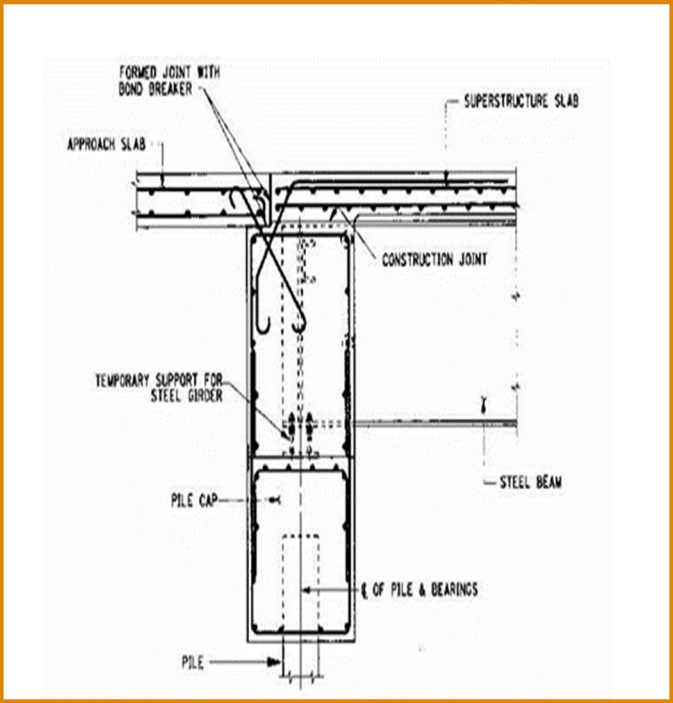
CIVIL ENGINEERING PRACTICE •

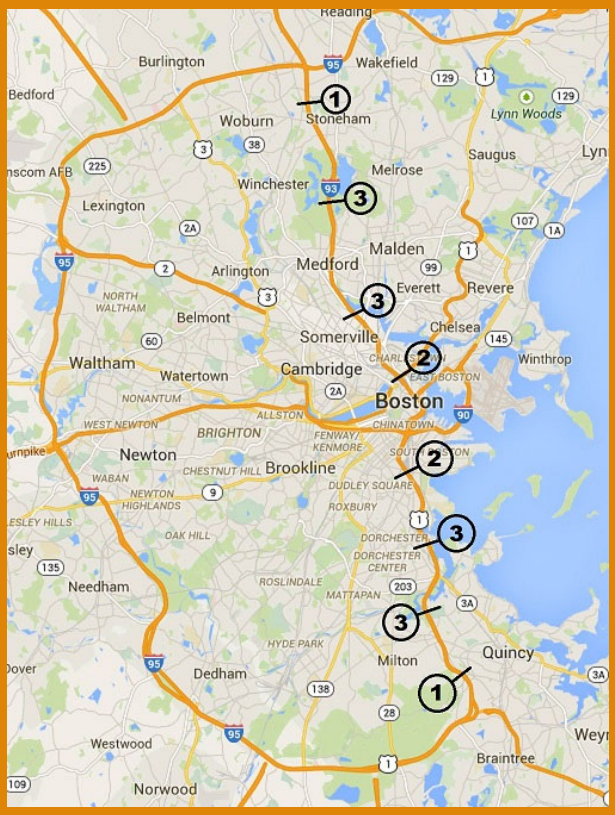
JOURNAL OF THE BOSTON SOCIETY OF CIVIL ENGINEERS SECTION / ASCE

FALL/WINTER 2020 VOLUME 28, Issue1 ISSN: 0886-9685

Pre-Drilled Holes Pile Support



Assessment of Tolls on I-93



CIVIL ENGINEERING PRACTICE • JOURNAL OF THE BOSTON SOCIETY OF CIVIL ENGINEERS SECTION / ASCE

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CIVIL ENGINEERING PRACTICE JOURNAL OF THE BOSTON SOCIETY OF CIVIL ENGINEERING SECTION / ASCE (ISSN 0886-9685) is published by the Boston Society of Civil Engineers Section / ASCE (founded 1848). Editorial, circulation, and advertising activities are located at: Boston Society of Civil Engineers Section / ASCE, The Engineering Center, One Walnut Street, Boston, MA 02108-3616; 617-227-5551. Known as the *Journal of the Boston Society of Civil Engineers Section / ASCE* until 1985, Vol. 71, Nos. 1 & 2.

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Editorial, Circulation & Sales Office:

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One Walnut Street
Boston, MA 02108-3616

617-227-5551

President's Message

Welcome to the latest incarnation of the BSCES Civil Engineering Practice Journal!

According to the BSCES web page, the "BSCES Civil Engineering Practice Journal published 98 volumes of peer reviewed technical articles almost continuously since 1914". Seventy-one volumes of the original version of the journal were published up through 1985 under the title "Journal of the Boston Society of Civil Engineers." The journal contents were evenly split between technical articles and a description of the activities of committees and members. Frequently these technical articles were extended tracts on presentations that had been previously offered at BSCES events.

At the tail end of Volume 71, published in 1985, a note relates that future issues would appear under the title *Civil Engineering Practice: Journal of the BSCES/ASCE*. Thus, was born the second incarnation of the BSCES journal (aka CEP) with the orange cover that many of us are well familiar with. It was intended to have a national readership and to "capture the spirt and substance of contemporary Civil Engineering." Further, the focus was to be on "techniques now being applied successfully in the analysis, justification, design, construction, operation and maintenance of Civil Engineering works." Volume 1 of CEP appeared in 1986 and the final volume of the printed journal, volume 27, appeared in 2011.

The quality of published articles and the range of covered topics is nothing short of astonishing. Prudential Center, Central Artery, Deer Island, Hancock Tower, Mass Turnpike, Zakim Bridge -- name the prominent engineering accomplishment and you'll find it described in some form within CEP. I just happen to have a stack of journals from 1956 to 1959 here on my desk. Perusing through these issues, the authors read like a who's who list of the engineering community that reveal the outstanding concerns of the day. Consider the following partial list of articles from those few years:

- Transportation for Greater Metropolitan Areas – Colonel S.H. Bingham, 1956
- Competitive Water Uses: Sanitary Engineering Aspects – Richard Hazen, 1957
- Analog and Digital Computers in Civil Engineering – Saul Namyet, 1957
- Disposal of Atomic Power Plant Wastes – Conrad Straub, 1957
- A Forward Look in Transportation – Ernest Herzog, 1957
- Hydraulic Model Study of Protective Works for Fleet Berths in Narragansett Bay – Peter Eagleson, 1959

While many of the techniques described over the years are no longer in common use, the entire span of articles provides an uncommon view to the state and development of engineering practice over the years. In light of the incomparable historical value of the published content, we have initiated the process of scanning and uploading the full set of published issues. The set of journals that will be initially available online are those for which multiple archived paper copies were available. The hope is that the process of scanning journal

will not end there and that eventually the full set will be accessible online.

Now for the third incarnation of the BSCES Journal! The end of the printed journal was a consequence of high production costs and the perceived diminishing interest of many engineers in documenting their projects and practices in print in today's cost-conscious business environment. We are very fortunate to have had the foresight of former journal editor, Ali Touran, to drive the effort to restart the journal in a digital format. We also applaud and are grateful to our new Editor in Chief, Dr. Gautham Das, Wentworth Institute of Technology and the volunteer editorial board, who have taken on the task of rebirth of the Civil Engineering Practice Journal. I encourage all BSCES members to read and contribute to this great resource, which is emblematic of the rich history of BSCES and its continued vitality into the future.

Dr. Bruce Jacobs, PE
President of Boston Society of Civil Engineers, ASCE Section
(2020-2021)

Message from the Editor-in-Chief

I am delighted to celebrate the launch of *BSCES Online Practice Journal*, this peer reviewed journal has over 100 publications which include innovative research and practice-oriented work in the field of Civil Engineering.

The BSCES Journal has been a repository of the amazing work that has been done in the New England/Boston Area. provide innovative approaches to deliver revolutionary cell-based products that could treat This journal is targeted for working professionals and for academics who want to publish their innovative projects and research with applications in Civil Engineering.

With the Covid-19 pandemic surging all over the world, the civil engineering and infrastructure construction and maintenance are essential to the health, safety, and economic well-being of our communities. *BSCES Online Practice Journal* is committed to keeping the profession informed during this unprecedented period.

We hope *BSCES Online Practice Journal* will become your primary platform to share findings and discuss all aspects of civil engineering in the development of future innovations that will benefit the community. We are now welcoming submissions for future issues of the journal.

Sincerely,

Gautham P Das
Editor-in-Chief
Journal.board@BSCES.org

ISSN: 0886-9685

Assessing the Impacts of Placing Tolls on Interstate Highways: An Illustrative Example on Interstate 93 in Boston

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Received 09/08/2020
Accepted for publication 11/18/2020
Published 11/21/2020

Abstract

As states continue to take on more responsibility in transportation, a major issue State Departments of Transportation (DOTs) face relates to funding future transportation investments. A funding approach being considered is the placement of tolls along selected interstate highways where tolls are not currently collected. Questions of interest to state transportation policy makers and DOT officials relate to the potential impacts of such an approach. An objective of this paper is to present a conceptual framework and analytical methods designed to assist in the evaluation of the impacts related to placing tolls on interstate highways. The application of the framework and methods is demonstrated with an illustrative example on a section of Interstate 93 in Boston. Results of the illustrative example indicate that placing tolls along selected interstate highways has the potential to provide significant revenue for State DOTs even when one considers additional investments to implement countermeasures to address potential privacy and equity impacts. Finally, it should be noted that new tolls are most effective along segments of interstate carrying large traffic volumes and providing readily available power and communication infrastructure. Other factors that can affect level of generated revenues and severity of negative impacts include location, potential for diversion, and spacing of toll plazas. Based on these results, further testing of the framework and methods is recommended as a part of impact analyses on other interstate segments where tolls are being considered. The framework and analytical methods presented in this paper will be of interest to State transportation policy-makers and fiscal planners.

Keywords: Transportation, tolls, congestion pricing

1. Introduction

The future of surface transportation funding is a major concern in the United States. The Federal and state fuel taxes, which are the major sources of revenue, have dwindled over the last few decades as a result of economic, technological, and political reasons. At the same time, expenditures to maintain and expand roadway and transit networks steadily increase and this trend is expected to continue in the future. Hence, identifying new funding approaches and revenue sources is extremely important especially as the Federal government continues to wrestle with its budget deficit and debt ceiling issues and State Department of Transportation (DOTs) are being asked to take on a larger role to suppor transportation

networks (National Transportation Policy Project, 2011). At present, many state transportation policymakers and DOT officials are considering alternative funding approaches to generate future revenue sources for transportation investments (Westervelt et al. 2015; Ahmadjian et al. 2009; Persad et al. 2004).

While there are many alternative transportation funding sources being considered by State Departments of Transportation (DOTs), an approach being explored by both state transportation policy makers and DOT officials is the placement of tolls along selected interstate highways where tolls are not currently collected. Examples of reasons for which the placement of tolls on such existing interstate highways is being considered are (WSL-JTC, 2010; Zmud and Arce, 2008):

- Current transportation funding approaches do not generate sufficient revenues to cover growing highway construction, rehabilitation, and maintenance costs;
- Charging tolls represents a simple, direct way to collect a user fee;
- Placing tolls on selected interstate highways may help to restore fairness among travelers in a region and assist in achieving regional equity goals;
- Congestion pricing as part of the toll policy may aid in accomplishing multiple policy objectives related to congestion, air quality, and energy consumption, and
- Tolls generally generate stronger public support over many other transportation funding alternatives.

Questions of interest to state transportation policymakers and DOT officials relate to the potential impacts or consequences of such approaches. Examples of these questions are:

- What will be the capital and operating costs to implement toll-based approaches on interstate highways on which tolls are not currently charged?
- What are the potential levels and nature of the revenues that can be collected with these tolls and how do these revenues compare to other funding approaches such as fuel taxes?
- What changes in demand can be expected? Will mode shifts and route diversion occur and at what levels?
- Will there be equity and privacy concerns that may lead to additional challenges in gaining public acceptance?

2. A Brief History of Toll Roads in the US

The concept of toll roads is not new. Contrary to common beliefs, toll roads became popular well before the invention of the automobile. Shortly after the American Revolution, the National Government began to realize the importance of roads for trade and the development of the country. As the result, the first toll roads on American soil appear in 18th Century. By the middle of 19th century turnpikes became common in the northeast ranging from several dozen to more than a hundred in each state. During the third decade of the 20th Century, many toll roads became limited access highways and an era of modern toll roads began (Federal Highway Administration 2012(2)).

Currently, twenty six states support some portion of their transportation budget from toll revenues and thirty-two states have various toll road authorities. While toll revenues constitute just a small part of the transportation budget in some states (Nevada, less than 1%), for other states toll receipts are indeed a major source of revenues (New Jersey, more than 50%) (Federal Highway Administration 2013; HNTB 2015).

While popular in a few areas in the U.S. for a long time period, the introduction of new toll roads has been somewhat slow throughout the rest of the nation. There are two major reasons for that phenomenon. First, federal, state, and local laws restrict implementation of tolls on many roads where tolls can be efficiently implemented. Second, traditional toll collection techniques that require construction of large toll plazas are expensive to build and operate and also create additional bottlenecks on already congested facilities.

Recent trends at all levels of government as well as modern advances in technologies suggest that these limitations and challenges can be overcome. For example, Section 1512 in Moving Ahead for Progress in the 21st Century (MAP-21), changes Federal statutory requirements governing tolls on interstates and further promotes the interstate toll pilot program (Federal Highway Administration 2012(1)). Also, the implementation of all-electronic toll collection (AETC) demonstrates the ability to reduce both capital and operating costs up to tenfold while maintaining high levels of accuracy (over 99.6% for toll transponders) without slowing down the traffic (Swank, 2013). As a result, many state transportation policymakers and DOT officials are considering new tolls to generate future revenue sources for transportation investments and looking for new methods to evaluate potential impacts associated with the implementation of toll facilities (Persad et al. 2004; Ahmadjian et al. 2009; Federal Highway Administration 2012 (1); Halsey 2014).This paper is intended to help state officials and policy-makers address these questions with the help of conceptual framework and analytical methods designed to assist in the evaluation of the impacts associated with placing tolls on interstate highways where tolls are not currently collected.

3. A Conceptual Evaluation Framework

Evaluating the impacts of alternative funding approaches (including placing tolls on existing interstate highways) is not a trivial task, because such an evaluation is complex and fraught with uncertainty (Walton et al. 2008). To facilitate the conduct of these evaluations the framework presented in Figure 1 is proposed as a guide.

The framework (4) includes four major elements:

1. Determination of policy objectives and their relative priority
2. Formulation of potential funding alternatives and associated revenue sources
3. Review of short and long range implications of each alternative
4. Identification of anticipated impacts

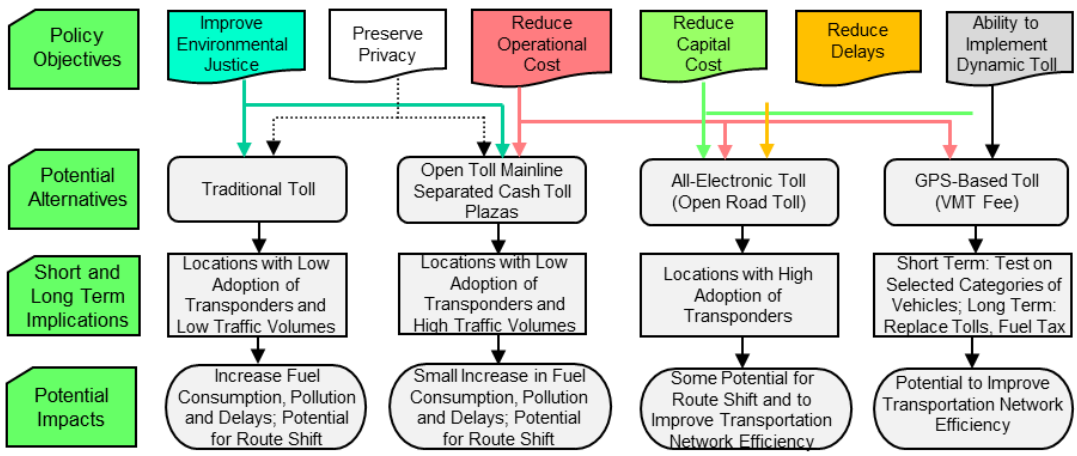

Figure 1. Toll Alternative Evaluation Framework.

Policy objectives may include: to minimize the capital and operating costs, to improve environmental justice, to preserve privacy, to promote fuel efficiency and more rapid adoption of alternative fuel vehicles, to reduce delays and to encourage more efficient use of existing and planned facilities. Potential alternatives may include implementation of traditional toll plazas that accept cash as well as electronic payments; a hybrid toll facility with Open Road Tolling (ORT) for vehicles equipped with electronic transponders along the mainline and traditional saparated toll plazas; an all-electronic Open Road Tolling (ORT); and tolling by location for GPS-equipped vehicles also known as Vehicle-Miles Traveled (VMT) fees. Short and long-term implications may include socio-geographic characteristics for a specific toll alternative, such as "areas with low adoption of electronic transponders"; specifying a level of traffic volumes, such as the "high"; or identifying a target category of vehicles, such as "heavy vehicles". Anticipated impacts relate to investment costs to implement the funding approach (e.g. capital and operating costs to collect tolls on interstate highways); changes in demand in terms of VMT, total trips, mode shift, and route diversion; level and nature of the revenue generated; and equity and privacy concerns. The magnitude of anticipated impacts would be assessed using quantitative and qualitative measures with the analytical methods presented in the next section.

4. Evaluating Impacts

The need for improved analytical methods to evaluate the impacts of various funding alternatives is well documented (Persad et al. 2004) and as suggested above such an evaluation will be a complex challenge. The evaluation approach presented in this paper is based on the premise that a unique set of benefits and costs is associated with each funding alternative, and thus for the purposes of evaluating the impacts of placing tolls on interstate highways where tolls are not presently charged, each funding alternative will be evaluated using one or more types of variables as defined below (Ahmadjian et al. 2009):

- Monetary Variables: these variables represent impacts that have a direct dollar value; examples include, for example, the expense incurred to purchase and install toll collection equipment (cost) and the amount of toll revenue collected (benefit);
- Non-Monetary, Monetizable Variables: these are variables that represent impacts not measured with direct dollar value (as is the case with monetary variables) but can be reasonably converted into monetary units; an example is the anticipated reduction (benefit) or the increase (cost) in user travel time; and
- Qualitative Variables: these are variables that represent potential benefits and costs of anticipated impacts not easily measured in monetary units (as is the case with the other two types of variables above); examples may be the benefits associated with anticipated shifts in travel modes by road users; with the provision of reduced tolls for selected population segments (e.g. local residents); or with preserving privacy.

A general formulation of impacts for each funding alternative, in its generalized form, may be represented as follows:

$$V_a = \sum M_b + \sum M_c + \sum N_b + \sum N_c \; and \; (\sum Q); \quad (1)$$

Where:

V_a is a total value of the impacts of each toll alternative
M_b is a monetary benefit
M_c is a monetary cost, usually a negative (-) value
N_b is a non-monetary, monetizable benefit
N_c is a non-monetary, monetizable cost , usually a negative (-) value, and
Q is a qualitative variable, which is analyzed separately.

It is clear, that qualitative variables that require separate evaluation may be a great challenge yet cannot be ignored. New analytical methods presented in this paper propose a more straightforward way to perform an analysis by incorporating the *cost of*

countermeasures to offset negative impacts of qualitative variables into the calculations of the total value of impacts of each toll alternative.

5. Two Analytical Methods

To assess the total value, V_a, two analytical methods are proposed (Plotnikov 2012). Because the units of the qualitative variables are not in dollars, it is proposed that the impacts associated with these variables be analyzed separately (Method 2) from the impacts represented with monetary and non-monetary, monetizable variables (Method 1). Both Methods are briefly described below and their application is demonstrated with an illustrative example in the next section of the paper.

5.1 Method 1: Analyzing Monetary and Non-monetary, Monetaizable Variables

To analyze impacts that can be measured with monetary and non-monetary, monetizable variables, it is proposed that the Net Present Value Method be used in the following form:

$$NPV = \sum_{n=1}^{n} \frac{M_{b,j,n}}{(1+i)^n} + \sum_{n=1}^{n} \frac{M_{c,j,n}}{(1+i)^n} + \sum_{n=1}^{n} \frac{N_{b,j,n}}{(1+i)^n} + \sum_{n=1}^{n} \frac{N_{c,j,n}}{(1+i)^n}; \qquad (2)$$

Where:

NPV is a net present value for a toll alternative,

$M_{b,j,n}$ is a monetary benefit for variable j during the year n,

$M_{c,j,n}$ is a monetary cost for variable j during the year n,

$N_{b,j,n}$ is a non-monetary monetizable benefit for variable j during the year n,

$N_{c,j,n}$ is a non-monetary monetizable cost for variable j during the year n, and

i is a selected discount rate.

Examples of monetary benefits for a toll alternative will be toll revenues and other revenues such as one from concessions and advertising. Monetary costs will include capital and operating costs to implement selected toll alternative including, for example, the initial cost of toll equipment and recurring operating and maintenance expenses. Non-monetary, monetizable benefits may include time savings resulted from the reduced congestion, among others. It should be pointed out that the calculation of benefits is not a trivial task and should be done cautiously, especially when non-monetary aspects are considered along with monetary benefits. To depict visually the Net Present Value for each toll alternative, it is proposed that cash flow diagrams be prepared. The application of Method 1 is presented in the illustrative example in the next section.

5.2 Method 2: Analyzing Qualitative Variables

Method 2 is proposed in order to assess impacts expressed with qualitative variables which by definition as stated above are not measured in monetary units. Method 2 consists of three steps:

Step1 includes the development of a rating scale to assess each impact. As letter grading is widely accepted in the transportation community (for example, in the conduct of highway capacity analyses), a rating scale consisting of levels A through F is proposed here. Impacts that may be assessed with a rating scale, as proposed in Tables 1, 2, and 3, include privacy, equity, and route shift. Note that the set of impacts to be analyzed is derived from the conceptual framework on a basis of specific set of policy objectives. For example, other factors that may be considered for further evaluation may include mode shift as well as impacts on the environment and other local concerns.

(a) Privacy Impact

In modern society it is practically impossible to preserve the absolute privacy of individuals traveling on today's transportation systems. However, it is desirable to give travelers a range of options to protect their identity. This will allow a wide range of users to select a level of privacy that they may desire depending on personal requirements or specific situations (Blumberg and Eckersley 2009).

Table 1 presents a rating scale designed to evaluate privacy impacts of new tolls:

As can be observed from Table 1, letter "A" is the highest available grade and represents an ideal situation of "absolute" privacy, while letter "F" represents the lowest available grade, a situation with an absolute lack of privacy. Major factors that may affect privacy include technology applications, density of toll equipment installations, presence of cameras and other sensor technologies, and choice of payment system.

(b) Equity Impact

Equity is a difficult impact to assess as it has many dimensions and can be defined in many ways (Collura and Cope 1982). In today's transportation planning lexicon equity impacts fall under the rubric of environmental justice. (Prozzi, J. et. al.) In order to improve the assessment of such impacts, the Boston MPO developed its Transportation Equity Program to provide for a systematic method of considering environmental justice in all of its transportation-planning activities. The results of these activities are incorporated in the development of MPO plans and studies. (CTPS, 2014)

During the preliminary analysis of potential environmental justice impacts on I-93 users it was found that the major impact from toll implementation will be on local residents, specifically those from minority and low-income communities. However, it was also found that changes in accessibility and activity locations can be minimized if special discount programs are made available to such residents. As a result, in the illustrative example presented in this paper, the only countermeasure that has been considered to offset equity impacts associated with placing tolls along the I-93 was a provision of special discounts for the selected categories of road users. Table 2 presents a rating scale to consider equity impacts

Table 1. Rating Scale to Assess Privacy Impacts.

LEVEL OF PRIVACY	BRIEF DESCRIPTION	EXAMPLE/COMMENT
A	No ability to detect or track vehicles or individuals	No detection
B	Low ability to detect or track vehicles or individuals	Manual data extraction from selective single-location, single-source records (e.g. recorded video)
C	Medium ability to detect or track vehicles or individuals	Automatic data extraction (e.g. ALPR) from single location, single-source records
D	High ability to detect or track vehicles or individuals	Automatic data extraction from multiple location, single-source data records
E	Very High ability to detect or track vehicles or individuals	Automatic data extraction from multiple location, multiple-source data records (e.g. video and toll transponder)
F	Full ability to detect or track vehicles and individuals inside and out of the vehicle	Automatic data extraction from continuous multiple-source data records (e.g. GPS, cellular transmitter, live HD video and ALPR)

(c) Route Shift

While some individuals might argue that route shift impacts should be measured in dollars (i.e. with monetary or non-monetary, monetizable variables), in this illustrative example and many other cases these impacts will be considered with qualitative variables. This is done for simplicity of analysis, and is considered on a basis of priorities set in policy objectives and a robustness of a network model available for a specific region. However if impacts, such as route shifts, wish to be considered in dollars with respect to travel time savings along the main route or delay on alternate routes terms, then non-monetary, monetizable variables could be employed and included in Method 1 and directly factored into the calculation of the Net Present Value. Table 3 presents a rating scale to evaluate levels of traffic diversion.

Step 2 includes the conduct of a survey to determine the views and attitudes of decision-makers regarding the impacts represented with qualitative variables. The survey is based on rating scale developed in the first step and attempt to assess collectively the decision-makers attitudes, views, and priorities of each impact represented with qualitative variables.

Step 3 estimates any additional capital and operational costs resulting from the implementation of measures required to limit undesirable impacts represented with qualitative variables. These costs will then be included in Method 1 and an extended NPV analysis will be performed.

Table 2. Rating Scale to Assess Equity Impacts.

LEVEL OF EQUITY	BRIEF DESCRIPTION
A	Discount for selected categories of users equal to or better than the discounts on other comparable facilities in the region.
B	Discount for selected categories of users equal to 80% discounts on other comparable facilities in the region.
C	Discount for selected categories of users equal to 60% discounts on other comparable facilities in the region.
D	Discount for selected categories of users equal to 40% discounts on other comparable facilities in the region.
E	Discount for selected categories of users equal to 20% discounts on other comparable facilities in the region.
F	No discounts available to any categories of users.

Table 3. Rating Scale to Assess Route Shift Impacts.

EVALUATION LEVEL	MAINLINE TRAFFIC SHIFT, %
Low (A)	Less than 5
Moderate-Low (B)	5-10
Moderate (C)	10-15
Moderate-High (D)	15-25
High (E)	25-40
Very High (F)	More then 40

6. Interstate 93 Boston: An Illustrative Example

The purpose of this section is to present an illustrative example to demonstrate the application of the conceptual framework and analytical Methods 1 and 2. A segment of I-93 in the Boston Metropolitan area will be used in the illustrative example. Central to the illustrative example are the formulation of policy objectives and alternative toll approaches as described in the conceptual framework and the analysis of anticipated impacts associated with each toll approach. Monetary variables will represent capital investment, operating costs and toll revenues. Non-monetary, monetizable variables will represent time savings for I-93 travelers resulting perhaps from less congestion due to some traffic shifts on alternative routes in the area. Qualitative variables will address impacts related to privacy, equity, and route shift on roads other then I-93.

6.1 Objectives and Selection of Toll Alternatives

The following policy objectives have been established to serve as the basis for the evaluation of toll alternatives along the segment of I-93:

- Toll revenues should be sufficient to cover operation and maintenance of the facility while providing substantial revenues to support future corridor improvements.
- Tolling schedules should be simple and easy to implement.
- Toll charges should be fair and equitable to road users; for example, proposed tolls should be comparable to tolls currently charged on similar segments of Massachusetts Turnpike and other toll facilities in Boston Metropolitan Area.
- Proposed conversions of I-93 into a toll facility should not divert significant portions of traffic onto secondary roads. However, minor shifts of traffic that will improve travel conditions for I-93 users may be considered beneficial.
- Road user privacy and equity concerns should be acknowledged and addressed in system design.

In order to satisfy major objectives listed above, an open road tolling (ORT) system on I-93 between exits 7 and 37 in the Boston Metropolitan Area has been proposed for implementation. This I-93 segment is from the I-95 interchange in Woburn north of Boston to the Route 3 interchange at the Braintree split south of Boston. The implementation of all-electronic tolls is included on the basis of recent experiences where innovations in open road tolling (ORT) technology have been implemented successfully to collect tolls at low cost and without slowing traffic (Opiola 2006; Cambridge Systematics 2010; Siegel et al. 2004).

Consistent with established policy objectives, an extensive preliminary evaluation was performed including seven potential tolling alternatives ranging from a single toll segment to a toll at every exit ramp. While the number and size of segments in each alternative vary, the total charge for the entire tolled segment of I-93 remains constant at $4. Following the preliminary analysis, three toll alternatives were subjected to a more detailed, comprehensive evaluation:

- Alternative 1: Two toll segments, $2 each (Approximate toll locations are represented with number (1) on Figure 2);
- Alternative 2: Four toll segments, $1 each (Approximate toll locations are represented with numbers (1) and (2) on Figure 2);
- Alternative 3: Eight toll segments, $0.50 each. (Approximate toll locations are represented with numbers (1), (2), and (3) on Figure 2. Also, two tolled ramps are added to minimize traffic diversion in the most congested downtown area. Ramp locations are not shown on Figure 2 because of the relatively large scale of the map.)

The first Alternative has been selected for its simplicity and low capital costs. The third Alternative has been selected for its ability to improve equity while keeping the total number of toll areas within reasonable limits. The second Alternative was selected as a compromise solution between the first and the third Alternatives.

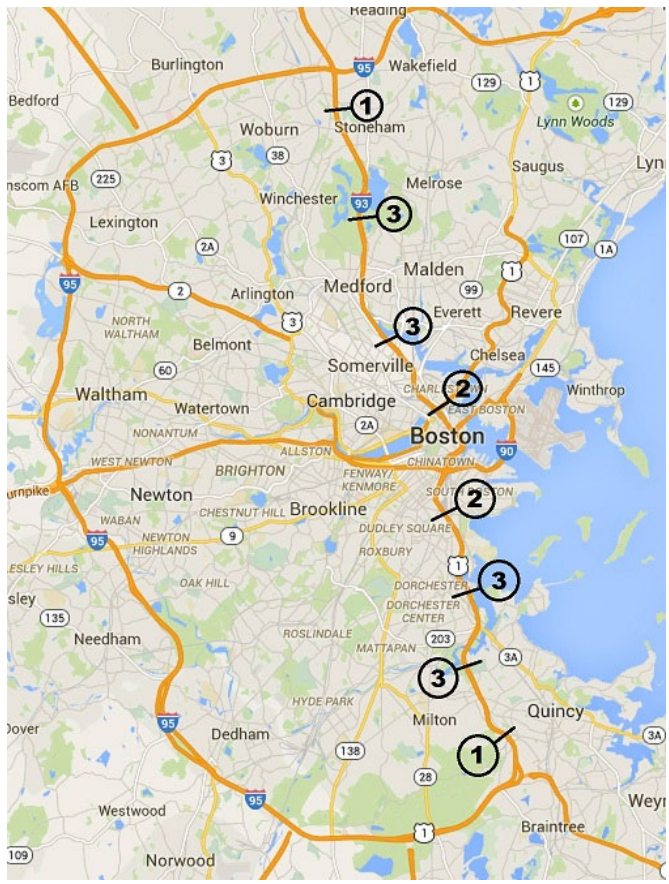

Figure 1. Approximate toll locations along I-93 for Alternatives 1, 2, and 3.

6.2 Method 1: Analyzing Monetary and Non-Monetary, Monetizable Variables

Cost Estimates (Monetary Costs, M_c)

Capital and operating cost estimates were obtained for the selected toll Alternatives based on costs incurred during the implementation of similar projects (Florida Turnpike 2011;

MassDOT 2011) and on the basis of estimates provided in relevant studies (Cambridge Systematics 2010).

Capital cost estimates include expenses associated with construction and purchasing of major items, such as electronic transponders (assumed to be distributed to road users free of charge), field equipment, such as ETC receivers and Automated License Plate Reader (ALPR), full-span gantries to carry field equipment, processing center with equipment, and communication. The cost of minor items is included in the contingency lump sum, which is assumed to be ten percent of total itemized capital costs. All capital costs are also presented in annualized form for the NPV analysis.

Operational cost estimates include the following: maintenance of electronic equipment at the gantries and at the processing center, infrastructure and communication maintenance, salary and benefits of toll road personnel. The maintenance cost of field electronic equipment was assumed to be 10 to 20 percent of equipment capital cost, depending on the selected gantry type. The maintenance cost associated with transponder operations was assumed to be 10 percent of capital cost. Cost to maintain the processing center was assumed to be 5 percent of capital cost. Operational cost to maintain and support fiber-optic trunk and related communication equipment was estimated to be 1 per cent of capital cost. In the case of existing fiber-optic network in place, it was assumed that a similar maintenance fee will be paid to lease required bandwidth from its owner. It was assumed the gate structure will not require any maintenance for the life of the structure. The average salary of personnel was assumed to be similar to the salary level of Massachusetts Turnpike employees and equal to $70,000 (Cambridge Systematics 2010). All operational costs are expressed on an annualized basis.

Revenue Estimate (Monetary Benefits, M_b)

The revenue estimates presented are based on actual (2010) traffic volumes, projected changes in traffic volumes generated by EMME - travel demand forecasting software, and a network model used in the Boston Metropolitan Area.

In order to make estimates of the potential revenues resulting from the introduction of the open road tolling on I-93, it was assumed that the following parameters will be similar to parameters observed along I-93 in Boston Metropolitan Area:

- Projected traffic growth
- Composition of traffic
- Population demographics
- Proportion of vehicles with toll transponders
- Toll rates, per mile
- Additional surcharges for vehicles without transponders, and
- Toll schedule for different categories of vehicles

Table 4 provides a comparison of capital and operating costs as well as toll revenues that might be expected from the implementation of toll collection on I-93. To acknowledge the uncertainty surrounding these cost and revenue estimates, ranges (rather than point estimates) are presented in Table 4 based on different assumptions regarding the technology option chosen.

Table 4. Costs and Revenues for Selected Toll Alternatives.

	TOLL ALTERNATIVE		
	1	2	3
Capital Costs ($M)	22.5- 61.4	28.6- 68.4	35.2- 78.1
Operational Costs ($M/year)	3.6-5.2	4.5-6.2	5.6-7.3
Toll Revenues ($M/year)	106.5-130.2	127.2-155.5	143.5-175.4

As can be observed from Table 4, both capital and operating costs vary significantly depending on a combination of selected toll Alternative and equipment options. Also, there is significant variation in projected revenues that can be attributed primarily to different levels of traffic diversion to avoid tolls. A variation of traffic diversion between alternatives resulted from a combination of the following factors: a) ease to avoid tolls and b) potential savings to traveler. However, the preliminary estimates suggest that implementation of any alternative is capable of providing sufficient revenue to cover both capital and operational costs as well as generating additional surplus of funds toward road maintenance and improvements, all within the first year of operations.

Time Savings or Losses

In order to make the impact analysis more complete, time savings or losses experienced by travelers affected by new toll road can either be monetized and included in the Net Present Value calculations or alternatively incorporated into the analysis as qualitative variables. Because calculations of travel time savings or losses for the entire road network that will be impacted by the implementation of proposed tolls can be extremely cumbersome and time consuming, only the time savings of travelers along I-93 has been estimated. Table 5 presents average travel times along the proposed I-93 toll segment. Travel time impacts on alternate routes resulting from diverted I-93 traffic were analyzed as a qualitative variable as discussed later.

The next step is to monetize projected travel time savings for NPV analysis. Total savings resulting from the reduction of travel time calculated as a sum of individual time savings for all three types of vehicles. The first type includes light/single occupancy vehicles (SOV), the second includes high occupancy vehicles (HOV) and medium trucks, and the third includes large trucks and buses. Composition of traffic and value of time by vehicle type are derived from regional traffic analysis models and presented in Table 6.

Table 5. Average Travel Times on I-93 During the AM Peak: Current Conditions and Alternatives 1, 2 and 3 for Both Directions from the I-95 Interchange in Woburn to the Route 3 Interchange at the Braintree Split.

ALTERNATIVE	AVERAGE TRAVEL TIME, MINUTES*
Current conditions: No tolls	40
Alternative 1: Two toll segments, $2 each	37
Alternative 2: Four toll segments, $1 each	36
Alternative 3: Eight toll segments and two ramps, $0.50 each	35

*Note: Travel time estimates presented in the table are based on actual (2010) traffic volumes, projected changes in traffic volumes generated by EMME - travel demand forecasting software, and a network model used in the Boston Metropolitan Area.

Table 6. Composition of Traffic and Value of Time by Vehicle Type.

VEHICLE CATEGORY	RELATIVE VOLUME IN AADT, %	VALUE OF TIME, $ PER HOUR
Type 1	67	30
Type 2	18	60
Type 3	15	120

6.3 Method 2: Analyzing Qualitative Variables

Method 2 assesses qualitative variables in three steps. The impacts of route shift, privacy, and equity will be considered as qualitative variables.

Step 1

In order to assess route shift resulting from implementation of tolls along I-93 in the Boston Metropolitan area, a comprehensive analysis for each toll Alternative was performed using EMME with the help of the Central Transportation Planning Staff in Boston.

The goal of the simulation was to identify toll schedules that will generate substantial revenues while minimizing potential negative impacts such as significant shift of traffic onto alternative secondary roads that do not have sufficient capacity. Table 7 provides a brief summary on potential route shift impacts as a result of implementing different toll Alternatives.

With the use of Tables 1, 2, 3 each toll Alternative is assessed by the transportation analyst in terms of privacy, equity, and route shift as presented in Table 8.

As can be observed from Table 8, Alternatives 1, 2 and 3 differ in anticipated levels of privacy. This is a result of several factors, including the density of toll equipment installation. It is assumed here (as indicated in Table 1) that the higher the density of toll equipment installations (including readers and cameras) the greater the potential for privacy concerns. The level of equity is graded at F in all three Alternatives because no discount was included in the original design to any category of drivers. Finally, the fewer number of toll collection locations will lead to higher levels of route shifts as higher tolls charged at a single location provide additional incentives to road users to avoid them.

Step 2

The second step in Method 2 includes the conduct of a survey to determine the views and attitudes of decision-makers regarding the impacts represented with the qualitative variables. The survey questionnaire consists of two sections and each section consists of three questions.

Questions presented in the first section ask decision-makers to identify an acceptable level for each qualitative variable on the basis of a rating scale. The rating scale uses numerical values rather then letters, unlike the rating scale presented earlier. There are two reasons for this change. The first is that only some of the decision makers are likely to be familiar with a letter-based scale that is commonly used in typical transportation level of service analyses. The second is that numerical values facilitate an evaluation of the opinions of the diverse group of decision makers by providing an "average" numerical rating that in turn can be easily converted back to a letter-based scale as necessary.

Questions presented in the second section ask decision-makers to identify their perceived level of relative importance of each qualitative variable on the basis of the rating scale provided. Again, the rating scale uses numerical values for consistency with rating scales presented in first section of the survey. After data is collected, an average score is calculated and then converted to match a desired scale (in our case it is 0 to 1, 1 being the most important). A survey also provides additional space for comments, so that additional requirements can be identified and included in the analysis.

Results of the survey are summarized in Table 9. Please note that the results presented in Table 9 are provided for illustrative purposes to demonstrate the application of Method 2.

Table 7. I-93 Mainline Volume Drop as a Result of Different Toll Alternatives.

ALT	MAINLINE ROUTE SHIFT
1	• Very High on the Northern Expressway • Very Low on the Bridge and at the Tunnel • Very High on the Southeast Expressway
2	• Moderate-High on the Northern Expressway • High on the Bridge • Very Low at the Tunnel • High on the Southeast Expressway
3	• Low on the Northern Expressway • Moderate on the Bridge • Moderate-Low at the Tunnel • Low on the Southeast Expressway

Table 8. Levels of Privacy, Equity and Route Shift for selected Toll Alternatives without Privacy and Equity Countermeasures.

ALTERNATIVE	**1**	**2**	**3**
Number of Toll Plazas	2	4	8*
Level of Privacy	C	D	E
Level of Equity	F	F	F
Level of Route Shift	F	E	G

*Note: Also two tolled ramps to minimize traffic diversion in the most congested downtown area.

Table 9. Summary of Completed Surveys on Acceptable Levels of Privacy, Equity, and Route Shift.

	DESIRED LEVEL	RELATIVE IMPORTANCE
Level of User Privacy Requirements for the Proposed Toll Road	C	0.8
Discount Requirements for the BMA Local Residents	A	0.7
Maximum Acceptable Level of Route Shift	C	0.5

Step 3

In the third step, additional capital and operational costs resulting from the implementation of countermeasures required to address impacts represented with qualitative variables are estimated.

Because the survey in Step 2 determined that the relative levels of importance of privacy (.8) and equity (.7) are viewed as high priorities by decision-makers, countermeasures to address privacy and equity concerns should be considered and where appropriate integrated into one or more Alternatives. For the purposes of the illustrative example additional costs associated with privacy countermeasures include capital costs for development of more robust and secure hardware and software system, increased operating costs due to more complex methods required for anonymous toll transactions, and potential losses of revenue due to reduced accountability of the system. Additional costs associated with equity countermeasures include potential losses of revenue associated with the development and implementation of a set of toll discounts for local residents similar to existing toll discounts on facilities in the Boston area. While countermeasures could also be formulated to address route diversion concerns, for the purposes of the illustrative example no such countermeasures were considered at this time. However, at the discretion of the decision-makers and depending on policy objectives this aspect could be revisited later.

Net Present Value Calculation for monetary and non-monetary variables

All calculations for the NPV analysis have been made on a basis of annualized benefits and costs and with the following assumptions (Cambridge Systematics 2010; Ahmadjian et al. 2009 Bazelon and Smetters, 1999), fairly typical for such economic analyses in the public sector :

- Toll rates and operating costs increase proportionally with inflation
- The discount rate is assumed to be 2% per year
- Traffic growth is assumed linear and consistent with the growth trends observed over previous decades (about 1% per year)
- The NPV is calculated for a 25 year period
- Capital costs of equipment with a life span less than 25 years are assumed to increase with the rate of inflation.

An inflation rate has been assumed to be 2% on the basis of recent trends in the U.S. economy. According to Department of Labor (Bureau of labor Statistics, 2013; Davis, 2014) , an average inflation rate during the five year period when this research has been conducted (2008-2012) was about 2.07% per year. For simplicity, the closest round inflation rate (2%) has been used for analysis.

Table 10 presents an evaluation matrix for Alternatives 1, 2, and 3. On the basis of performed evaluation, Alternative 3 appears to be the "best" way to implement tolls along the selected segment of I-93. It has the highest NPV, generates the greatest level of expected toll revenues, provides the highest time savings to travelers, and satisfies the decision-makers' privacy and equity concerns as well as the other Alternatives do, while minimizing traffic diversion.

While presenting the evaluation results in a format shown in Table 10 is appealing to some decision makers, others may prefer a graphic representation showing benefit-cost streams and NPV values (Ahmadjian and Collura 2012). Figure 3 provides an example of these benefit-cost streams and the resulting estimate of NPV for Alternative 3. Included are both monetary and non-monetary, monetizable costs identified in Method 1, as well as the monetary costs of the countermeasures intended to minimize potential privacy and equity concerns addressed in Method 2. These incorporate the capital and operating costs of the more robust and secure system and expected reduction in revenue due to the implementation of discounted tolls for local residents.

Table 10. The Evaluation Matrix for Alternatives 1, 2, and 3.

	Alternative 1	Alternative 2	Alternative 3
Initial Capital Costs, $M	22.5-61.4	28.6-68.4	35.2-78.1
Annualized Capital Costs, $M	2.1-5,6	2.7-5.9	3.3-6.5
Annual Operational Costs, $M	3.6-5.2	4.5-6.2	5.6-7.7
Annual Operational Revenues, $M	106.5-130.2	127.2-155.5	143.5-175.4
Annual Monetized Time Savings, $M*	21.0-25.7	37.9-46.4	51.6-63.1
Annual Costs to Satisfy Privacy Requirements, $M	-	4.5	7.8
Annual Costs to Satisfy Equity Requirements, $M	7.9	9.5	10.7
NPV Estimated for 25 Year Period, $B	2.5	3.2	3.8

*Note: This non-monetary benefit may or may not be counted toward the total NPV of the alternative.

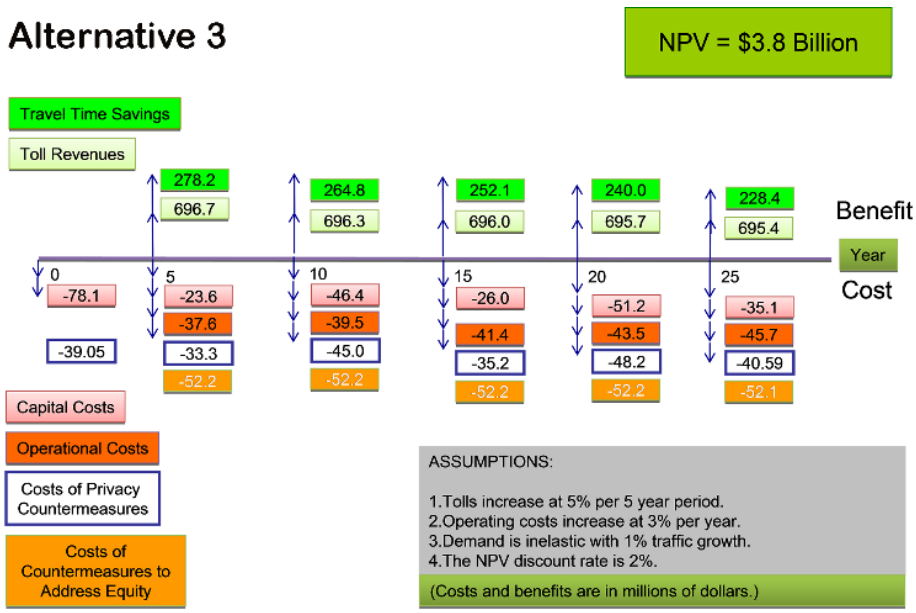

Figure 3. Benefit-Cost Streams for Alternative 3.

6. Conclusions and Recommendations

A major conclusion is that the conceptual framework and analytical methods presented in this paper can serve as useful tools for state transportation policymakers and DOT officials as they consider different toll alternatives. Another important conclusion is that careful selection of tolling schedules as well as the number and location of toll gantries along the road segment can significantly increase gross revenues, while at the same time may reduce the undesirable diversion of traffic to alternative routes that do not have sufficient capacity. The results of the illustrative example indicate that placing tolls along selected interstate highways has the potential to provide a significant source of revenue for State DOTs even if additional investments to implement countermeasures to address potential privacy and equity impacts are considered. Finally, it should be noted that new tolls are most effective along segments of interstate that carry large traffic volumes and have readily available power and communication infrastructure. Other factors that can affect level of generated revenues and severity of negative impacts include location and spacing of toll plazas. Based on the results reported in this paper, a further testing of the framework and analytical methods is recommended as a part of benefit-cost analyses on new tolls along selected interstate highways. The framework and analytical methods presented in this paper will be of interest to State transportation policy-makers and fiscal planners.

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ISSN: 0886-9685

Predrilled Holes for Pile Support of Skewed Integral Abutment Bridges

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Received 09/08/2020
Accepted for publication 11/18/2020
Published 11/21/2020

Abstract

Integral abutment bridges, despite their limitations, represent the preferred option of state transportation agencies for both new and replacement bridges. This is due to their advantages over conventional bridges, including construction and maintenance cost savings. Bridge length is one of most notable limitations of integral abutment bridges. This is attributed to the fact that longer integral abutment bridges cause high bending stresses in the piles supporting integral abutments. Consequently, finding a way to reduce pile stresses may allow construction of longer integral bridges. Predrilled oversize holes of adequate depth filled with loose sand after pile driving is one of the most effective methods to reduce pile bending stresses. This paper investigates the relation between depth of predrilled holes and reduction in pile bending stresses. Analysis results confirm the reduction in pile bending stresses as a result of predrilling holes of adequate depth filled with loose sand and effective depth for predrilled holes is discussed.

Keywords: Bridges, steel; Bridge abutments; Bridges, skew; Steel piles; Soil properties; Finite element method

1. Introduction

Integral abutment bridges are girder bridges with no expansion joints in the bridge deck and no bearings at the abutments. The ends of girders are integral with the abutments. During thermal expansion and contraction, the superstructure and abutments move together into and away from the backfill.

Integral abutment bridges have been used for decades in the United States. A testament of their excellent performance over the years is the fact that the current policy of the vast majority of states is to build integral abutment bridges whenever possible. Although the use of integral abutment bridges offers numerous advantages over conventional bridges, there are limitations on their use. This is attributed to three reasons: (1) relatively limited research has been conducted on integral abutment bridges, (2) nationally-accepted design specifications for integral abutment bridges do not exist, and (3) there has been very little verification of the behavior of integral abutment bridges or direct evaluation of the validity of design assumptions through field monitoring (Bonczar et al. 2005). Consequently, state transportation agencies rely solely on their past experiences and refinement when constructing integral bridges. This implies a number of limitations on their use; the most notable include bridge length, skew, curvature, foundation types, bridge sites, and provision for approach slabs at both ends of the bridge.

This paper examines the relation between depth of predrilled holes and abutment pile bending stresses using a number of parameters; soil profile, skew, number of spans, and bridge length for the case of a skewed steel integral abutment bridge. In addition, the paper investigates the effects of predrilled holes on pile deflections and on the axial load carrying capacity of piles. End bearing steel H piles oriented with their weak axis perpendicular to the centerline of the bridge are used.

2. Integral Abutment Bridges

Integral abutment bridges are girder bridges with no expansion joints in the bridge deck and no bearings at the abutments. The ends of girders are integral with the abutments; cast into a concrete end diaphragm, which is rigidly connected to a concrete pile cap. The pile cap is supported by a single row of vertical piles (Fig. 1). Pile flexibility accommodates thermal expansion and contraction of the superstructure.

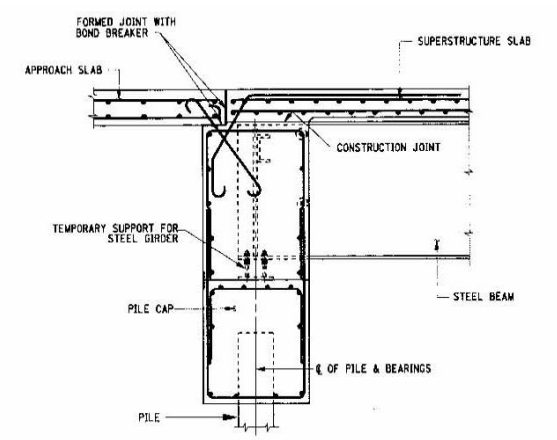


Fig. 1. New York State DOT steel superstructure integral abutment detail

Integral abutment bridges differ from conventional bridges and rigid frame bridges in the manner superstructure movement is accommodated. The superstructure movement is due to temperature changes, creep, and shrinkage and is primarily horizontal translation. An integral abutment bridge accommodates superstructure movement by flexure of the piling and by provision of cycle-control (expansion) joints at the roadway end of the approach slabs. Conventional bridges accommodate superstructure movement by means of deck expansion joints combined with fixed and expansion bearings. Rigid frame bridges accommodate the effects of temperature change, creep, and shrinkage with full height abutment walls that are fixed or pinned at the footing level.

Use of integral abutment bridges offers numerous advantages over conventional bridges. This includes lower construction and maintenance costs (Amde, A.M. and Klinger, J.E. 1988), faster construction, enhanced protection for weathering steel girders (Wasserman and Walker 1996), reduction in the magnitude of impact loads that leads to improved vehicular riding quality (Mistry 2005), enhanced seismic performance (Greimann et al. 1987; Hoppe and Gomez 1996), additional live load capacity to resist potentially damaging overloads (Wasserman and Walker 1996), and lower end span to interior span ratio (Harvey and Kennedy 2002).

Despite their advantages over conventional bridges, there are limitations on the use of integral abutments. The reason for those limitations is to reduce the magnitude of passive earth pressures behind the integral abutments and minimize stresses in the integral abutment piles. The list of limitations aim to reduce the magnitude of passive earth pressures behind the abutments include (1) bridge length to control the amount of backfill compression and consequently the magnitude of passive pressures, (2) abutment height to reduce the magnitude of passive pressures, (3) use of well-graded, free-draining backfill material to control the magnitude of passive pressures, (4) use of approach slab to prevent backfill compaction by the vehicular traffic, and (5) use of moderate skew to shorten the length of abutments exposed to passive pressures. The list of limitations aim to minimize pile stresses include (1) use a single row of vertical slender piles to minimize resistance to longitudinal deck movements, (2) use of predrilled holes around piles in stiff soils, and (3) use of only slight curvature. In addition, there are limitations related to the suitability of the bridge site; this includes adequate depth to bedrock as well as potential for liquefaction and scouring. Furthermore, successful implementation of integral abutment bridges requires attention to construction sequence and good guidance to the contractor in the contract documents (Harvey and Kennedy 2002).

3. Predrilled Holes Around Piles Supporting Integral Abutments

For stiff soil conditions, predrilling oversize holes and surrounding the piles with loose granular soil (Fig. 2) has emerged as an effective method to increase pile flexibility (Dunker and Liu 2007), reduce bending stresses in the piles and increase their vertical load capacity (Yang et al. 1985; Greimann et al. 1986; Greimann and Amde 1988; Crovo 1998; Faraji 1997; Khodair and Hassiotis 2003).

Yang et al. (1985) demonstrated that predrilling holes to replace stiff soils with loose sand greatly increases the vertical load carrying capacity of piles. The predrilled length of the holes was a significant factor. Using HP 250x62 steel piles, 1.8-3 m of length was necessary to take full advantage of predrilling. Mourad and Tabsh (1998) report that the predrilled holes need to be 3-6 m deep, measured from the pile head. Crovo (1998) and Wasserman (2001) report that the depth of the prebored holes should be at least 2.5 m while Mistry (2005) recommends the use of 3 m deep predrilled holes. Table 1 summarizes state DOT practices governing the use of predrilled holes around piles supporting integral abutments.

Table 1. State DOT practices for predrilled holes (Olson et al. 2009)

State	Comments
IA	Predrill to 8 feet for bridges over 130 feet long, and fill the hole with bentonite
IN	Predrill to 8 feet if foundation soil is hard
KS	Not reported
MA	Predrill to 8 feet and fill with loose granular material
ME	Predrill to 10 feet
MI	Predrill to 10 feet
MN	Predrill only in very compact soil to facilitate pile driving rather than to influence IAB behavior
MO	Predrill only in new fill to prevent downdrag on the piles
NE	Predrill to the engineer's recommendation
NJ	Predrill to 8 feet for bridges over 100 feet long
NY	Predrill to 8 feet and fill with loose granular material
OH	Not recommended
OR	Not recommended
SD	Predrill to 10 feet
TN	Not reported
VT	Predrill only in very compact soil
WI	Not reported
WV	Predrill to 15 feet, or predrill to bedrock if rock is between 10 and 15 feet below ground surface

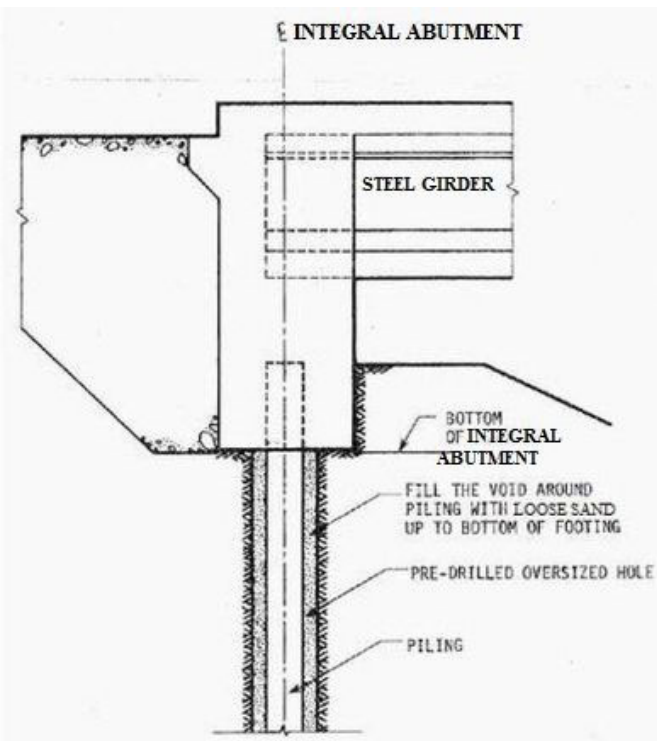


Fig. 2. Predrilled hole detail (Yang et al. 1982)

4. Pile Section Size and Length Considerations

This study is performed using HP 250X62 steel piles oriented with their weak axis perpendicular to the centerline of the bridge (Fig. 3). In practice, however, whenever the HP 250X62 piles have insufficient structural resistance, bigger HP sections are used. This includes HP 250x85, HP 310x110, HP 310x125, HP 360x152, or HP 360x174 sections. The flexibility of the pile sections, regardless of their size is ensured by making the holes twice the diameter of the pile and sufficiently deep (Dunker and Liu 2007). For HP sections, the equivalent diameter of the pile is equal to twice the length of the equivalent radius given by the expression (Fig. 4)

$$R_{equiv.} = (b + h) / \pi$$

where

$R_{equiv.}$ is the equivalent radius of the HP steel section

b is the flange width of the HP steel section

h is the depth of the HP section

π is a mathematical constant that is the ratio of a circle's circumference to its diameter and is equal to 3.14

Although steel H-piles are most frequently used to support integral abutments, cast-in-place concrete piles, prestressed concrete piles, steel pipe piles (open ended or concrete filled), drilled shafts, and spread footings are also used by states.

Use of minimum length of piling is critical for integral abutment bridges. This is due to the fact that the overall length of a pile is relevant to the pile's flexibility and its ability to accommodate abutment movement—the longer the pile, the more flexible is (GangaRao et al. 1996) and the higher is its lateral load carrying capacity (Begum and Muthukkumaran 2008). Consequently, there is a need to ensure that the piles have sufficient flexibility to accommodate the horizontal displacements of the superstructure (Mistry 2005) and that the depth of overburden provides fixed support conditions. This precludes the use of integral abutments where the depth to bedrock is considered shallow, less than 4 m from the ground surface (Hartt et al. 2006) or where piles cannot be driven

through at least 3 to 4.5 m of overburden (Burke 1993; Hoppe and Gomez 1996). Others (Vermont Agency of Transportation Integral Abutment Bridge Design Guidelines) stipulate a minimum pile embedment length of 5 m below the bottom of the pile cap. For instances where one abutment is founded directly on bedrock, but there is sufficient depth for piles to flex at the other abutment, the abutment on bedrock may simply be considered the center of the bridge and piles at the other abutment can be checked for thermal movement based on the entire length of the bridge, rather than half the length (Dunker and Abu-Hawash 2005).

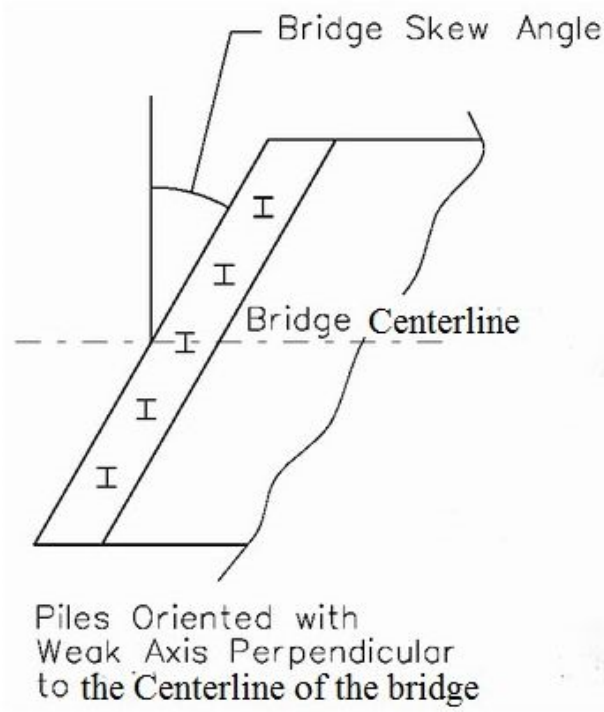


Fig. 3. Pile orientation

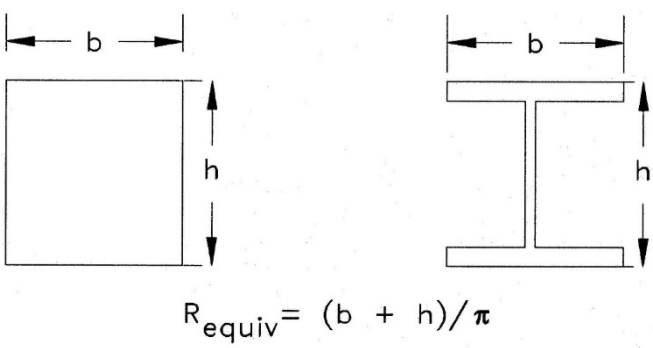


Fig. 4. Equivalent radius of HP section

5. Finite Element Model

A three-dimensional nonlinear finite element model was developed for this parametric study. The model includes the bridge superstructure and substructure as well as the soil behind and below the integral abutments. The model consists of shell elements for the deck slab, girders, and piles; solid elements for the abutments; and nonlinear spring elements for the soil. The study was conducted using the ABAQUS general-purpose nonlinear finite element analysis program. ABAQUS contains an extensive library of elements that can model any geometry. In addition, it has an extensive list of material models that can simulate the behavior of engineering materials including metals and reinforced concrete as well as geotechnical materials such as soils and rock.

The shell element type S4R5 was used to model the deck slab, girders, and piles. The S4R5 shell element is a 4-node doubly-curved thin shell that reduces integration with hourglass control using five degrees of freedom per node, 1, 2, 3 (Δx, Δy, Δz) and two in-surface rotations. The solid element C3D8 was used to model the abutments. The C3D8 element is an 8-node linear brick element with three active degrees of freedom, 1, 2, 3 (Δx, Δy, Δz). The nonlinear spring element SPRING1 was used to simulate the soil behavior. This spring element has six active degrees of freedom, 1, 2, 3, 4, 5, 6 (Δx, Δy, Δz, Θx, Θy, Θz).

The bridge superstructure (Fig. 5) is comprised of a 178 mm thick reinforced concrete slab that sits on six steel girders spaced at 1.83 m. The width of the overhangs on each side is 610 mm. The girders are integrated into the abutments at both ends of the bridge (Fig. 1 and Fig. 9). The abutments are 915 mm wide and have a height of 2.31 m

The deck slab is modeled as shell elements. Nodes are placed at the mid-depth of the reinforced concrete slab at both ends of the bridge superstructure cross section, mid-points between girders, and at the ends and midpoint of girders' top flange

The steel girders are modeled as shell elements with nodes at each end of the flanges and three nodes along the web; two of the web nodes located at the intersection of the web and the flanges. The nodes at the top of each girder are connected to the nodes in the mid-depth of the concrete deck slab using a rigid connection. In addition, the nodes for the concrete deck slab and steel girders are repeated along the bridge length; each node is repeated ten times at equal spaces along the length of each span.

Fig. 6 illustrates the mesh layout along the length of the bridge and the integration between the bridge superstructure and the abutment. It also illustrates the mesh in the abutment cross section; a three-node layer at the top of the abutment, along the slab center line; top, center, and bottom of girders; along top of pile, and at the bottom of the abutment.

The abutments are modeled as solid elements; each element has eight nodes. The nodes are along the same lines in the superstructure and each layer along the abutment cross section

has three nodes; two at the edges and one along the abutment centerline

Piles are modeled as shell elements with the same number of nodes at each layer as the steel girders, that is, seven nodes at each layer (Fig. 7). The pile itself is divided into twenty layers at equal spaces and one layer at the top of the pile. The length of the HP 250X62 steel piles is 12.5 m, where 300 mm is embedded into the abutment and the remaining 12.2 m is driven into the soil, and divided into twenty equally-spaced layers; the length of each layer is 610 mm.

Soil is modeled using nonlinear springs. There are three springs at each node along the length of the pile (Fig. 8). The vertical spring acting in the z-direction represents the friction resistance to the vertical movement of the pile. The other two springs simulate soil resistance to lateral movement in the x and y directions. All three springs are nonlinear and their force-displacement values are calculated using the modified Ramberg-Osgood model. The tip of the end bearing pile sits on rock and represents fixed end boundary conditions.

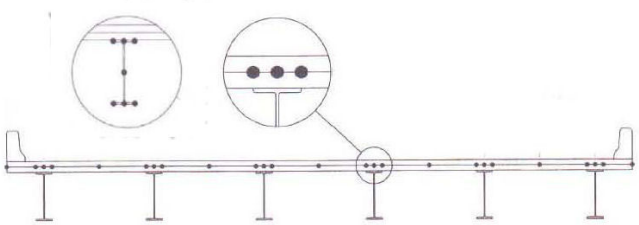


Fig. 5. Bridge superstructure model

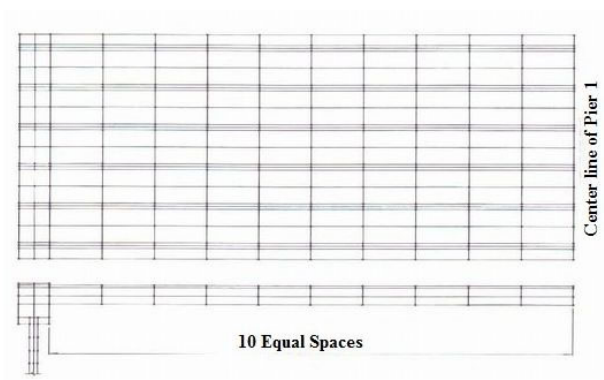


Fig. 6. Plan and elevation view of the mesh layout of the first span of the model

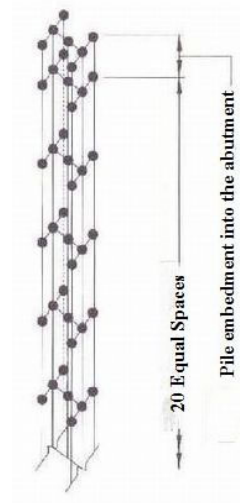


Fig. 7. Layout of nodes in the steel pile

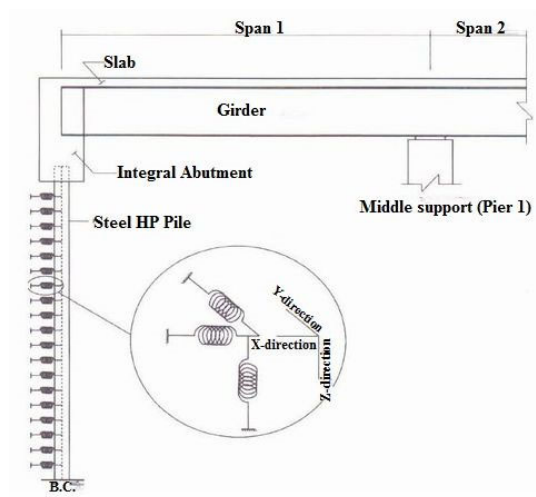
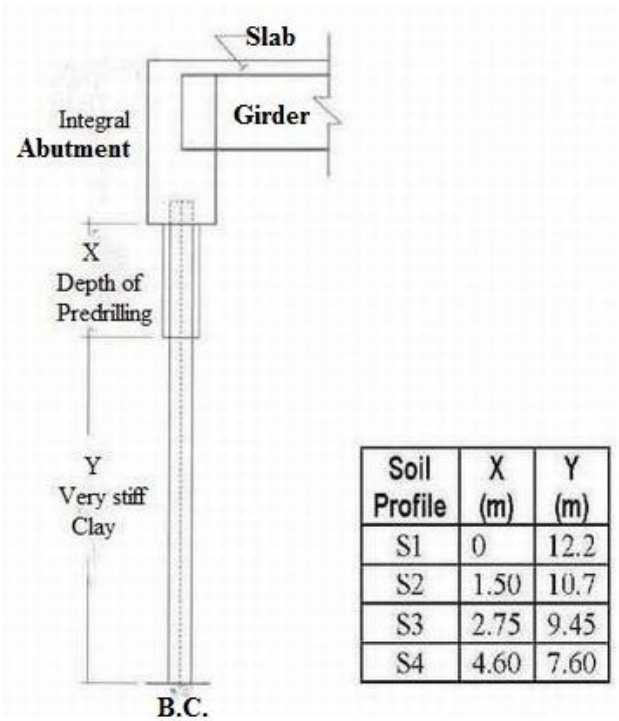


Fig. 8. Soil modeling

6. Soil Profiles

Four depths of predrilling are used in this study (Fig. 9). The first soil profile, S1, consists of only one type of soil; very stiff clay that extends from the bottom of the abutment to the pile tip. The other three soil profiles consist of two types of soil; the top layer is loose sand and represents a pile placed in a predrilled hole filled with loose sand, and the bottom layer is very stiff clay that extends to the pile tip. Fig. 9 indicates the depth of both layers for all four soil profiles S1 to S4. Table 2 describes the soil properties of both types of soil.



Soil Profile	X (m)	Y (m)
S1	0	12.2
S2	1.50	10.7
S3	2.75	9.45
S4	4.60	7.60

Fig. 9. Layout of the four soil profiles

Table 2. Soil Properties

Loose Sand (Layer X in Fig. 9)	Effective unit soil weight (submerged unit weight) γ = 8.6 KN/m^3 Angle of internal friction Φ = 30°
Very Stiff Clay (Layer Y in Fig. 9)	Effective unit soil weight (submerged unit weight) γ = 10.2 KN/m^3 Undrained cohesion of the clay soil Cu = 239.4 KPa Strain of clay at 50 percent of soil strength ε_{50} = 0.005

7. Pipe-Soil Interaction

According to Greimann et al. (1984), soil characteristics can be described by three types of soil resistance versus displacement curves. The first characteristic is represented by a p-y curve (Fig. 10), which describes the relationship between the horizontal resistance (horizontal force per unit length of pile) of the soil at a depth z along the pile length and the corresponding horizontal displacement of the pile at that depth. The second characteristic is represented by an f-z curve, which describes the relationship between the vertical skin frictional resistance (vertical force per unit length of pile) of the soil at depth z along the pile length and the relative vertical displacement between the pile and the soil at that depth. The third characteristic is represented by a q-z curve, which describes the relationship between the bearing resistance (vertical force on the effective, pile-tip area) at the pile tip and the vertical settlement of the pile tip. All three types of curves assume nonlinear soil behavior.

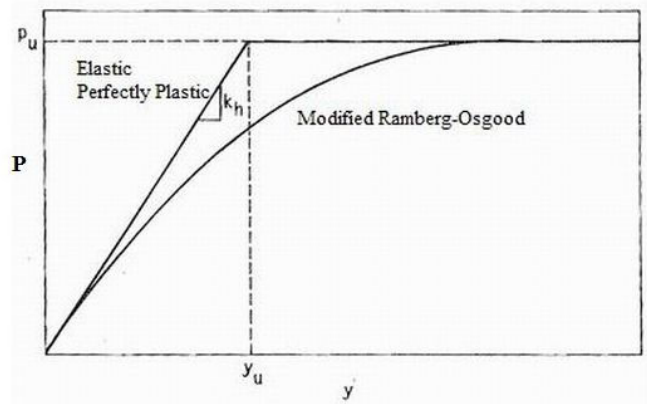


Fig. 10. Modified Ramberg-Osgood P-y curve (Amde et al. 1987)

8. Modified Ramberg Osgood Model

Research conducted by Amde et al (1982), Greimann et al (1984), Greimann and Amde (1988) utilized an idealized model based on the modified Ramberg-Osgood model to approximate the load-displacement curves for the modeling of the nonlinear pile-soil interaction. The parameters needed for the model are calculated from the soil and pile properties. The modified Ramberg-Osgood model is used to approximate all three types of load-displacement curves.

The modified Ramberg-Osgood P-y curve is expressed mathematically using the equation

$$p = \frac{k_h y}{\left[1 + \left[\frac{y}{y_u}\right]^n\right]^{1/n}}$$

$$y_u = \frac{p_u}{k_h}$$

where

k_h is the initial lateral stiffness of the soil

p is the lateral soil resistance

p_u is the ultimate lateral soil resistance at depth z along the pile length

n is a dimensionless shape parameter. The effect of the shape parameter on the modified Ramberg-Osgood equation is shown in Fig. 11

y is the lateral displacement of the pile

y_u is the lateral displacement of the pile in inches that is associated with an elastic- plastic soil material when the resistance p equals the resistance p_u

Fig. 10 presents a comparison between the modified Ramberg-Osgood curve and a typical *p-y* curve. The figure shows that the typical *p-y* curve simplifies the nonlinear soil

behavior with the use of an elastoplastic curve. This curve has two parts: (1) elastic portion, which is defined with a slope equal to the secant soil modulus for the case of clay, and initial soil modulus for the case of sand, and (2) plastic portion, which is the ultimate soil resistance per unit length of pile, p_u.

Fig. 11 presents a non-dimensional form of the modified Ramberg-Osgood *P-y* equation in terms of p/p_u versus y/y_u. The graph clearly indicates the effect of shape parameter n in the modified Ramberg-Osgood *P-y* equation.

9. Effect of Skew Angle and Depth of Predrilled Holes on Maximum Pile Bending Stresses

All four soil profiles S1 to S4 (Fig. 9) were analyzed for temperature load using a temperature variation of 90□F and five skew angles (0º, 15º, 30º, 45º, and 60º). The results of the analyses are shown in Table 3. The models used in the finite element analyses comprised of four equal spans of 30.5 meters. Piles were oriented with their weak axis perpendicular to the centerline of the bridge regardless of the skew angle. In addition, piles were assumed perfectly elastic to eliminate any possibility of plastic hinge formation.

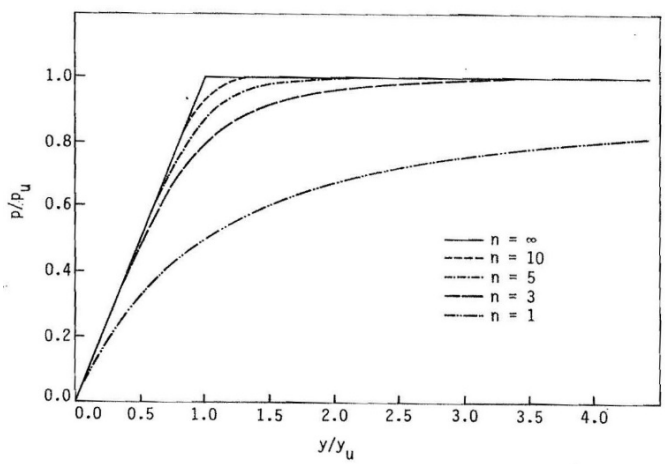


Fig. 11. Non-dimensional form of the modified Ramberg-Osgood P-y equation (Amde et al. 1987)

The results tabulated in Table 3 are plotted in Figs. 12 and 13, which indicate higher pile bending stresses with increasing skew angle (Fig. 12), but reduced pile bending stresses with increasing depth of predrilled holes (Fig. 13). The graph in Fig. 13 indicates a significant reduction in pile stresses when the depth of predrilled holes reaches 2.75 m. Further reductions in pile stresses are shown for predrilled depths exceeding 2.75 m; however, the reduction in pile stresses for depths exceeding 2.75 m is small in percentage terms compared to pile stress reductions for depths up to 2.75 meters.

Fig. 13 indicates that the skew angle impacts the effectiveness of predrilling holes. This can be explained by the fact that at larger skew angles the length of abutment exposed to passive pressures increases from width W to W/cosΦ where Φ is the skew angle.

In order to confirm that 2.75 m is an effective depth for predrilled holes in integral abutment bridges, another set of models having a 2.15 m deep predrilled holes was analyzed. The results indicate that the reduction in pile stresses due to 2.15 m deep predrilled holes exceed the reduction in pile stresses produced by soil profile S2, that is, 1.5 m deep predrilled holes, but is less than the reduction in pile stresses produced by soil profile S3, that is, 2.75 m deep predrilled holes. This is shown in Figs. 14 and 15. Fig. 14 illustrates the percent reduction of pile stresses versus depth of predrilled holes for varying skew angles and Fig. 15 illustrates the percent reduction of pile stresses versus skew angle for varying depth of predrilled holes.

Table 3. Maximum pile stresses in soil profiles S1 thru S4 for varying skew angles

Skew Angle	Pile Stresses (MPa) for Soil Profile			
	S1	S2	S3	S4
0º	398	376	301	287
15º	401	385	309	295
30º	406	392	317	302
45º	414	403	330	316
60º	424	417	359	346

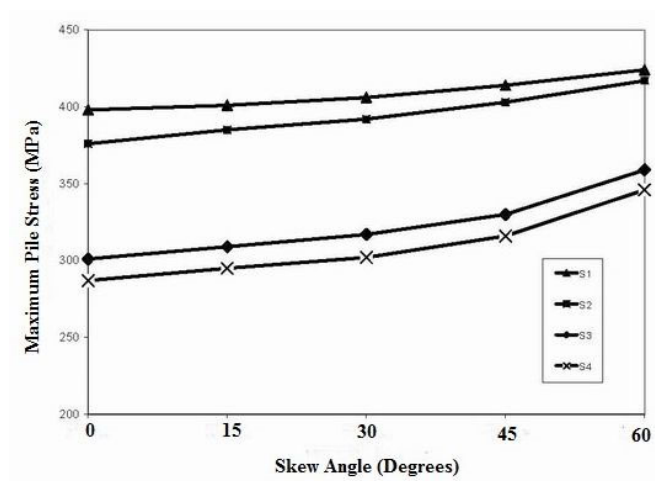


Fig. 12. Maximum pile stresses versus skew angle for different soil profiles

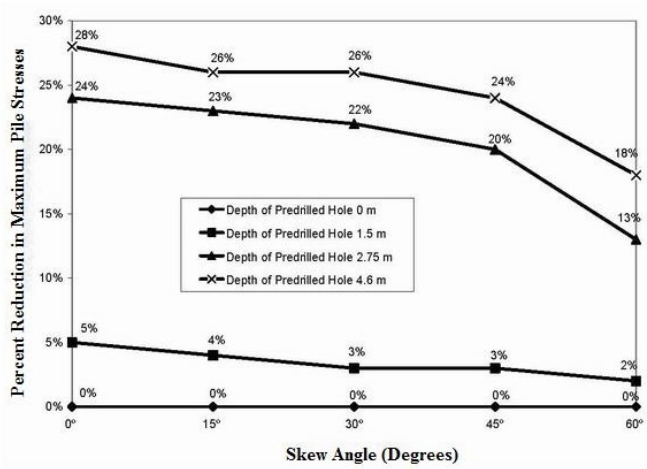


Fig. 13. Reduction in maximum pile stresses versus skew angle for varying depth of predrilled holes

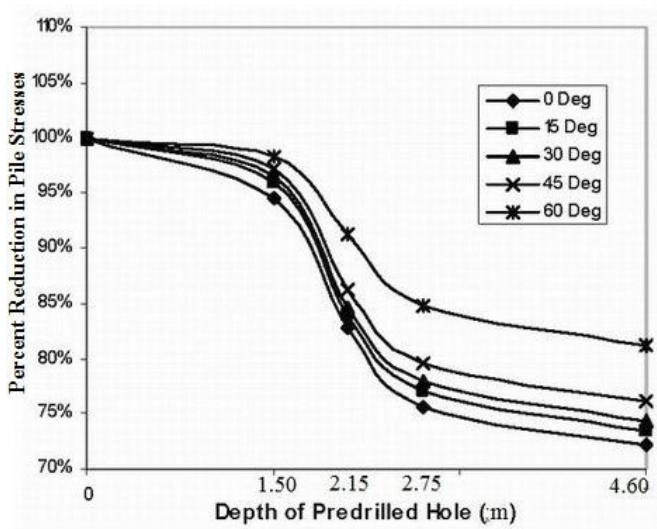


Fig. 14. Percent reduction in pile stresses versus depth of predrilled holes for varying skew angles

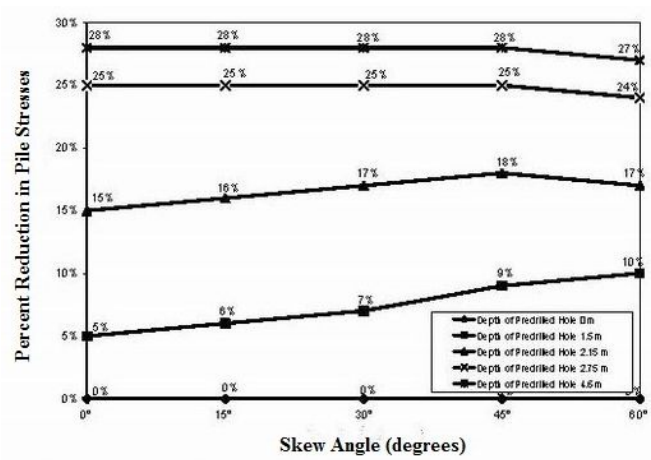


Fig. 15. Percent reduction in pile stresses versus skew angle for varying depth of predrilled holes

10. Effect of Predrilled Holes on Pile Deflections and Pile Axial Load Carrying Capacity

To study the effect of predrilled holes on pile deflections, two different soil profiles were used: (1) the very stiff clay profile (S1), and (2) the soil profile with 2.75 m deep predrilled hole filled with loose sand (S4). The deflected shapes (Fig. 16) indicated that in the very stiff clay profile (S1), the point of fixity was about 3.35 to 4 m below the bottom of the abutment, while for piles driven in very stiff clay with 2.75 m deep predrilled hole filled with loose sand at the top (S4), the point of fixity was about 4 to 4.9 m below the bottom of the abutment. The models used were for an 8-span bridge consisting of eight 30.5 m long spans, for a total bridge length of 244 m at a skew angle of 60º.

There is a significance in the fact that the depth of the point of fixity is higher in piles without predrilled holes compared to piles with predrilled holes. This is like analyzing a short cantilever column with an axial load applied at its top and subject to a lateral movement compared to analyzing a longer column subject to the same loading conditions as the short one. The short column will fail under a lower axial force than the long column for the same amount of lateral deflection. This is shown in Fig. 17, which indicates that piles with 2.75 m deep predrilled hole filled with loose sand (S4) have a higher vertical load carrying capacity compared to piles in very stiff clay profile (S1) for the same amount of lateral displacement.

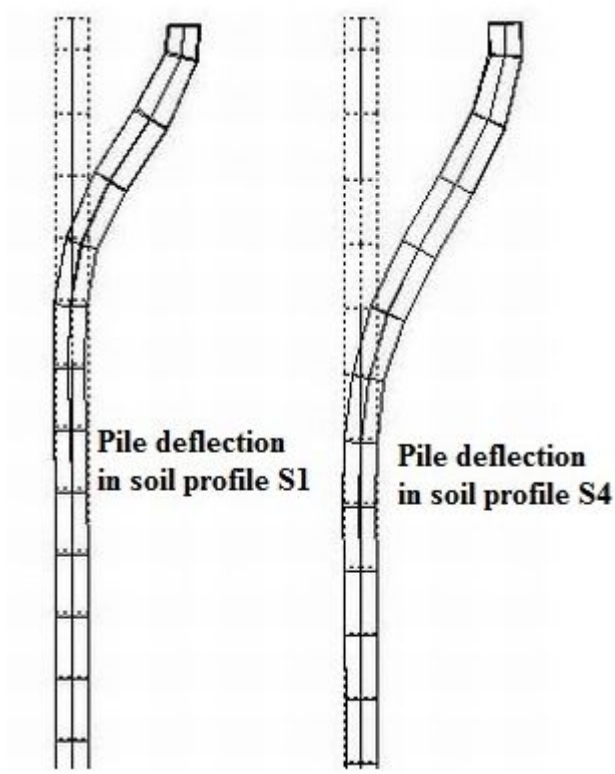


Fig. 16. Effect of predrilled holes on pile deflections

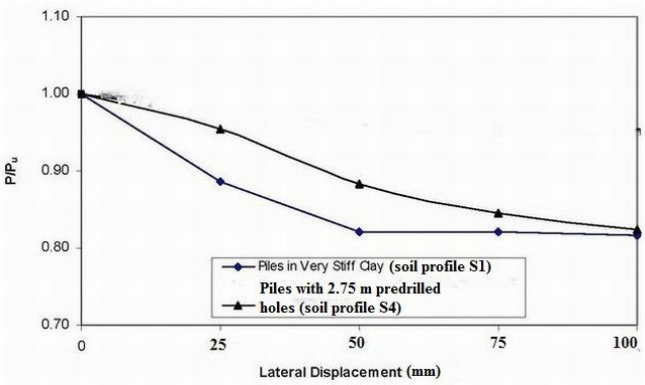


Fig. 17. Axial load carrying capacity of piles versus lateral displacement

11. Effect of Number of Spans on Maximum Pile Bending Stresses

To study the effect of number of spans on the maximum stresses in the piles, two models for 61 m long bridges were analyzed using a different number of spans. The first model consisted of two 30.5 m long spans, while the second model consisted of four 30.5 m long spans. Both models were analyzed using the three-dimensional finite element model described earlier using various skew angles. This investigation used two different soil profiles; the very stiff clay profile (S1) and the soil profile with 2.75 m deep predrilled hole filled with loose sand (S4). The results from both analyses are shown in Table 4. The results indicate a reduction in maximum pile stresses of the order of 30 percent when the number of spans increases from 2 to 4 for both soil profiles.

Table 4. Effect of number of spans on maximum pile bending stresses (MPa)

No. Spans	2	4	2	4
Predrilled Hole	No	No	Yes (2.75 m)	Yes (2.75 m)
0° Skew	343	203	233	144
15° Skew	348	209	239	147
30° Skew	354	217	244	153
45° Skew	368	230	258	163
60° Skew	396	247	287	181

12. Effect of Bridge Length on Maximum Pile Bending Stresses

Four different models were used to investigate the effect of bridge length on maximum piles stresses. The models had different bridge lengths; 61 m, 122 m, 244 m, and 366 m. Each model was analyzed using five different skew angles, that is, 0, 15, 30, 45, and 60 degrees in two soil profiles; the very stiff clay profile (S1) and the very stiff clay with 2.75 m deep predrilled hole filled with loose sand (S4). The results, which are shown in Figs. 18 and 19 show the pile stresses as a percentage of the pile stresses in the piles of the 366 m long bridges at 60-degree skew.

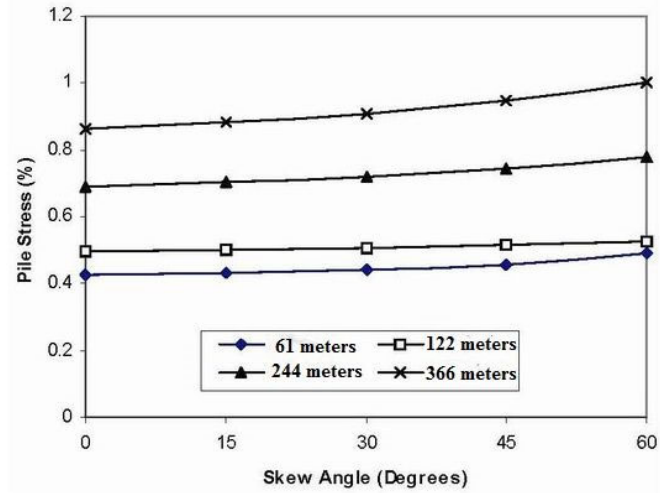


Fig. 18. Effect of bridge length on maximum pile stresses in soil profile S1

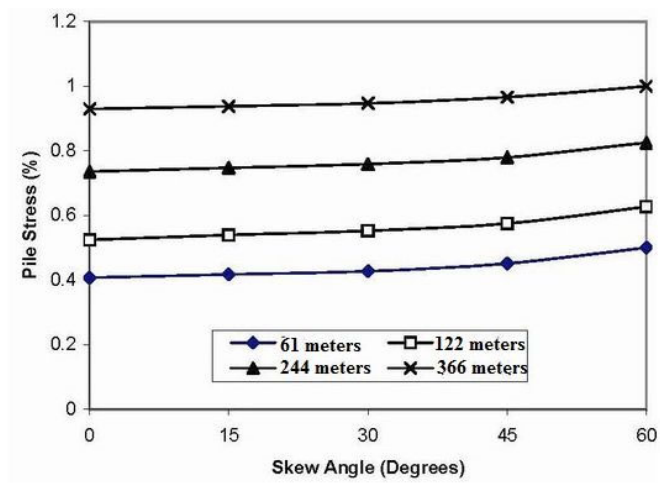


Fig. 19. Effect of bridge length on maximum pile stresses in soil profile S4

13. Conclusions

The results of this study point out to a number of observations related to the behavior of skew integral abutment bridges. This includes the following: (1) increased bridge length and skew angle induces higher bending stresses in the piles supporting the integral abutments, (2) increased number of spans and use of predrilled holes around the piles of integral abutments results in reduced pile bending stresses, and (3) use of predrilled holes increases the vertical pile load carrying capacity.

Consequently, use of predrilled holes around piles provides the benefits of reduced pile bending stresses and increased pile vertical load carrying capacity. The results of the analysis also indicate that predrilling to a depth of 2.75 m from the bottom of

the integral abutment provides almost 100 percent of those benefits. In fact, the results indicate that predrilling to a depth of more than 2.75 m yields marginal additional benefits in terms of reduced pile bending stresses and higher vertical capacity in the piles. This is attributed to the fact that most of the horizontal deflections and the largest bending moments occur within the top 2.75 to 3 m of pile length. Consequently, the surrounding soil in this region controls the behavior of the pile, regardless of the type of soil present below this depth.

In conclusion, the results of the analysis suggest that the use of predrilled holes filled with loose sand around the piles of skew integral abutment bridges is an effective method to reduce bending stresses in the piles and increase their vertical load carrying capacity. The depth of predrilled holes is 2.75 m measured from the bottom of the integral abutment.

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ISSN: 0886-9685

Using Calcium Stearate as a Modifier for fly ash stabilization

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Received 09/08/2020
Accepted for publication 11/18/2020
Published 11/21/2020

Abstract

Heavy metal leaching from improperly disposed fly ash remains a concern to human health and the environment as a result of their toxicity. The objective of this experiment is to evaluate the performance of calcium stearate to encapsulate heavy metals in fly ash to prevent chemical leaching. To assess the ability of the additive to solidify heavy metals, batch leaching tests and column leaching tests were performed. The methodology involved testing fly ash with various amounts of additive. The fly ash with the additive was evaluated by measuring the concentration of heavy metals such as Chromium, Lead and Cadmium that leached from the fly ash. Batch leaching results indicated that the concentration of chromium was 65 $\frac{\mu g}{L}$ for fly ash alone. However, with 1% calcium stearate the concentration dropped to 45.5$\frac{\mu g}{L}$. Column leaching results revealed that the concentration of cadmium was 166 $\frac{\mu g}{L}$ for 3% calcium sterate. On the other hand, with 1% calcium sterate the concentration fell to 93.4$\frac{\mu g}{L}$. Further testing revealed that 1% calcium stearate reduced the amount of heavy metal leaching from fly ash. It is hypothesized that the calcium stearate attacks the transition metals rendering the metal essentially inactive due to the free lime hydration.

Keywords: **Heavy metal leaching, Fly ash, Water-repellant**

1. Introduction

Fly ash is a common by-product of the coal combustion process that has been studied for reuse in the construction industry for many years. Yao et al. (2014) explain that the world generates approximately 750 million tons of coal-based fly ash each year. Coal fly ash is hazardous both to human health and the environment if disposed of improperly. Due to the pozzolanic properties of fly ash which increases the durability, workability, and strength of concrete products, the global demand for increased usage of fly ash has significantly increased in growing construction industries. However, there is a tremendous difference between the amount of fly ash produced globally and the amount disposed of properly through recycling and other means.

According to the Environmental Protection Agency, (EPA 2015) in the United States alone, over 110 million tons of Coal Combustion Residuals (CCRs) have been generated. However, there are less than 25% of the total quantity of fly ash produced in the world has been recycled (Zentar et al., 2012). Given the realities that up to 75% of fly ash are disposed of by other means such as placed in embankments, landfills, and ash ponds, there is a high probability of heavy metal leaching into groundwater. The issue is the presence of trace metals in the fly ash which leach out of the combined materials and cause detrimental effects to the environment as well as humans. Hence, the removal or solidification of these metals is of utmost importance attempting to recycle fly ash and creating a sustainable final product. Water dissolution, phosphating and adding sodium carbonate have been reviewed and tested as possible methods to remove these metals, but not all elements have become effectively stabilized (Aubert et al., 2006). In terms of solidification, most solutions lead to mixing the by-product with cement, which has been proven effective but fails to control the leaching of various metals. Yao et al. (2014) states currently, 20% of coal-fired fly ash is used in concrete production with it also being used in road base construction, and

soil amendment. More recently, a form of organic modification that uses organosilanes to irreversibly bond with fly ash has been introduced as water-repellant technology (Daniels et al., 2009). With this idea in mind, CALSAN™ 50, a product made by BASF which is a calcium stearate dispersion in water was believed to be a suitable replacement (BASF 2009). The objectives of this paper are to (1) review previous work regarding the use of waterproofing substances as an option for the solidification of fly ash, (2) evaluate the performance of calcium stearate through chemical leaching testing in various conditions and (3) provide commentary on whether this is a viable option for attenuation of fly ash metals.

2. Background

Many methods have been tested to explore the possibility of recycling fly ash in order to reduce the volume of waste while providing a comparable and safe alternative to landfills. Among these methods are the inclusion of slag, alkali-activation, metakaolin, Portland cement and nano-silica (Rashad, 2014). For this experiment, the process of encapsulation will be explored as an option to mitigate leaching levels to EPA regulatory standards.

In order to begin comprehending how to prevent fly ash from leaching into the ground, a basic understanding of what fly ash is must be acquired. As stated previously, fly ash is a common by-product of the coal combustion process. Depending on how the coal is processed, a wide range of properties could be seen when analyzing different ashes. Cho et al. (2005) state coal fly ash is extremely varied in its mineral makeup, and its characteristics depend heavily on the kind of coal used and the way in which it was burned as well as several other factors. On average, fly ash occurs as fine particles with a spherical shape and an average size of fewer than twenty micrometers, has low to medium bulk density and a high surface area for its small size. The pH values range from 1.2 to 12.5, but the trend is usually towards alkalinity.

The common heavy metals found in fly ash include Chromium, Lead, Cadmium, Nickel, Barium, Strontium, Vanadium and Zinc (Yao et al., 2015). Since there are hazardous metals in fly ash a method must be created in order to encapsulate the contaminants eliminating the chance for leaching to occur.

There have been many tests performed in order to determine the best method of containing the toxic metals that are present in fly ash. The leaching of toxic metals from fly ash is the primary inhibitor to the widespread use of fly ash in construction projects. Van der Sloot speaks how leaching is a function of the surface exposed to the leaching fluid. The ratio of the particle surface area to the volume occupied by the particles, the average particle size, and internal pore structures in the material all control the surface area where dissolution from the solid to the liquid can occur. Smaller particle sizes produce larger surface area, allowing for increased contact between the solid material and the leaching fluid, resulting in increased contact between leaching fluid and leachable constituents (Van der Sloot, et al. 1997).

Kerkhoff shows that fly ash can be used as liming material on acid soils or acid mine soils or alkali soils for improving the pH of the soils depending on the nature of soil and ash. Increases in pH induced by alkaline fly ash addition is a desirable property and could be used for detoxifying elements like Cd, Al, and Mn. Due to the fine nature of fly ash, it improves the WHC of sandy soils removing the compaction of clay soils. It improves the physical and chemical properties of soil as well as the biological properties of problematic soils. Application of fly ash, particularly unweathered ones, shows a tendency of accumulating elements like B, Mo, Se, and Al, whose toxic levels are responsible for a bad influence on animal and human health, (Kerkhoff 2001).

Cinquepalmi et al. investigated the use of fly ash as an artificial aggregate in Portland cement mortars and found that the release of each metal increased with an increasing amount of artificial aggregate in the mortar specimens. Also, it was found that pH has a great influence on the leachability of heavy metals from the cement mixture. The greatest effect was shown at pH levels below 6-8 and as the pH rose, the release of metals decreased to a point where the amount of artificial aggregate in the mixture had nearly no effect at all (Cinquepalmi et al., 2008).

The ionic strength of the leaching fluid also influences solubility and leaching behavior. Ionic strength is the relationship between the concentration of ions in solution and the charges of those ions. Ionic strength impacts reaction rates as well as the solubility of ionic species, with solubility, generally increasing as ionic strength increases, (Lowenbach, 1978).

Becker, et al., (2013) determined that after several experiments using the Environmental Protection Agency's testing guidelines for fly ash leaching, fly ash leached toxic metals at levels exceeding the regulatory limits. It was noted that pH levels can directly affect the rate and amount of leached metals, and each batch test conducted should be adjusted to accurately reflect site-specific conditions. There is typically a non-linear relationship between leachate metal concentrations and fly ash content, thus requiring full testing be done to better estimate the amount of leached material. Additionally, column leach testing was found to not accurately represent long term leaching risks, and therefore should not be conducted with that purpose in mind.

Singh, et al., compared the chloride resistivity of nanoparticles to fly ash and concluded that in general, fly ash does not prevent chloride permeability as well as silica nanoparticles. A silica nanoparticle content of 3% was shown to improve chloride ion resistance up to nearly 40% when compared to plain cement. Adding fly ash alone reduces the strength of the concrete compared to not adding any, but adding the fly ash and a small amount of silica increases the compressive strength of the concrete. Park et al, (2006) found that adding small amounts of silica, 4% by weight, increased strength in concrete bricks up to nearly 20.8MPa. Commercial

bricks made with natural sands had compressive strengths of around 11.9MPa. Most notably, it was found that the silica helped reduce the leaching of the hazardous substances from the solidified ash.

"It was observed that the efficiency of nanoparticles such as nano-silicon oxide depends on their morphology and genesis, as well as on the application of superplasticizer and additional treatment options such as thermal treatment and ultra-sonification. The method studied here is capable of manufacturing a wide range of nanoparticles with engineered parameters such as particle size, porosity and surface conditions. It was demonstrated that all synthesized nano-silicon oxides improve the early compressive strength of Portland cement mortars, but at later stages of hardening, strength was adversely affected by these additives. The major problem of nano-silicon oxide application and such strength loss is related to the agglomeration of nanoparticles at the final drying stage. High-temperature treatment affects the performance of these additives and should be avoided. Further research is required to modify the sol-gel method in order to avoid the formation of agglomerates and to achieve better dispersion of developed nano silicon oxides" (Drexler 1991).

(Malhotra 1988) explains that "Concrete containing Class F fly ash exhibited higher long-term resistance to chloride-ion penetration compared to Class C fly ash concrete. The best long-term performance was recorded for both of the 53 percent and the 67 percent Class F fly ash and 70 percent of Class C fly ash concrete mixtures as they were found to be relatively impermeable to chloride ions in accordance with ASTM C 1202. Except for control mixture C-4, the differences in the Coulomb values of the high-volume fly ash mixtures are not significant. All fly ash concrete mixtures showed excelling performance with respect to chloride-ion penetration resistance."

Al-Saadoun, et al., (1993) determined that fly ash blends of cement typically outperform plain cement in corrosion resistance of reinforcing steel. Fly ash blends significantly affected the initiation time for corrosion and in some cases tripled the time required for corrosion to begin. Notably, lignite fly ash performed best when compared to bituminous and sub-bituminous fly ash. Since the corrosion of steel is an electrochemical process, it was shown that having a 25% fly ash mixture can increase the electrical resistivity of cement threefold.

Fly ash has a tendency of not acting as a pozzolanic material until at least one week after being mixed. Fraay, et al., (1989) determined that fly ash created as a byproduct of bituminous coal (class F) contains crystalline inclusions which are primarily comprised of alumina-silica glass. This crystalline structure is the cause of the delayed hydration when mixed with Portland cement. The pozzolanic reaction is explained by the dependency of breaking down the glass on the alkalinity of the pore water. High alkalinity in conjunction with the precipitation of reaction by-products can inhibit the breaking down phase, ultimately delaying the pozzolanic reaction from the fly ash. Additionally, the pH and temperature of the pore water can affect the reactivity of the fly ash as well as the solubility of the glass. In most cases, lower pH levels create less reactive fly ash which, in turn, delays the pozzolanic state from occurring. Cao., (2010) determined that by influencing the micro-gradation of cement the delayed pozzolanic reaction of fly ash could be counteracted and early strength could be improved. Since fly ash slows the early strength generated by concrete, slag was incorporated into the design to improve early strength. Since slag on its own can produce harmful air voids, the fly ash will be used to counteract this by filling the air voids. Cao found that the optimal fly ash to slag ratio was 4:1 and performed best when an activator was used. When the activator content was at 2%, the day one compressive strength and flexural strength was approximately 21% and 10% higher, respectively, when compared to concrete with just fly ash. These numbers rose when the activator was increased to 3%. The activator significantly speeds up the hydration process and increases early strength in cement with fly ash and slag additives. This same method could potentially be used in speeding up the hydration of fly ash cement mixes to help move the pozzolanic reaction along.

Rashad reviewed many experiments that tested various admixtures to fly ash and the effect that occurred. One experiment studied the compressive strength with various amounts of fly ash added to Portland cement. From the experiment, it was found that a ratio of 60 percent fly ash to 40 percent of Portland cement yielded the best results for mechanical performance. This is good to keep in mind because for the proposed experiment a comparable product must be made in terms of strength and not just reduced leachability (Rashad, 2014).

Most of these procedures were completed using one or more of the following testing methods: batch, column or block. For the purposes of this paper, it was found sufficient that batch testing and column testing be performed.

"Batch extraction tests typically involve mixing a sample of waste or other fill material with a specific amount of leaching solution without renewal of the leaching solution. The mixing is performed over a short period with the aim of trying to reach equilibrium with some type of rotary agitation. The mixing is followed by filtration and analysis of the filtered liquid phase" (Washington State Department of Ecology 2003).

"A column leaching procedure is used to classify waste as hazardous or not and to determine the effectiveness of a waste treatment process. A column leaching procedure using a leaching fluid being pumped into a sample material in a column. One should then use a tube coming out of the sample column and into an effluent to sample the extracts" (Washington State Department of Ecology 2003).

By using calcium stearate technology, an effective way of encapsulating heavy metals within a cement matrix is expected.

3. MATERIALS AND METHODS

The fly ash studied was donated from Headwaters Resources and came from Brayton Point. Headwaters Resources lists the material density at 2.38 grams per centimeter cubed.

The calcium stearate (Calsan™ 50) was procured from BASF Corporation in New Jersey. It is a part of the metal stearate family and is used primarily as a coating lubricant.

The columns used for the column leaching tests had dimensions of 4.5 inches in diameter by nine inches in height for a total volume of 143.14 cubic inches. They were connected to a gravity head system to conduct the column tests.

A spectrophotometer was used to determine the amount of light that a sample of the leachate could absorb. The intensity of light that reaches the detector would then be used to measure the quantity of heavy metals that have leached into solution.

The batch leaching was performed using varying fly ash and calcium stearate mixtures and adding the leachate of nitric acid and distilled water at a pH of 4.2 to 4.4. The pH was set at this level to mimic the worst-case scenario and cause the most leaching to occur from the fly ash mixtures. The amount of leachate was kept at a constant 50 mL and the liquid to solid ratio was altered using various amounts of fly ash and calcium stearate mixtures. After creating ratios of 100:1, 50:1 and 10:1, each with a different quantity of calcium stearate (0%, 1%, 2% and 3% by weight), the mixtures were shaken for 48 hours. Once the shaking was complete, the leachate was extracted from the surface with pipettes. Ten mL of the solution was mixed with cadmium, chromium, and lead reagents in a test tube and shaken for three minutes each to ensure a complete mixture. Each test tube was then placed in the spectrophotometer to measure the amount of heavy metal leaching that occurred. This process was repeated twice for duplicate testing.

The column leaching test was performed by first mixing the fly ash and calcium stearate mixtures with a specified amount of water to achieve optimum moisture content and maximum dry density. Once this was complete, the columns were then filled with each mixture in five equal lifts, compacting each with approximately 25 drops of a compaction hammer. The columns were then attached to a gravity head system as seen in Figure 1, with one inlet tube feeding leachate from a holding tank and one outlet tube to allow for the collection of the passed leachate. Based on the pore volume that was calculated to be one liter (Source needed), it was determined that each time one liter of solution passed through the column, it would be tested for the selected heavy metals. The data would then be input into the Yalcin Leaching Model in order to predict the leaching behavior for an extended period of time. Each pore volume was tested twice to provide duplicate results.

"A Yalcin leaching model was formulated to capture the observed experimental leaching behavior of the contaminant exhibiting an initial increase in concentration followed by a decrease in concentration with further leaching until it reaches a low steady-state concentration. The model is as follows.

$$C(t)=C_s - C_s \cdot e^{-Kk_bt} + C_0 \cdot e^{-Kk_bt}$$

Where k_b is the dissolution rate coefficient (min^{-1}), $K=(S/S_0)^a$, S=Solid phase concentration (mg/g), S_0=Initial solid phase concentration (mg/g), C_s is the effective saturation concentration (solubility) of contaminant (mg/L), t is the time (min), and a is a dimensionless empirical constant" (Das 2007).

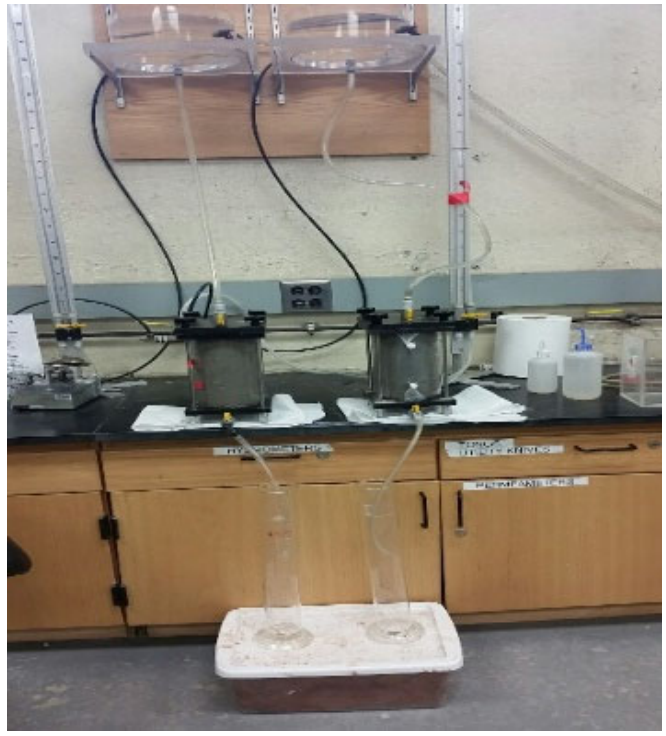

Figure 1. Gravity Head Column Leaching

4. RESULTS AND DISCUSSION

Tables 1 through 3 show the results of the batch leaching tests that were conducted on each L:S ratio containing 0-3% calcium stearate. For each metal, a general trend could be seen that the 1% calcium stearate mixture produces a constant value or a decrease in heavy metal concentration when compared to the plain fly ash solution, most notably for chromium. The same results could be seen in the graphical representation in Figures 2 through 4. The same general trend could be seen in this instance as well.

Table 1. Batch Leaching Average Concentration of Cadmium

Cadmium		
L:S Ratio	Calcium Stearate (% by weight)	Average Concentration (mg/L)
100:1	0	0.069
	1	0.072
	2	0.0697
	3	0.0655
50:1	0	0.0635
	1	0.068
	2	0.0715
	3	0.0815
10:1	0	0.0805
	1	0.0673
	2	0.0805
	3	0.1425

Table 2. Batch Leaching Average Concentration of Chromium

Chromium		
L:S Ratio	Calcium Stearate (% by weight)	Average Concentration (mg/L)
100:1	0	0.065
	1	0.0455
	2	0.0475
	3	0.0495
50:1	0	0.0685
	1	0.081
	2	0.0585
	3	0.051
10:1	0	0.119
	1	0.0495
	2	0.0835
	3	0.0925

Table 3. Batch Leaching Average Concentration of Lead

Lead		
L:S Ratio	Calcium Stearate (% by weight)	Average Concentration (mg/L)
100:1	0	0.1495
	1	0.155
	2	0.166
	3	0.155
50:1	0	0.1775
	1	0.1555
	2	0.1765
	3	0.181
10:1	0	0.135
	1	0.1545
	2	0.190
	3	0.3275

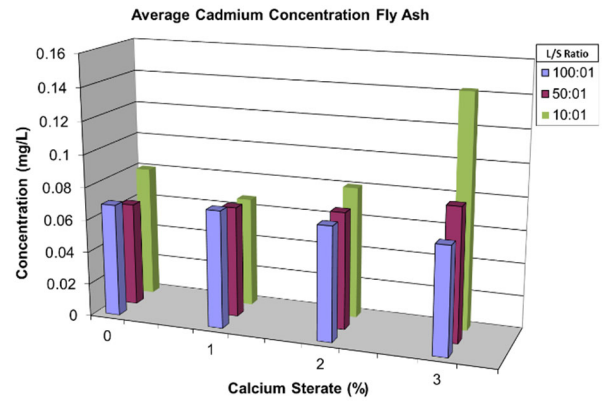

Figure 2. Batch Leaching Average Concentration of Cadmium

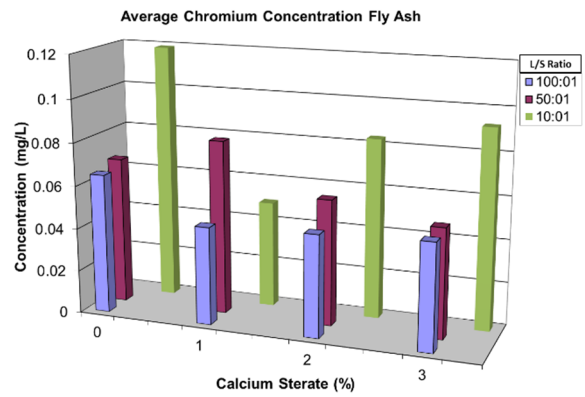

Figure 3 Batch Leaching Average Concentration of Chromium

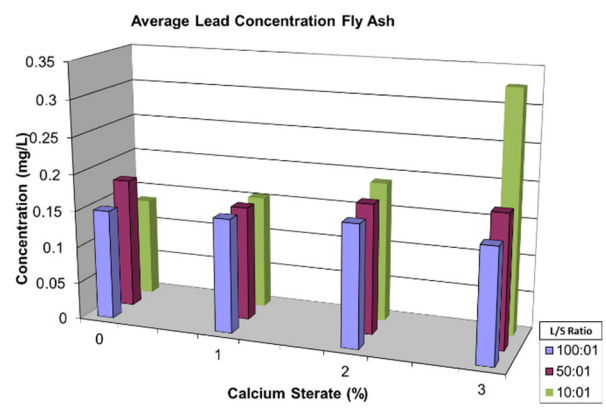

Figure 4. Batch Leaching Average Concentration of Lead

Table 4 shows the results of the column leaching tests based on the average values from each pore volume collected. These values vary widely due to the lack of information for the succeeding pore volumes which could not be collected due to time restraints. From the information gathered, as seen in Figures 5-16, it has been shown that the data collected will fit the predicted concentrations, but more data would be needed to create more accurate results.

Table 4. Column Leaching Average Heavy Metal Concentrations

Column Leaching Data			
Heavy Metal	Calcium Stearate (% by weight)	Average Concentration PV1 (µg/L)	Average Concentration PV2 (µg/L)
Chromium	0	197	259.5
	1	540.5	143.5
	2	478.5	144
	3	514.5	153
Cadmium	0	78.4	60.1
	1	93.4	63.25
	2	131	133.8
	3	166	137.8
Lead	0	189	138.5
	1	222	141
	2	309	310
	3	413	307

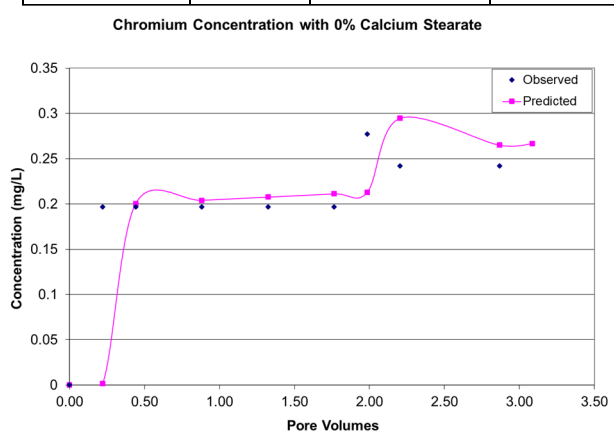

Figure 5 - Yalcin Leaching Model (0% C.S. - Chromium)

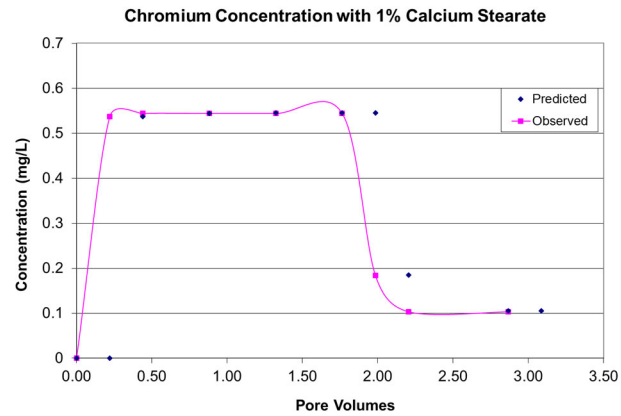

Figure 6 - Yalcin Leaching Model (1% C.S. - Chromium)

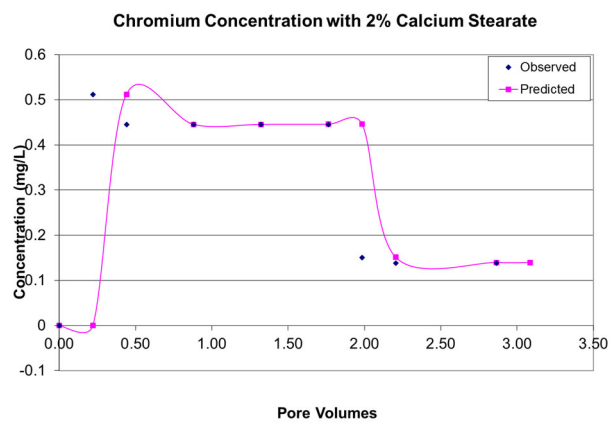

Figure 7 - Yalcin Leaching Model (2% C.S. - Chromium)

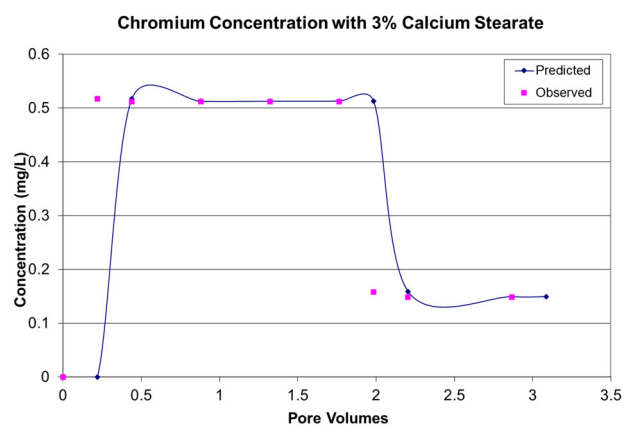

Figure 8 - Yalcin Leaching Model (3% C.S. - Chromium)

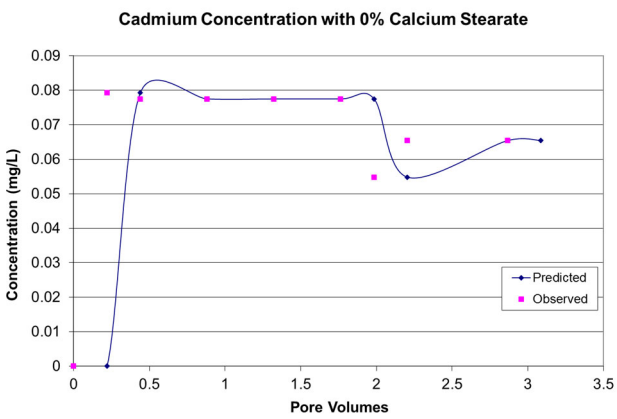

Figure 9 - Yalcin Leaching Model (0% C.S. - Cadmium)

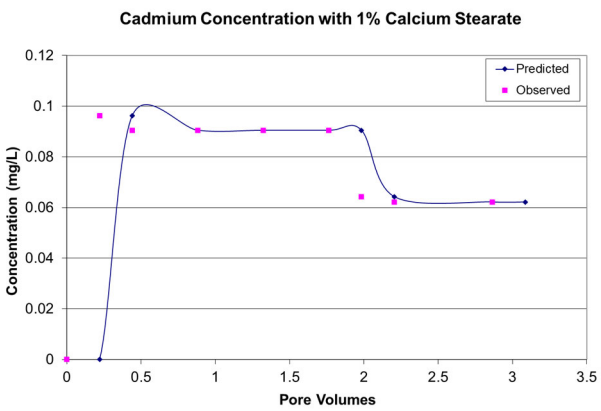

Figure 10 - Yalcin Leaching Model (1% C.S. - Cadmium)

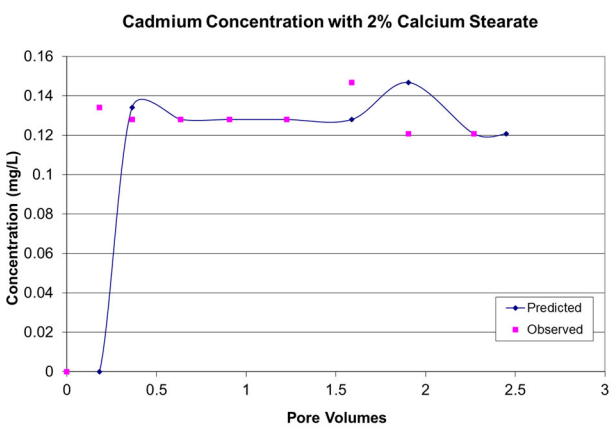

Figure 11 - Yalcin Leaching Model (2% C.S. - Cadmium)

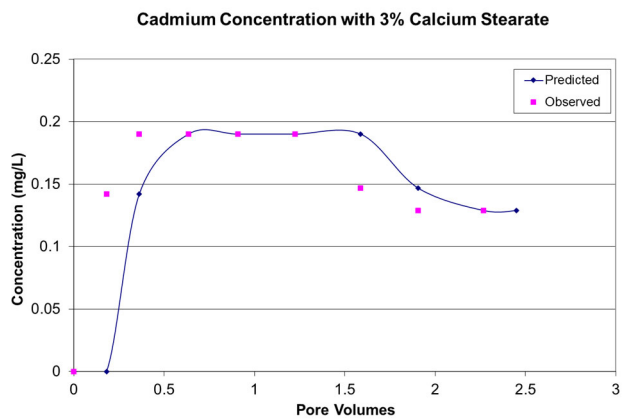

Figure 12 - Yalcin Leaching Model (3% C.S. - Cadmium)

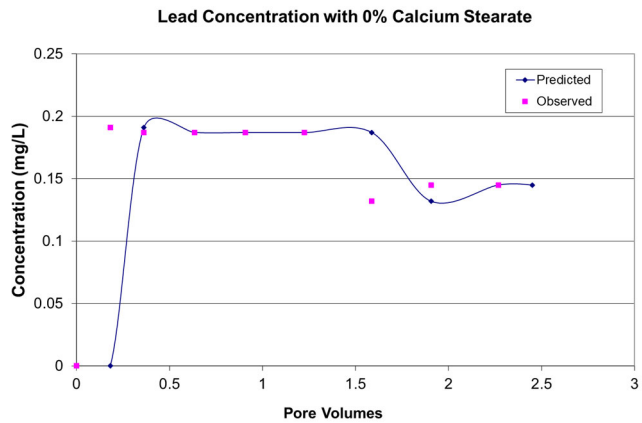

Figure 13 - Yalcin Leaching Model (0% C.S. - Lead)

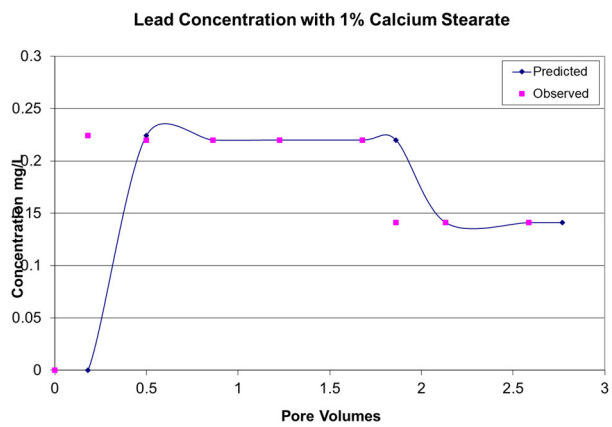

Figure 14 - Yalcin Leaching Model (1% C.S. - Lead)

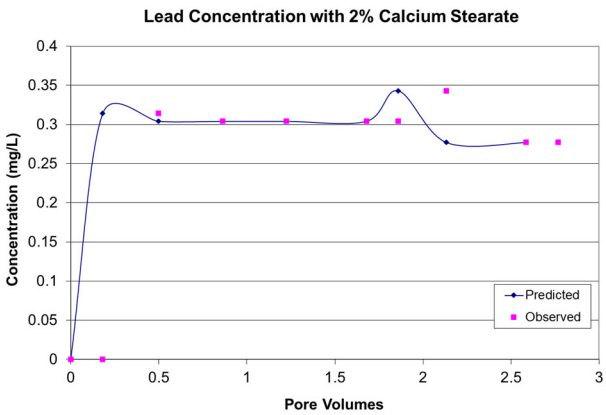

Figure 15 - Yalcin Leaching Model (2% C.S. - Lead)

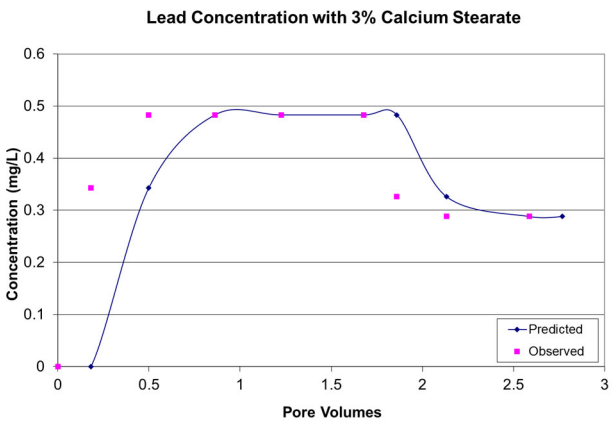

Figure 16 - Yalcin Leaching Model (3% C.S. - Lead)

After final review of all the compiled data, the batch leaching and column leaching data generally indicate that by adding 1% calcium stearate to fly ash, the overall concentration of leached heavy metals, Chromium, Cadmium, and Lead, was reduced by the greatest amount when compared to the 0%, 2%, and 3% mixtures. At higher concentrations of calcium stearate, a trend of increasing concentrations of leached metals was observed in both testing methods.

The Yalcin leaching model was used as a numerical modeling method rather than an analytical method due to time constraints and the limited pore volumes that we're able to be obtained. This does not discredit the data obtained, instead, it shows that the recorded results follow a predicted trend. This trend could become more accurately depicted if subsequent pore volumes were collected and tested allowing for precise analytical modeling to predict leaching behavior for months or even years in the future.

Since calcium stearate is a molecule consisting of a central calcium ion with two stearate groups attached, it could be considered a scavenger additive. The stearate group is essentially a long-chain carbon molecule, making it similar in nature to polymer chains. Using the chain, calcium stearate will attack the transition metals, found in the form of metal chlorides by the following reaction

$$Ca(O_2C_{18}H_{37})_2 + MCl_2 \rightarrow CaCl_2 + MSt_2$$

where $Ca(O_2C_{18}H_{37})_2$ is the calcium stearate, MCl_2 is the metal chlorides (Cr, Cd, and Pb), $CaCl_2$ is calcium chloride and MSt_2 which is the new metal stearate chain (Equistar n.d.). The calcium chloride is an inactive chemical and the new metal stearate is the heavy metal that combines with the long stearate chains, effectively encapsulating the contaminant.

During chemical bonding, there is only a certain amount of bonds that could be made and electrons that could be transferred. This being the case, it was determined that the 1% calcium stearate addition provided the optimum conditions in terms of bonds available. When additional quantities of calcium stearate were added, there was an excess of electrons that were available to bond creating a poorer performance of the additive.

Due to limited data, future research to be conducted will include triplicate testing for both the column and batch leaching tests. Furthermore, numerous additional pore volumes would be collected to more accurately predict the contaminant leaching behavior using the Yalcin leaching model. Other options include testing for more heavy metals or using different leaching models such as the Van Genuchten leaching model.

5. ACKNOWLEDGMENTS

For the donation of fly ash, Stephen Berlo from Headwaters Resources is to be acknowledged.

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