ELIZABETH			HACKENSACK			JERSEY CITY			LONG BRANCH		
		082,084			081			078			072			076			073			077		
		MAT.	INST.	TOTAL	MAT.	INST.	TOTAL	MAT.	INST.	TOTAL	MAT.	INST.	TOTAL	MAT.	INST.	TOTAL	MAT.	INST.	TOTAL	MAT.	INST.	TOTAL
015433	CONTRACTOR EQUIPMENT		98.7	98.7		98.7	98.7		100.4	100.4		100.4	100.4		100.4	100.4		98.7	98.7		98.3	98.3
0241, 31 - 34	SITE & INFRASTRUCTURE, DEMOLITION	98.9	103.0	101.7	99.4	103.5	102.3	111.1	104.1	106.3	115.9	104.1	107.8	111.3	104.6	106.7	99.4	104.5	102.9	105.2	104.4	104.7
0310	Concrete Forming & Accessories	107.0	124.2	121.9	99.6	124.3	121.1	96.2	125.0	121.2	106.6	125.0	122.6	96.2	125.0	121.2	99.6	125.0	121.7	100.1	124.7	121.5
0320	Concrete Reinforcing	79.0	114.8	97.0	103.6	116.4	110.1	79.8	123.6	101.9	79.8	123.6	101.9	79.8	123.6	101.9	103.6	123.6	113.7	79.8	123.6	101.9
0330	Cast-in-Place Concrete	85.3	130.0	102.9	82.7	131.7	102.0	106.0	130.9	115.8	91.0	130.9	106.8	103.6	131.0	114.4	82.7	131.0	101.7	91.9	129.9	106.9
03	CONCRETE	97.1	123.4	109.9	98.0	124.3	110.8	105.8	125.9	115.6	100.6	125.9	112.9	103.8	125.9	114.6	98.0	125.8	111.5	102.8	125.3	113.7
04	MASONRY	97.3	127.6	115.8	89.2	127.6	112.6	93.0	128.1	114.4	108.0	128.1	120.2	96.7	128.1	115.8	86.5	128.1	111.9	100.9	127.6	117.1
05	METALS	92.7	104.6	96.6	97.8	104.7	100.1	92.7	111.9	99.1	94.2	111.9	100.1	92.8	111.9	99.1	97.9	109.6	101.8	92.8	109.4	98.3
06	WOOD, PLASTICS & COMPOSITES	109.4	123.9	117.8	99.5	123.9	113.6	98.9	123.9	113.4	112.4	123.9	119.1	98.9	123.9	113.4	99.5	123.9	113.6	100.5	123.9	114.0
07	THERMAL & MOISTURE PROTECTION	101.2	124.8	110.8	101.1	124.5	110.6	101.6	126.4	111.6	101.7	126.4	111.7	101.3	123.9	110.5	101.1	126.4	111.4	101.2	125.0	110.9
08	OPENINGS	100.0	121.6	105.4	102.4	121.6	107.2	105.4	123.6	110.0	103.4	123.6	108.5	102.6	123.6	107.9	102.4	123.6	107.8	98.4	123.6	104.8
0920	Plaster & Gypsum Board	102.9	124.0	118.0	98.7	124.0	116.8	95.2	124.0	115.8	102.9	124.0	118.0	95.2	124.0	115.8	98.7	124.0	116.8	97.4	124.0	116.4
0950, 0980	Ceilings & Acoustic Treatment	92.7	124.0	113.4	102.0	124.0	116.5	92.7	124.0	113.4	94.8	124.0	114.1	92.7	124.0	113.4	102.0	124.0	116.5	92.7	124.0	113.4
0960	Flooring	98.1	133.1	108.4	94.5	133.1	105.8	93.4	164.3	114.2	98.7	164.3	117.9	93.4	164.3	114.2	94.5	164.3	114.9	94.7	164.3	115.1
0970, 0990	Wall Finishes & Painting/Coating	90.1	121.1	109.2	90.1	121.1	109.2	90.0	123.1	110.4	90.0	123.1	110.4	90.0	123.1	110.4	90.1	123.1	110.5	90.1	121.1	109.2
09	FINISHES	100.0	125.9	114.4	100.2	125.9	114.6	97.8	131.5	116.6	101.4	131.5	118.2	97.6	131.5	116.5	100.2	131.5	117.7	98.8	131.1	116.9
COVERS	DIVS. 10 - 14, 25, 28, 41, 43, 44, 46	100.0	110.4	102.1	100.0	110.5	102.1	100.0	110.3	102.1	100.0	110.3	102.1	100.0	110.4	102.1	100.0	110.4	102.1	100.0	110.3	102.1
21, 22, 23	FIRE SUPPRESSION, PLUMBING & HVAC	99.6	123.3	109.2	100.0	124.9	110.1	99.6	125.7	110.2	100.0	125.6	110.4	99.6	125.7	110.2	100.0	125.7	110.4	99.6	124.7	109.8
26, 27, 3370	ELECTRICAL, COMMUNICATIONS & UTIL.	97.2	136.9	117.7	102.6	136.9	120.3	98.6	137.0	118.4	99.3	137.0	118.8	98.6	136.9	118.4	103.8	136.9	120.9	98.3	132.7	116.0
MF2010	WEIGHTED AVERAGE	98.1	122.1	108.7	99.4	122.6	109.6	99.7	124.6	110.7	100.6	124.5	111.2	99.4	124.5	110.5	99.4	124.4	110.4	99.0	123.3	109.7

(a) NJ

DIVISION		NEW YORK								
		NEW YORK			QUEENS			STATEN ISLAND		
		100 - 102			110			103		
		MAT.	INST.	TOTAL	MAT.	INST.	TOTAL	MAT.	INST.	TOTAL
015433	CONTRACTOR EQUIPMENT		114.3	114.3		114.5	114.5		113.9	113.9
0241, 31 - 34	SITE & INFRASTRUCTURE, DEMOLITION	123.0	125.1	124.5	115.7	128.0	124.2	127.1	124.5	125.3
0310	Concrete Forming & Accessories	106.0	184.9	174.6	92.3	176.2	165.3	92.1	176.5	165.6
0320	Concrete Reinforcing	101.5	179.8	141.0	100.5	187.6	144.4	95.9	179.8	138.2
0330	Cast-in-Place Concrete	100.4	180.1	131.8	95.4	162.5	121.8	99.7	164.2	125.1
03	CONCRETE	106.4	180.1	142.3	103.6	171.5	136.7	107.4	171.0	138.4
04	MASONRY	103.9	178.2	149.2	108.3	175.0	149.0	106.6	175.1	148.4
05	METALS	119.0	143.1	126.9	105.2	141.8	117.3	102.8	142.9	116.0
06	WOOD, PLASTICS & COMPOSITES	107.5	187.8	153.9	88.7	177.9	140.3	91.7	178.3	141.7
07	THERMAL & MOISTURE PROTECTION	104.1	166.8	129.6	105.4	163.1	128.9	104.4	164.3	128.8
08	OPENINGS	90.2	178.0	112.3	87.1	172.1	108.5	85.0	172.8	107.1
0920	Plaster & Gypsum Board	112.0	190.1	167.8	97.9	180.4	156.8	98.6	180.4	157.0
0950, 0980	Ceilings & Acoustic Treatment	102.5	190.1	160.4	82.7	180.4	147.2	83.5	180.4	147.5
0960	Flooring	100.3	177.0	122.8	104.4	177.0	125.7	95.2	177.0	119.2
0970, 0990	Wall Finishes & Painting/Coating	106.1	155.0	136.2	121.2	152.7	140.6	106.1	152.7	134.8
09	FINISHES	107.5	181.6	148.8	107.6	174.7	145.0	100.8	175.0	142.2
COVERS	DIVS. 10 - 14, 25, 28, 41, 43, 44, 46	100.0	137.9	107.5	100.0	133.5	106.7	100.0	134.3	106.8
21, 22, 23	FIRE SUPPRESSION, PLUMBING & HVAC	100.0	168.4	127.7	99.7	166.7	126.9	100.1	166.8	127.1
26, 27, 3370	ELECTRICAL, COMMUNICATIONS & UTIL.	105.0	173.3	140.3	100.1	173.3	137.9	96.9	173.3	136.3
MF2010	WEIGHTED AVERAGE	104.8	166.9	132.2	101.4	163.8	128.9	100.8	163.6	128.5

(b) NYC

Table 10: Construction cost (material and labor) for NJ and NYC (RSMeans, 2012a, b).

The overweight truck flow in the vicinity of the AHB WIM station is shown in Figure 13 for the pavement damage. A Monte Carlo simulation scheme was adopted to estimate bounds for the traffic volume on these corridors. The simulations were carried out assuming a normal distribution for the ADTT on each corridor and a coefficient of variation equal to 20%. Based on the same approach as NJ study, the unit pavement damage cost of the AHB (Interstate highway) is approximately \$0.0345 to 0.0698 per ESAL-lane-mile. Similarly, the unit pavement damage cost for VDM and RKW (local roads) is approximately \$0.117 to \$0.648 per ESAL-lane-mile. The unit pavement damage cost for NYC is approximately 27.6% to 34.2% higher than the unit pavement damage cost (\$0.027 to 0.052 per ESAL-lane-mile for the Interstate highway and \$0.092 to 0.483 per ESAL-lane-mile for local roads). More sophisticated and accurate analysis can be performed if the information for NYC pavement characteristics, construction history, and cost are given. Therefore, further research is required with the help of reliable information from NYCDOT for monitoring the performance of the road infrastructure network in NYC.

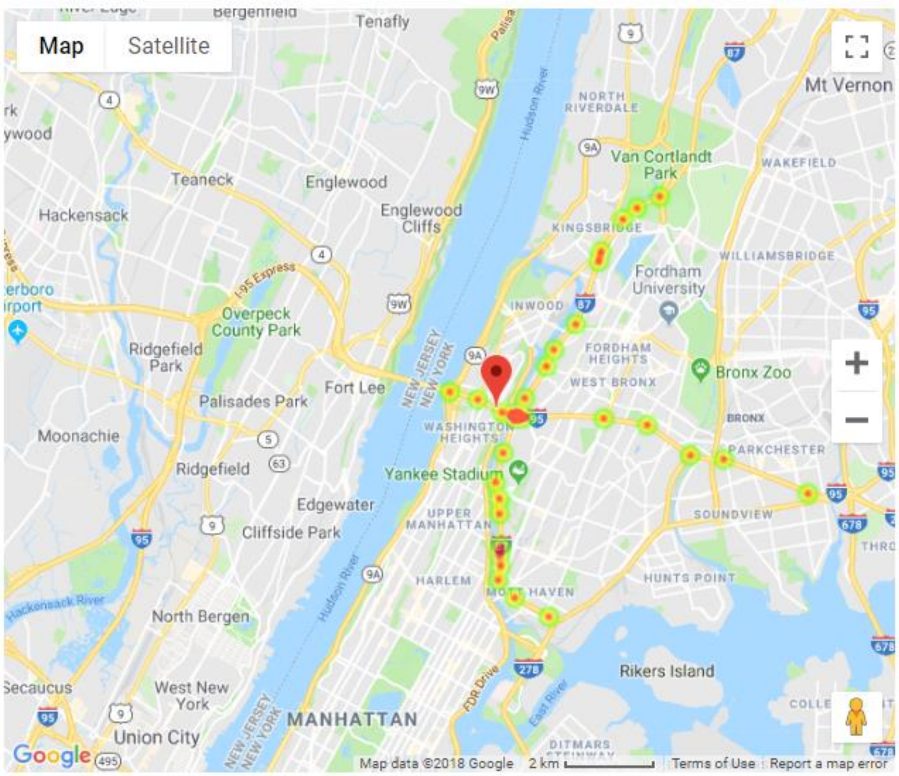


Figure 13: Overweight truck flow in the vicinity of AHB.

Section 4 – Effect of Overweight Trucks on Bridges in NYC

This section describes the approach to evaluating the effect of overweight trucks on bridges in NYC. First, traffic compositions and characteristics are analyzed through data processed from three WIM sites. This input is used to estimate the service life of different bridge components. Then we discuss deterioration models for bridge girders and bridge decks. The service life of bridge components is calculated for two cases to quantify the marginal effect of overweight vehicles. The BLCCA is used to obtain the economic impact of overweight vehicles.

Subsection 4.1 Analysis of Overweight Trucks in the Traffic Composition Based on NYC WIM

The WIM data from three NYC WIM sites (AHB, VDM, and RKW) were analyzed. The results of the statistical analysis for the three WIM sites show that the highest percentage of overweight trucks is found at the VDM station, which had 21.7% of the total truck count for 2016 and 20.3% for 2017. The AHB WIM station had the smallest percentage of overweight trucks of the three sites, with an average of 9.4% over 4 years—maximum 10.3% and minimum 8.5%—from 2014 to 2017. However, the WIM data from AHB show the highest ADTT among the three sites, and therefore the highest ADTT for overweight trucks. A summary of the WIM truck data statistics for the three sites and the years analyzed is listed in Table 11. Table 11 also shows that the percentages of overweight trucks at all the WIM sites are consistent over the years, although the number of recording days is different from year to year. As a comparison, Table 11 also lists vehicle statistics from WIM data collected on Interstates and state roads in NJ for 2016. As an example, the WIM station on Interstate I-78 shows similar ADTT and percentage of the overweight trucks as observed at the AHB WIM site. Similarly, the ADTT and percentage of overweight trucks at VDM and RKW are similar to NJ-55, which is a NJ state road.

WIM	Year	No. of Days	No. of Trucks	ADTT	No. of Overweight (OW) Trucks	OW ADTT	% of OW Trucks
AHB	2014	153	1,821,726	11,907	186,886	1,221	10.3%
	2015	306	4,005,031	13,088	393,986	1,288	9.8%
	2016	366	4,503,730	12,305	411,627	1,125	9.1%
	2017	178	1,996,659	11,217	168,985	949	8.5%
VDM	2016	294	532,922	1,813	115,598	393	21.7%
	2017	118	190,857	1,617	38,685	328	20.3%
RKW	2016	245	473,182	1,931	76,341	312	16.1%
	2017	89	160,911	1,808	23,421	263	14.6%
I-78	2016	73	917,099	12,563	77,953	1,068	8.50%
NJ-55 (551/552)	2016	365	1,138,070	3,118	146,811	402	12.90%
	2016	365	963,235	2,639	175,309	480	18.20%

Table 11: Summary of truck statistics at the NYC WIM sites compared to NJ WIM sites.

Subsection 4.2 Deterioration Models for Bridge Girders

In order to evaluate the effect of overweight vehicles on the bridge network, an estimate of each bridge's service life is needed. Three prototypes of bridges were considered in this study: (1) simple span steel multibeam bridges, (2) simple span steel girder-floorbeam bridge, and (3) simple span prestressed concrete multibeam bridge. For each type of bridge, two components—deck and girder—were considered for the cost analysis.

Steel Girders (AASHTO 2017 *Manual for Bridge Evaluation*)

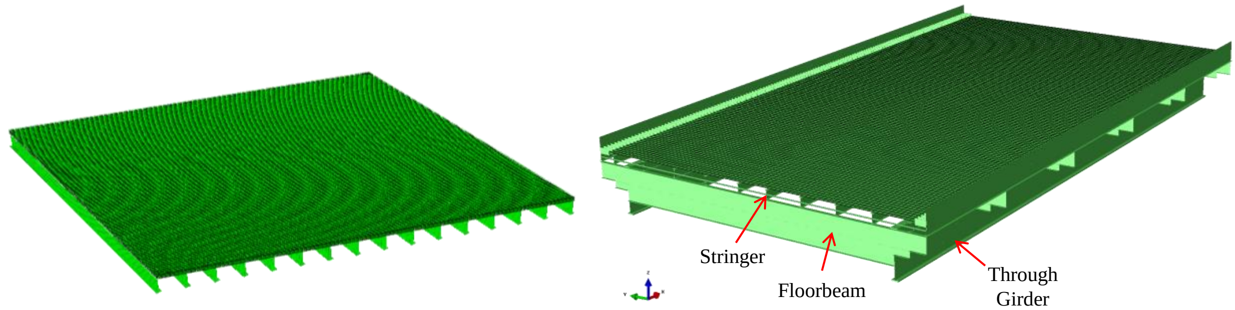
The evaluation method employed for steel-girder bridges is based on the AASHTO *Manual for Bridge Evaluation* (AASHTO, 2017). For this method, the remaining life was determined by the following Equation 4.

$$Y = \frac{\log\left[\frac{R_R A}{365n[(ADTT)_{SL}]_{PRESENT}[(\Delta f)_{eff}]^3} g(1+g)^{a-1}+1\right]}{\log(1+g)} \quad (7.2.5.1\text{-}1) \qquad (4)$$

where:

R_R = Resistance factor specified for evaluation, minimum, or mean fatigue life as given in Table 7.2.5.2-1, The Manual for Bridge Evaluation (AASHTO, 2017)
A = Detail-category constant given in Load and Resistance Factor Design (LRFD) Design Table 6.6.1.2.5-1, AASHTO LRFD Design Specifications, 2017
N = Number of stress-range cycles per truck passage estimated according to Article 7.2.5.2
g = Estimated annual traffic-growth rate, percent, expressed as a decimal; i.e., 5 percent = 0.05
A = Present age of the detail in years
$[(ADTT)_{SL}]_{PRESENT}$ = Present average number of trucks per day in a single lane
$(\Delta f)_{eff}$ = The effective stress range

Detailed structural analysis using finite element modeling was performed. The established finite element models are shown in Figure 14 to represent the multibeam bridges and girder-floorbeam bridges. The prototype 1 bridge has a span length of 86.2 ft, total width of 112.75 ft, skew angle of 21^o, 16 girders, girder spacing of 7.25 ft, slab thickness of 10 in, and girder depth of 54 in. The prototype 2 bridge has a span length of 142.6 ft, width of 106 ft, two girders, six floorbeams, and twelve stringers. The modulus of elasticity of the steel girders and diaphragms are taken as 29,000 ksi, and the Poisson's ratio is taken as 0.3. For the concrete slab, the elastic modulus and Poisson's ratio were taken as 3,600 ksi and 0.18, respectively. Neither the concrete nor steel components are expected to deform beyond the elastic range due to design loads, so only elastic material properties are considered. A dynamic impact factor of 0.15 for fatigue is included. Two traffic scenarios were created to consider the effect of overweight vehicles, one with overweight vehicles and the other without overweight vehicles. Then the fatigue life for each traffic scenarios were calculated to be used in the cost analysis.



(a) Multibeam bridge (b) Girder-floorbeam bridge

Figure 14: Finite Element (FE) models for steel bridges.

Prestressed Concrete Girders

Based on previous studies, the deterioration of prestressed concrete (PC) girders were found to be caused by the corrosion of prestressing tendons near the beam ends induced by cracking and spalling of enclosing concrete (Lou et al., 2017). A combination of external load effect and environmental effect causes the deterioration process. Based on a recent study using NJ bridge and WIM data, a strong correlation was found between the expected service life of PC girders and overweight truck count as shown in Figure 15. Along with the aforementioned PC beam-end deterioration mechanism (the cracking and corrosion of beam ends), this strong correlation indicates that overweight trucks could be the major cause of accumulated damage to PC girders. Therefore, an estimation function for the service life of PC multibeam girders was proposed in Equation 5, which would be used to estimate the service life of PC multibeam girders under different traffic scenarios. Figure 16 illustrates the estimation function for service life based on the proposed equation. Because there are limited NYC WIM data in the network, this empirical model is used to calculate the service life under different traffic scenarios.

$$y = 88.2 - 17.76 \times \log\left(ADTT \times p\right) \tag{5}$$

where

y is the estimated service life of PC multibeam girders,
p is the percentage of overweight trucks.

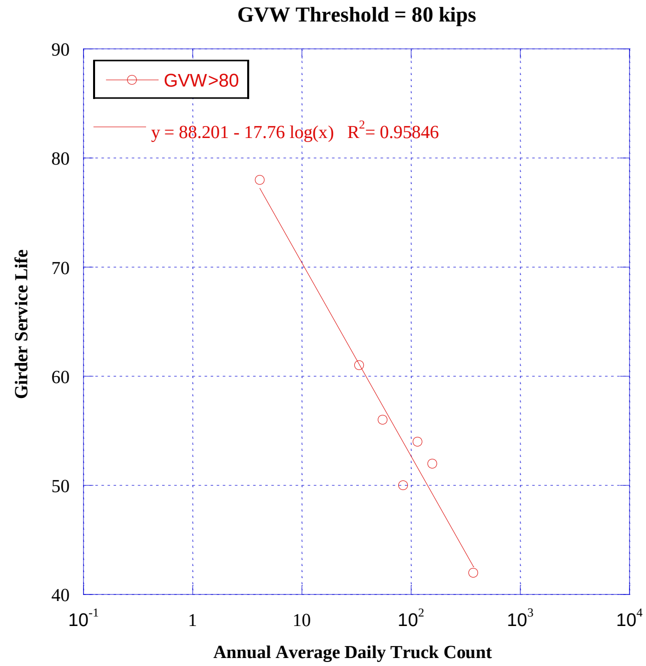


Figure 15: Expected service life vs. daily overweight truck count.

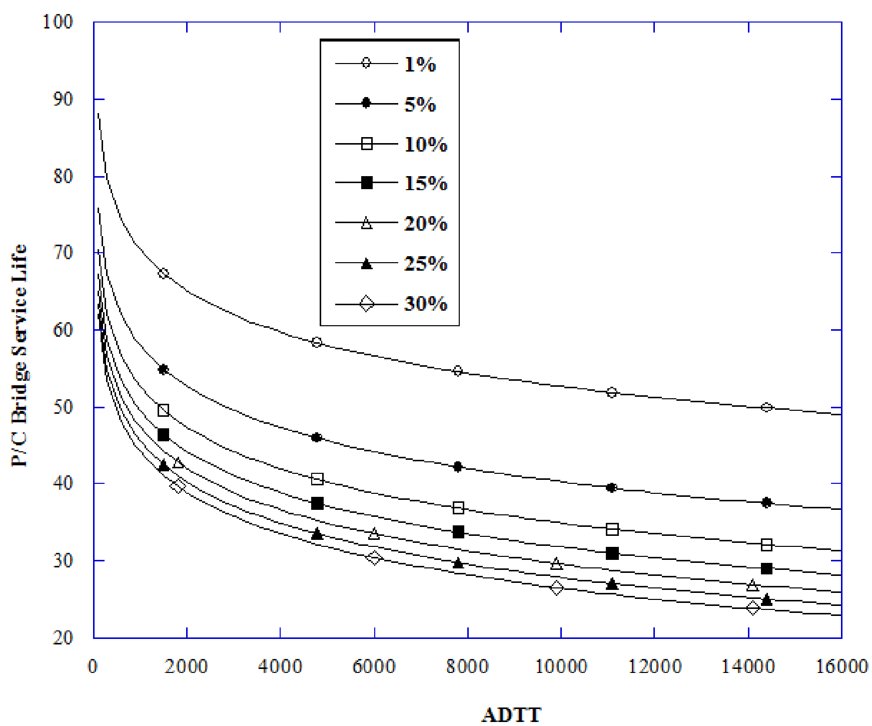


Figure 16: Estimated service life of PC multibeam bridge per % of overweight truck.

Subsection 4.3 Deterioration Models for Bridge Decks

In order to evaluate the effect of overweight trucks on bridge decks, data from the National Bridge Inventory (NBI) were processed in the vicinity of the selected WIM sites. Based on the NBI data, the average deterioration and service life of bridge decks near the WIM site are estimated and this can be treated as the deterioration of bridge decks under traffic with overweight vehicles. The data are briefly described and analyzed below for each WIM site.

AHB Site

The left panel of Figure 17 shows the map of NYC near the AHB WIM station (red square) with the allowed truck routes in NYC highlighted in brown while the red lines indicate potential paths for the overweight trucks that cross AHB in and out of the George Washington Bridge (GWB). The right panel of Figure 17 shows the location of the bridges owned by NYC (blue) and NYS (red) in proximity to the WIM station. The first column of Table 12 shows the list of the bridges, while columns two and three list the latitude and longitude of the bridges. Column four indicates the ownership of each bridge according to the NBI database. The fifth and sixth columns list the year of construction and reconstruction of the structure (if available). However, some types of intervention, such as deck overlay or the addition of extra girders to keep the bridge operational, are not considered reconstruction. The last column gives a description of the features of each bridge according to the NBI database. Given the paths for trucks in and out of the GWB, the bridges were selected following the I-87 and I-95 routes that interchange with the AHB WIM site just after the Harlem River.

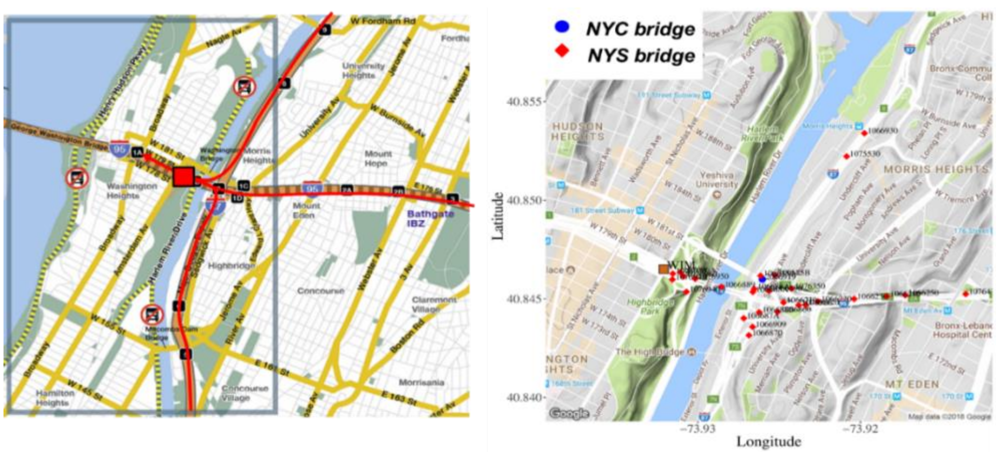


Figure 17: Major truck routes and bridges in the vicinity of the AHB WIM station.

BIN	Latitude	Longitude	Owner	Year Built	Year Reconstruction	Feature Carried
2066919	40.845992	-73.926039	NYC	1888	1992	181st Street
2242319	40.844781	-73.911319	NYC	1923	1969	Grand Concourse
2242329	40.846400	-73.909461	NYC	1909	N/A	Grand Concourse
2245480	40.845061	-73.943800	NYC	1930	2000	To GWB Opp 171 St
1066210	40.844814	-73.924733	NYS	1962	2012	Undercliff Ave
1066220	40.844869	-73.922608	NYS	1962	N/A	E L Grant Highway
1066230	40.844997	-73.920389	NYS	1964	N/A	Nelson Avenue
1066240	40.845131	-73.918367	NYS	1964	N/A	Jesup Avenue
1066250	40.845183	-73.917244	NYS	1964	N/A	Macombs Road
1066300	40.845025	-73.907539	NYS	1960	N/A	Weeks Avenue
1066850	40.846169	-73.926144	NYS	1964	N/A	Rte I95
106685A	40.845353	-73.926622	NYS	1964	2013	Rte I95
106685B	40.846194	-73.925250	NYS	1964	1992	Rte I95
1066860	40.844339	-73.925100	NYS	1962	N/A	Undercliff Avenue
1066870	40.843153	-73.926864	NYS	1964	N/A	Rte I87
106687A	40.844017	-73.927183	NYS	1964	N/A	Rte I87
1066889	40.845600	-73.928547	NYS	1962	2013	Rte I95
106688A	40.845500	-73.926536	NYS	1962	2013	Rte I87
106688B	40.844322	-73.926231	NYS	1962	1992	Rte I95
1066890	40.845411	-73.925650	NYS	1964	2013	Rte I95
1066909	40.843578	-73.926636	NYS	1964	2013	Rte I87
1066920	40.844692	-73.923372	NYS	1962	N/A	181st St to 95I NB
1066930	40.853397	-73.919758	NYS	1955	N/A	W Tremont Avenue
1076350	40.845519	-73.924233	NYS	1962	N/A	Undercliff Ave
1076360	40.844683	-73.923800	NYS	1962	N/A	Ramp From GWB
1076470	40.845244	-73.913528	NYS	1962	N/A	East 174th St
1076930	40.846278	-73.931544	NYS	1961	N/A	Ramps to/from HRD
1076940	40.845358	-73.930706	NYS	1961	N/A	Ramp to HRD SB
107694A	40.845981	-73.931572	NYS	1961	N/A	Ramp to U.S. 1
1076950	40.846017	-73.930167	NYS	1961	2011	Ramp to U.S. 1
1076960	40.846136	-73.930889	NYS	1961	N/A	Ramp to GWB
107696A	40.846350	-73.931086	NYS	1961	N/A	Ramp to GWB

Table 12: List of NYC/NYS bridges Near AHB WIM Station.

A preliminary analysis of the deck condition rating for the bridges listed in Table 12 is summarized in the deterioration curve of Figure 18 as a function of the years. The abscissa and the ordinate of each point in Figure 18 indicate the length of each cycle and the specific condition rating for the sample of bridges from 1992 to 2016 (both NYC and NYS bridges). According to this analysis, a bridge deck, on the average, would reach a condition rating of 4 after approximately 32 years.

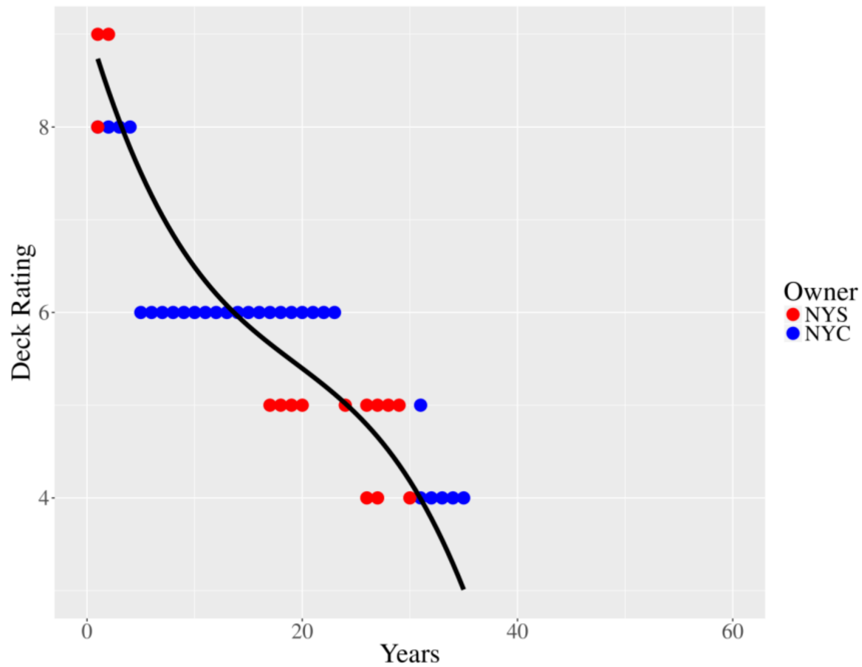


Figure 18: Deck condition rating time series for all bridges near AHB.

VDM Site

The left panel of Figure 19 shows the map of NYC near the VDM WIM station, including the allowed truck routes (brown), while the red lines indicate potential paths for the overweight trucks to cross the WIM site. The right panel of Figure 19 shows the location of the bridges owned by NYC (blue) and NYS (red) in proximity to the WIM station.

Table 13 lists the bridges owned by both NYC and NYS near the WIM site. Some of the bridges were selected on the route that from the Queensborough bridge goes on Queens Boulevard, crosses the VDM WIM site on Van Dam Street and continues on Greenpoint Avenue to Brooklyn because some trucks might turn on to Thomson Avenue or exit to the I-495 eastbound. Note that bridges were also selected on those arterials.

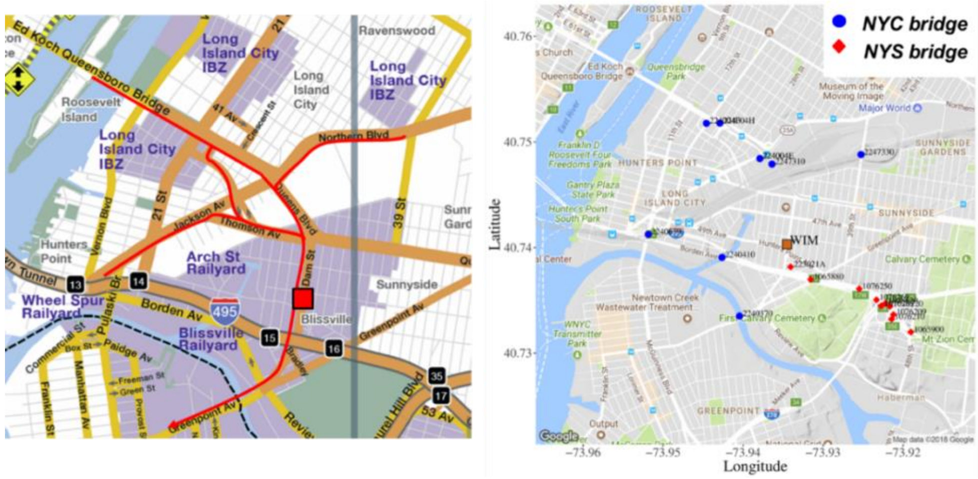


Figure 19: Major truck routes and bridges in the vicinity of the VDM WIM station.

BIN	Latitude	Longitude	Owner	Year Built	Year Reconstruction	Feature Carried
224004E	40.74843333	-73.93791111	NYC	1957	1990	Rte 25
224004F	40.75176389	-73.94458611	NYC	1957	1990	Rte 25
224004H	40.75176667	-73.94291667	NYC	1929	1980	Rte 25
224004I	40.74843333	-73.93791111	NYC	1957	1983	Rte 25
2240370	40.73350278	-73.94044444	NYC	1989	N/A	Greenpoint Avenue
2240410	40.73906389	-73.94264444	NYC	1908	1988	Borden Ave Bridge
2240639	40.74125	-73.95186389	NYC	1954	1995	McGuinness Blvd
2247310	40.74789722	-73.93638056	NYC	1910	2002	Rte 25
2247330	40.74879444	-73.92526667	NYC	1910	1995	39th Street (N.BR)
1065880	40.73697778	-73.93154444	NYS	1969	2002	Greenpoint Avenue
1065900	40.73199722	-73.91902222	NYS	1955	N/A	48th Street
1076209	40.73364722	-73.92121111	NYS	1969	2002	Rte I-495
1076210	40.73321944	-73.92143056	NYS	1969	N/A	Rte I-495
1076220	40.73442778	-73.92163611	NYS	1969	N/A	LIE WB Servc Road
1076239	40.73447222	-73.922725	NYS	1969	2003	Rte I-495
1076250	40.73608333	-73.92548333	NYS	1969	2002	Rte I-495
1076269	40.73504722	-73.92333056	NYS	1969	2002	Rte I-495
1078620	40.7347	-73.92223889	NYS	2003	N/A	I-495 WB TO GRN PT
223021A	40.73816944	-73.93406389	NYS	1940	1995	Van Dam Street Ramp

Table 13: List of NYC/NYS bridges near VDM WIM station.

The deck condition rating for the bridges listed in Table 13 is shown in the deterioration curve of Figure 20 as a function of years. The abscissa and the ordinate of each point in Figure 20 indicate the length of each cycle and the specific condition rating available for the sample of bridges from 1992 to 2016 (both NYC and NYS bridges). According to this analysis, a bridge deck, on the average, would reach a condition rating of 4 approximately 36 years after construction or reconstruction.

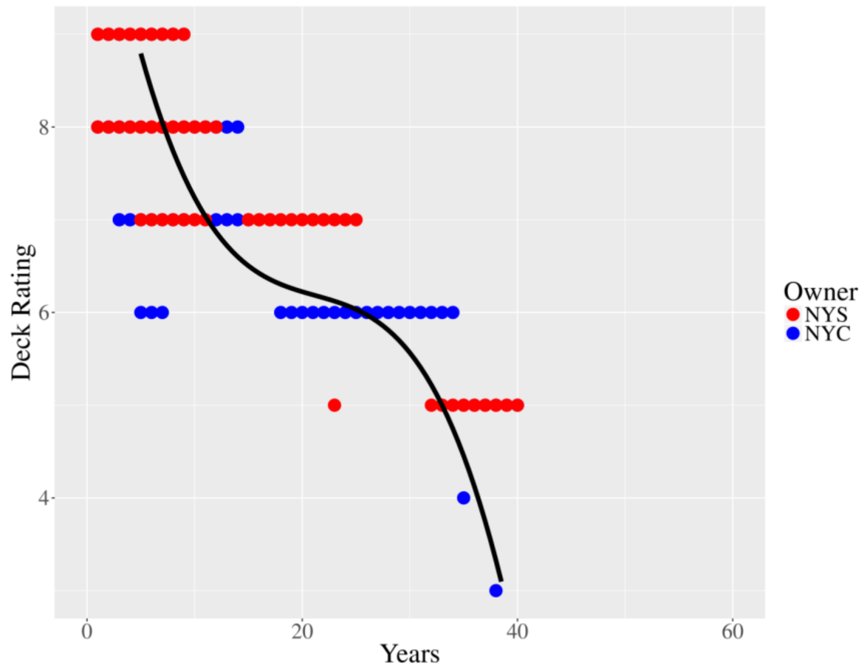


Figure 20: Deck condition rating time series for all bridges near VDM.

RKW Site

The left panel of Figure 21 shows the map of NYC near the RKW WIM station and the allowed truck routes (brown), while the red lines indicate potential paths for the overweight trucks to cross the WIM site. The right panel of Figure 21 shows the location of the bridges owned by NYC (blue) and NYS (red) in the proximity of the WIM station.

Table 14 lists the bridges owned by both NYC and NYS near the WIM site. Truck traffic from the RKW WIM site can either split west and join I-678, which connects the north of Queens to JFK Airport or can merge on Conduit Avenue either northbound or southbound; therefore, some of the bridges were selected from these two corridors.

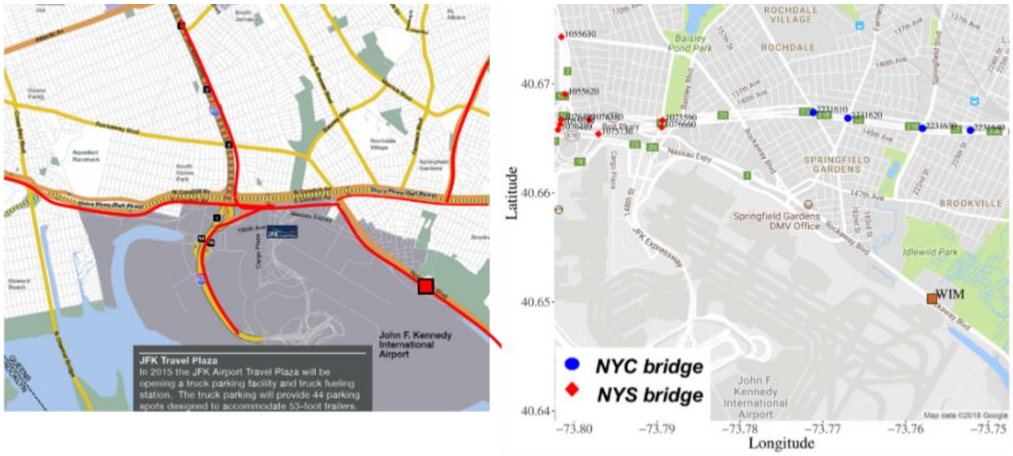


Figure 21: Major truck routes and bridges in the vicinity of the RKW WIM station.

BIN	Latitude	Longitude	Owner	Year Built	Year Reconstruction	Feature Carried
2231590	40.666631	-73.810325	NYC	1941	2000	130th Street
2231610	40.667372	-73.771153	NYC	2003	N/A	Guy Brewer Blvd
2231620	40.666861	-73.766975	NYC	1940	1994	Farmers Boulevard
2231630	40.665917	-73.757989	NYC	1940	N/A	Springfield Blvd
2231640	40.665747	-73.752183	NYC	1940	1986	225th St
2231650	40.665747	-73.741347	NYC	1947	1995	Rte 27
2231660	40.665764	-73.742056	NYC	1936	N/A	Rte 27
2231670	40.666417	-73.739950	NYC	1952	N/A	N Conduit Ave Wb
2231680	40.666394	-73.740589	NYC	1936	1995	N Conduit Ave Wb
2231690	40.668556	-73.738028	NYC	1954	N/A	Francis Lewis Blvd
2231700	40.668839	-73.738261	NYC	1954	1998	Francis Lewis Bvd
1055619	40.666289	-73.801719	NYS	1948	N/A	Rte I678
1055620	40.669039	-73.801150	NYS	1988	N/A	133rd Avenue
1055630	40.674306	-73.801553	NYS	1948	N/A	Rockaway Blvd
1055640	40.677681	-73.803125	NYS	1948	2004	Foch Blvd
1075590	40.666656	-73.789361	NYS	1967	N/A	150th St
1075730	40.665403	-73.797086	NYS	1967	N/A	Rte I678
1076380	40.666694	-73.798233	NYS	1967	1996	Rte I678
1076489	40.665797	-73.801964	NYS	1948	N/A	Rte I678
1076499	40.666711	-73.801594	NYS	1948	N/A	Rte I678
1076660	40.666028	-73.789467	NYS	1939	1965	150th Street

Table 14: List of NYC/NYS bridges near RKW WIM station.

The analysis of the deck condition rating for the bridges listed in Table 14 is summarized in the deterioration curve of Figure 22 as a function of years. The abscissa and the ordinate of each point in Figure 22 indicate the length of each cycle and the specific condition rating available for the sample bridges listed in Figure 22 from 1992 to 2016 (both NYC and NYS bridges). According to this analysis, a bridge deck, on the average, would reach a condition rating of 4 after approximately 40 years of (re)-construction.

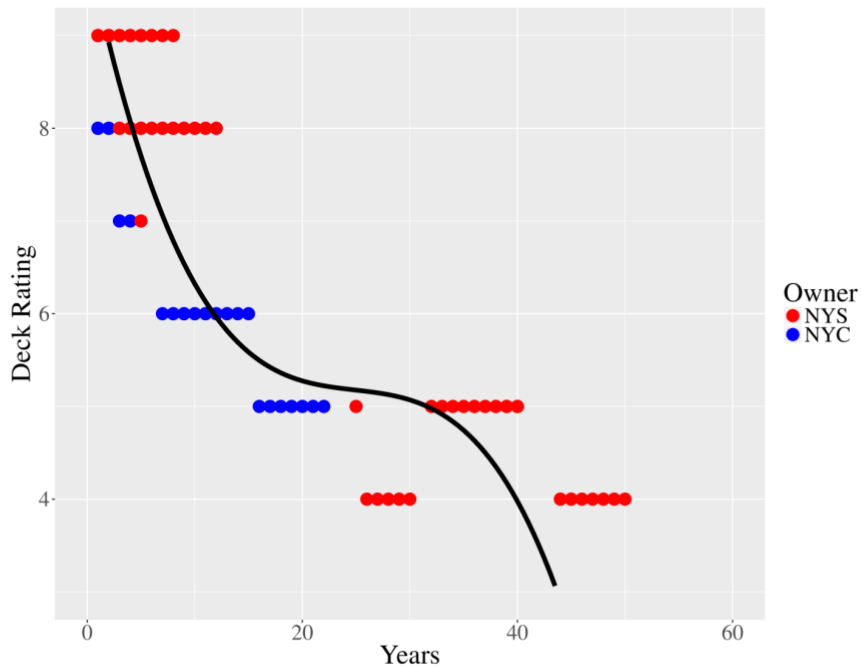


Figure 22: Deck condition rating time series for all bridges near RKW.

Subsection 4.4 Preliminary Bridge LCCA for NYC WIM Sites

After the service life of different bridge components was estimated based on the predicted functions and deterioration modeling, two scenarios were considered to quantify the economic impact of overweight trucks.

- Case 1: “All trucks” which represents current truck traffic with overweight trucks
- Case 2 (Base Case): “Legal truck” traffic only without overweight trucks

Then, the service life was calculated for the two aforementioned cases. Both cases were analyzed for a period of 75 years. The annual maintenance costs were assumed to be the same for both scenarios since the routine maintenance is usually determined by agencies’ policy regardless of traffic conditions. Annual truck traffic increase of 1.5% was assumed for all sites. The team also assumed the following for this preliminary cost analysis:

- Discount rate: 3% (typical)
- Deck replacement cost: \$600/ft^2 (estimated \$800 x 75%)
- Cost for bridge components is calculated separately to simplify the analysis
- Bridge reconstruction and replacement cost: \$4,200/ft^2

BLCCA has been required by regulation because of its importance for infrastructure investments, including the Highway Bridge Program. In BLCCA, costs are paid by either the agency or the user. Agency costs are usually the direct expenditures of funds for planning, design, construction, operation, and maintenance of a bridge. There are several economic indicators related to agency cost, such as NPV, equivalent uniform annual cost (EUAC), and salvage value (or residual value). NPV converts all costs to a single base-year cost. For assets with useful life remaining at the end of the analysis period, a salvage value should be estimated. After all agency costs are converted to NPV or EUAC, the costs of various investment options can be compared.

The NPV is defined as the sum of the present values of the individual cash flows of the same entity and has wide application in pavement LCCA. The NPV of agency cost during the analysis period is computed using the discounted monetary value of future costs, salvages by transforming costs occurring in different time periods, and salvages at the end of analysis period to a common unit of measurement. The expressions for mentioned indicators, NPV, EUAC, and salvage value, are shown in the equations below. A cash flow diagram for BLCCA corresponding to the deterioration of decks is shown in Figure 23.

$$NPV = C + M_i\left(\frac{1}{1+r}\right)^{n_i} + \ldots + M_j\left(\frac{1}{1+r}\right)^{n_j} - S\left(\frac{1}{1+r}\right)^{N} \tag{6}$$

where

NPV=Net present value or present worth;

C= Present cost of initial rehabilitation activity;

M_i= Cost of the ith maintenance and rehabilitation (M&R) alternative in terms of constant dollars;

r=Discount rate;

n_i= Number of years from the present to the ith M&R activity;

N= Length of the analysis period in years; and

S= Salvage value at the end of the analysis period.

$$EUAC = NPV\left[\frac{r(1+\mathrm{r})^{T}}{(1+\mathrm{r})^{T}-1}\right] \tag{7}$$

$$S = \left(1\text{-}\frac{L_A}{L_E}\right)C \tag{8}$$

where

S=Salvage value (or residual value) of rehabilitation alternative;

L_A=Analysis life of rehabilitation alternative in years;

L_E=Expected life of the rehabilitation alternative; and

C= Cost of the rehabilitation alternative.

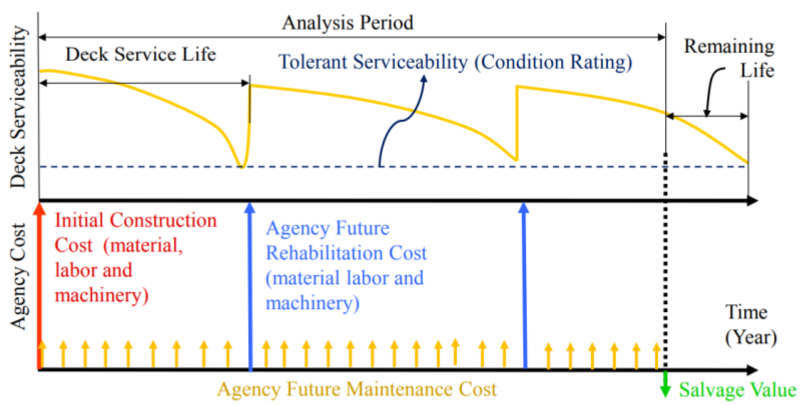


Figure 23: Cash flow diagram for BLCCA (Lou, 2016).

Damage Cost by Overweight Vehicles from Bridge Decks

The required input from WIM data analysis is summarized in Table 15. Due to the limited WIM data in NYC, the team could not perform a full correlation between the deck service life and loading. However, the team has previously performed the correlation based on NJ data. The team also performed the BLCCA based on the following approach and assumptions.

- Case 1 with "all trucks": The service life is obtained from Section 4.3. This is the actual service life based on NYC bridge data.
- Case 2 (Base Case) with "legal truck": The team first use the traffic input (Table 15) from NYC WIM data and deterioration models developed from NJ to obtain the service life reduction in percentage. This step will indicate the relative impact of overweight vehicles in NYC. Then, the service life of NYC bridges in Case 2 could be calculated using the service life reduction and service life in Case 1.

The bridge impact cost results are shown in Table 16. The EUAC is in dollar per year per deck area ($/year-ft^2). The "Case 1 Unit Cost (whole truck)" is the marginal cost (per year) divided by the total weight of overweight vehicles (per year), therefore in the unit of dollar per ton per deck area per trip. The "Case 2 Unit Cost (overweight part)" is the marginal cost (per year) divided by the total weight of only the overweight part of the overweight vehicles (per year). The bridge impact cost is the marginal cost (or the difference) between Case 1 and Case 2 unit costs. A comparison of unit cost between NJ and NYC is shown in Figure 24. Based on the data from these three WIM stations, the unit cost in NYC is significantly higher than that in NJ. The unit cost (overweight part) for bridge decks near RKW is 1.46 times the maximum found in NJ.

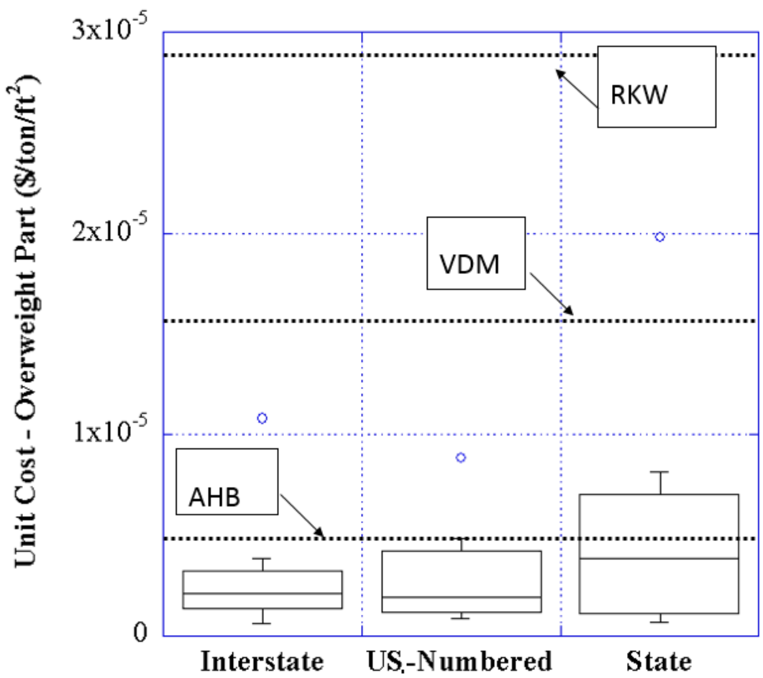


Figure 24: Comparison of unit cost for the overweight part between NJ and NYC.

Damage Cost by Overweight Vehicles from Bridge Girders

In this section, the team used the deterioration models specified in Section 4.2 to quantify the impact of overweight vehicles on different types of bridge girders. Similarly, the summary of impact costs are presented in Table 17, Table 18, and Table 19 for steel multibeam girders, steel girder-floorbeam girders, and PC girders, respectively. The same conclusions can be made that the unit cost in NYC is significantly higher than in NJ. The unit cost of the overweight part for bridge girders near RKW is 3.27 and 3.61 times the maximum found in NJ for steel multibeam and steel girder floorbeam girders, respectively.

Route	Year	Days of Data	With Overweight (Case 1)					Without Overweight (Case 2)				Only Overweight Truck			
			Avg Axles /Truck	ADTT	Axles Per Day	Equiv. wheel Weight	Equiv. GVW	ADTT (legal)	Axles Per Truck	Equiv. Wheel Weight	Tonnage	Equiv. Wheel Weight	Total Tonnage of OW Trucks	OW Tonnage of OW Trucks	OW %
AHB	2014	153	3.7	11907	44303	17.3	53.35	10685	3.6	10.4	2.47E+07	19.5	6.27E+06	5.50E+05	10%
AHB	2015	306	3.7	13088	48433	14.9	53.73	11801	3.6	9.9	5.49E+07	16.7	1.34E+07	1.33E+06	10%
AHB	2016	366	3.7	12305	45498	14.6	53.10	11181	3.6	9.9	6.14E+07	16.5	1.39E+07	1.54E+06	9%
AHB	2017	178	3.7	11217	41433	15.0	52.73	10268	3.6	9.7	2.72E+07	17.1	5.69E+06	8.34E+05	8%
VDM	2016	294	2.7	1813	4979	20.4	58.98	1419	2.5	11.4	4.42E+06	21.8	3.89E+06	1.24E+06	22%
VDM	2017	118	2.7	1617	4448	20.4	56.70	1290	2.5	11.3	1.62E+06	21.9	1.27E+06	4.18E+05	20%
RKW	2016	245	2.6	1931	4970	17.4	58.85	1620	2.3	10.7	3.69E+06	18.8	2.52E+06	8.10E+05	16%
RKW	2017	89	2.5	1808	4510	18.9	58.42	1545	2.2	10.6	1.23E+06	20.5	8.04E+05	2.56E+05	15%

Table 15: Input data from WIM for deck analysis.

Route	Service Life (Case 1)	Service Life (Case 2)	EUAC (Case 1)	EUAC (Case 2)	Marginal Cost (Case 1 - Case 2)	Unit Cost (whole truck)	Unit Cost (OW part)	Life Reduction	OW %
Unit	(years)	(years)	($/year/ft2)	($/year/ft2)	($/year/ft2)	($/ton/ft2/trip)	($/ton/ft2/trip)	(%)	(%)
AHB	41	62	$ 25.84	$ 21.96	$ 3.88	6.81E-07	4.65E-06	34%	8.5
VDM	40	102	$ 26.12	$ 19.66	$ 6.46	5.07E-06	1.54E-05	61%	20.3
RKW	35	73	$ 28.04	$ 20.60	$ 7.45	9.26E-06	2.91E-05	52%	16.1

Table 16: Economic impact of OW trucks on bridge decks at the three NYC WIM stations.

Route	Service Life (Case 1)	Service Life (Case 2)	EUAC (Case 1)	EUAC (Case 2)	Marginal Cost (Case 1 - Case 2)	Unit Cost (whole truck)	Unit Cost (OW part)	Life Reduction	OW %
Unit	(years)	(years)	($/year/ft2)	($/year/ft2)	($/year/ft2)	($/ton/ft2/trip)	($/ton/ft2/trip)	(%)	(%)
AHB	47	63	169.96	150.90	19.06	1.37E-06	1.23E-05	25%	9.1%
VDM	132	244	134.74	130.74	4.00	1.03E-06	3.22E-06	46%	21.7%
RKW	129	219	134.96	131.28	3.68	1.46E-06	4.55E-06	41%	16.1%

Table 17: Economic impact of OW trucks on steel multibeam girders at three NYC locations.

Route	Service Life (Case 1)	Service Life (Case 2)	EUAC (Case 1)	EUAC (Case 2)	Marginal Cost (Case 1 - Case 2)	Unit Cost (whole truck)	Unit Cost (OW part)	Life Reduction	OW %
Unit	(years)	(years)	($/year/ft2)	($/year/ft2)	($/year/ft2)	($/ton/ft2/trip)	($/ton/ft2/trip)	(%)	(%)
AHB	40	54	182.74	160.06	22.68	1.63E-06	1.47E-05	26%	9.1%
VDM	117	227	135.84	131.10	4.74	1.22E-06	3.82E-06	48%	21.7%
RKW	114	202	136.11	131.72	4.39	1.74E-06	5.42E-06	43%	16.1%

Table 18: Economic impact of OW trucks on steel girder-floorbeam at three NYC locations.

Route	Service Life (Case 1)	Service Life (Case 2)	EUAC (Case 1)	EUAC (Case 2)	Marginal Cost (Case 1 - Case 2)	Unit Cost (whole truck)	Unit Cost (OW part)	Life Reduction	OW %
Unit	(years)	(years)	($/year/ft2)	($/year/ft2)	($/year/ft2)	($/ton/ft2/trip)	($/ton/ft2/trip)	(%)	(%)
AHB	34	88	196.30	139.10	57.20	4.11E-06	3.70E-05	61%	9.1%
VDM	42	88	178.74	139.10	39.64	1.02E-05	3.20E-05	52%	21.7%
RKW	44	88	175.52	139.10	36.42	1.44E-05	4.50E-05	50%	16.1%

Table 19: Economic impact of OW trucks on PC girders at three NYC locations.

Section 5 – Conclusions and Recommendations

This project aims to monitor the impact of overweight trucks on bridges and pavements under the jurisdiction of NYCDOT. Unlike NJ, where large amount of truck traffic data are available from statewide WIM stations, for NYC, data from only three existing permanent fixed WIM stations were available at the time of this study. Moreover, the current condition and construction costs for pavement projects were not readily available, which required assumptions to be made for further analysis.

The methodology to estimate the unit pavement damage in NYC and NJ presented herein assumes that the pavement structure and deterioration characteristics between NJ and NYC are similar, while loads and unit costs are site specific. A differentiation between pavement profiles can also be done once data are available for NYC. Regarding the impact quantification for Interstate highways, the unit pavement damage cost in NJ varies from $0.027 to $0.052/EASL-lane-mile, while the unit pavement damage cost in NYC (AHB) ranges from approximately $0.0345 to 0.0698/ESAL-lane-mile. On the local roads (RKW and VDM), the range for unit pavement damage cost in NJ is $0.092-0.483/ESAL-lane-mile, while that for NYC is $0.117-0.648/ESAL-lane-mile. The calculation shows that the unit pavement damage cost for NYC can be up to 34.2% higher than NJ. Although in the current stage, the analysis is limited to the three locations, the approach can be easily extended to more locations if the required data can be obtained for more sites. We believe that, with more WIM data and detailed pavement information, the proposed methodology will provide a more representative estimate of pavement damage caused by overweight trucks on the roadway infrastructure in NYC.

Based on the previous research by the team and their methodology developed for evaluating the impact of overweight vehicles on the NJ bridges, the economic impact of overweight vehicles on bridges is quantified by dollar per ton of overweight per deck area per trip for three case studies in NYC (Nassif et al., 2015). The preliminary analysis of data collected for NYC WIM stations shows that a considerable reduction of the service life of bridges can be expected near these WIM stations, which increments the costs to the responsible transportation agency due to overweight vehicles. The team also found that the unit bridge damage cost in NYC is constantly higher than for most NJ highways. The unit damage cost of overweight trucks for RC bridge decks, steel multibeam girders, and steel girder-floorbeam girders near RKW is 146%, 327%, and 361% of the maximum damage cost found in NJ, respectively. However, both the limited amount of data and the quality of information available at the time of the analysis are not sufficient to provide a comprehensive evaluation of the impact of overweight trucks for the entire NYC infrastructure network. Hence, the research team recommends that the following tasks be performed to better understand the movement and concentration of the overweight vehicles on NYC roads:

- Truck data collected by NYCDOT for a period of several years using the Global Positioning System in NYC would provide a better understanding of the truck movements in the city, especially in terms of high-frequency links usage by the trucks. Even if the amount of data is limited, this analysis can provide an invaluable first input to establish origin-destination matrices in the city.

- Once areas with high truck traffic volume, based on this data set and other available data sets such as the New York Best Practice Model, are identified, the team proposes to select up to 10 key locations in the vicinity of NYC bridges to install WIM sensors. The team will periodically collect WIM data at these 10 key locations using a portable WIM system for 90 days, and data will be processed to characterize both truck configurations and weight statistics. At the same time, the truck traffic information collected at the sites will be also used to promote three (out of ten) sites with high-accuracy WIM sensors for continuous truck monitoring by NYCDOT.
- Data collected above can be integrated spatially using truck percentages from the closest WIM link and link AADTs. This assumes that the same truck traffic is correlated to the roadway segment where the closest WIM sensor is located. This method can be applicable for the close links with same or similar roadway functional class (Interstate, state, and U.S. highways). However, as the distance from the WIM sensor increases and the functional class changes, the estimation of truck traffic may not reflect the real trends for the link.
- When the data are not available, instead of obtaining a point estimate for truck traffic on a roadway link, it might be advisable to analyze the past truck traffic data and define lower and upper bounds for the truck percentages. In case there is a large variation in the overweight truck traffic based on the limited data, these percentages can be used for best/worst case scenario analysis. Moreover, if the truck traffic data for the different roadway functional classes can be obtained, the scenario analyses can be further refined based on the functional class. For example, the percentage of truck traffic on local streets is expected to be smaller than that on the Interstate highways.
- Roadway characteristics (speed limits, number of lanes, functional class) and truck traffic are both fused together into a regression model. This predictive model accounts for the spatial correlation between the links to capture a localized truck traffic trend.
- Based on both data collected from the WIM sites and the element condition ratings from the NYCDOT bridge inventory, pavements and bridge deterioration models will be calibrated for developing deterioration cost models.

The finding and the tools developed in the proposed research will be used by NYCDOT to estimate the cost of damage to NYCDOT infrastructure due to overweight truck traffic.

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