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CAUSES & CURES FOR CRACKING OF CONCRETE BARRIERS

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7. Author(s) Dr. Haluk Aktan, Ph.D, P.E. and Mr. Upul Attanayaka		6. Performing Organization Code WSU
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16. Abstract

The research objective was to investigate the causes of premature deterioration of concrete bridge barriers with the goal of developing strategies for corrective actions. The synthesis of all the data collected in various tasks revealed that barrier deterioration starts with early formation of vertical cracking as a result of the tensile stress due to early-age thermal load. Volume change of concrete due to temperature and shrinkage occurs simultaneously. An increase in drying shrinkage arising from delays in curing also affects the barrier cracking. Additionally, drying shrinkage, beyond the very early ages, increases the width of cracks that have formed due to thermal loads. Penetration of moisture and other corrosive agents through the cracks causes reinforcing steel corrosion leading to horizontal cracking and finally the delamination and spall. Insufficient consolidation of concrete often observed in slipformed barriers accelerated the deterioration. Concrete parameters controlling the thermal load are the cement type, content, and fineness, ambient temperature at the time of concrete placement, and the time of inception of curing. From field inspection data and the finite element results it was found that the minimum possible full-length vertical crack spacing is equal to barrier height. One of the most important recommendations drawn from this study is the implementation of crack arrestors at about 3-feet intervals. The crack arrestors, some with cracks formed at full length, should be sealed with durable silicone-based flexible material during first scheduled maintenance cycle. Additional recommendations include the substitution of cement with mineral admixtures and the development of detailed specifications for barrier construction.

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CAUSES AND CURES FOR CRACKING OF CONCRETE BARRIERS

Submitted to the

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A Michigan DOT Center of Excellence

WAYNE STATE UNIVERSITY

Department of Civil & Environmental Engineering
5050 Anthony Wayne Drive
Detroit, MI 48202

Dr. Haluk Aktan, Ph.D, P.E
Professor
Tel: 313-577-3825
Fax: 313-577-9850
E-mail: Haluk.Aktan@wayne.edu

Mr. Upul Attanayaka
Graduate Research Assistant
Tel: 313-577-9293
Fax: 313-577-9850
E-mail: upul@wayne.edu

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EXECUTIVE SUMMARY

This research need was established in a report by J.F. Staton and J. Knauff (1999), titled, "*Evaluation of Michigan's Concrete Barriers"*. The report described that many of the current generation barriers used by the Michigan Department of Transportation (MDOT) are deteriorating at a rate greater than expected. This study was designed in order to further evaluate the observations described in the above report and to develop a comprehensive understanding of the barrier life span, from construction to repair or replacement.

The objectives of this project were to investigate the causes of concrete bridge barrier deterioration with the goal of developing strategies for corrective action. The strategies were developed by examining and determining the distress states and mechanisms. Material selection, mixture design, and/or modifications to construction procedures are proposed to reduce premature deterioration.

The project was designed in eight tasks including literature review, a nation wide survey of State Highway Agencies (SHAs), inspection of existing barriers, monitoring barrier construction, laboratory testing, and data synthesis. The primary goal of the project was to develop recommendations in order to minimize or eliminate barrier cracking.

As a first step, a preliminary inspection of existing barriers was performed and observed distresses were documented. Observed distresses and their causes were examined in the literature review. The causes of distress are classified as design related, material related, and construction practice related. In any case, most of the distress types can be eliminated with good construction practices. Most distresses are observed at early ages and initiate from unmitigated volume change of concrete at very early ages. Improper construction practices that initiate the progression of distress are early removal of forms in form-cast barriers, insufficient consolidation, and lack of wet curing.

The survey responses of State Highway Agencies provided information on their experiences with regard to the early-age cracking problem with barriers and, materials and construction practices used for concrete barriers (especially New Jersey type).

All the survey respondents indicated that premature distress is observed on concrete bridge barriers. Those observed distresses are of the same types observed in Michigan. Though all the respondents identified premature distress, only Illinois, New Mexico, Vermont, and Virginia acknowledged that they have experienced an overall durability problem with the bridge barriers. Nationally, both form-cast and slipformed barriers are commonly used. Precast New Jersey type barriers are also used in States of Florida, Maryland, New Hampshire, New York, Texas, Vermont, and Virginia. Sprayed curing compound on the slipformed barrier surfaces is the most often cited method of curing. States of Illinois, Minnesota, New Hampshire, New Mexico, New York, and Washington use a different concrete mix design for the barriers than for the deck. All the respondents specified the use of ground granulated blast-furnace slag (GGBS) and flyash (FA) in the mix design for the goal of reduction of the concrete permeability. States of Massachusetts, New York, and Virginia also use silica fume (SF) along with GGBS and FA. Except in States of Alabama, Idaho, Nevada, North Dakota, Virginia, and Washington the most prevailing method of surface finishing is the rubbed method. Most of the respondents emphasized changing the mix design and curing procedure for improved durability of New Jersey type barriers.

During field inspections, the condition of barriers of a total of 21 bridges consisting of 155 barrier segments with a total length of 3,729 ft. were documented. According to the inspection data, the vertical (or transverse) cracking is the leading cause of most of other distress types. The number of vertical cracks is important in establishing the deterioration rate.

Horizontal cracks can be classified as either local or continuous. The local cracks are often on the barrier vertical face. Continuous horizontal cracks are also mostly observed on the vertical face, about the level of top longitudinal reinforcement. In barriers with horizontal cracking, significant section loss is often observed around the barrier top portion.

The construction of four bridge superstructure replacement projects with New Jersey type barriers was monitored. In two of the projects, the barriers were slipformed. In the other two projects, barriers were form-cast using metal forms on the traffic bearing side and wood forms on the fascia.

In the slipformed barriers, concrete was not sufficiently consolidated. Though it is difficult to conclusively evaluate the barrier interior without taking well distributed core samples, while the joints were being cut, honeycombing and large cavities were observed. The curing compound was sprayed using the MDOT recommended procedure. However, the spray was not uniform over the barrier surface as specified in the MDOT Standard Specifications for Construction. Two days after placement, inspection revealed map cracking on most of the portion of the barrier surface as well as few full-length vertical cracking. In form-cast barriers, the top surface of the concrete is not covered or protected from direct atmospheric exposure. Forms were removed approximately 18 hours following construction (this is in compliance with the specification requirements). Curing compound was not applied after form removal even though it is required by the specifications. Visual inspection upon form removal did not reveal any visible cracking.

Mechanical properties of barrier concrete were obtained from compressive strength and elasticity modulus tests in accordance with ASTM C 39 and ASTM C 469, respectively. Ultrasonic pulse velocity (UPV) test was performed in compliance with ASTM C 597. The rapid chloride permeability test (RCPT) was performed in accordance with ASTM C 1202. The absorption and air permeability tests were performed for determining the void ratio and to obtain the limits of absorption. The absorption test was performed in compliance with ASTM C 642. The air-permeability test was performed using a special apparatus not yet standardized by ASTM. Test results indicate that two of the bridge barriers were cast with 28-day concrete strength in excess of 6000 psi. Several core samples were obtained from existing barriers. Core samples taken from distress-free zones showed lower air-permeability values than the measurements of the standard specimens obtained from new barriers indicating greater resistance to moisture as well as chloride ion penetration. Core samples taken from the zones of distress displayed excessive leakage. The existence of cracks and large voids triggered the leakage. The permeable pore space measured from the absorption test did not reveal any significant difference between cores and standard specimens. UPV test results measured on cores and standard specimens from new barriers also showed similar results. The Coulomb values measured according to ASTM C 1202 on standard specimens were significantly higher than that of core specimens. It was also observed that there is significant variability in the concrete properties obtained from cores and standard specimens.

The primary factor affecting durability of concrete barriers in Michigan was determined as the formation of multiple full- or partial-depth vertical cracks. The causes of cracking were identified as the internal restraint stresses resulted from thermal and shrinkage loads during cement hydration. It was also established that other distress types often emanate from vertical cracking. The reduction or control of vertical cracking will most certainly improve barrier service life.

Inspection data from existing barriers showed that the average full-length vertical crack spacing is twice the barrier height. Upon the inspection of newly slipformed barriers (two days following placement), full-length vertical cracks were also observed at a ratio of average full-length crack spacing to barrier height also as two. On the other hand, finite element analysis of a barrier segment with full base restraint showed that the full-length vertical crack spacing is equal to the barrier height. Vertical crack spacing increases with reduced base restraint conditions. The difference between observed crack spacing in the field and the results of the analysis is due to reduced barrier base restraint.

According to the findings of this study, the early barrier deterioration is initiated by the vertical cracking and accelerated by the presence of voids, cavities, and the overall concrete quality of the barrier. Recommendations are made by emphasizing the fact that early-age crack control, or in more general terms crack management, is the key to durable barriers. It is recommended that crack arrestors are used by placing a trim inside the forms at approximately 3 feet intervals. The crack arrestors, some with cracks formed at full length, should be sealed with a durable silicone-based flexible material during the first scheduled maintenance cycle. Additional recommendations include substitution of mineral admixtures, shifting barrier casting process to evening or night, protecting the top surface of form-cast barrier with curing compound or a wet burlap, and delaying form removal to five or even seven days after concrete placement.

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1 INTRODUCTION

This research need was established in a report prepared by J.F. Staton and J. Knauff (1999), titled, "*Evaluation of Michigan's Concrete Barriers"*. The observations described in the report indicated that many of the current generation barriers used in Michigan are deteriorating at a rate greater than expected.

Prior to the 1960's standard bridge railings were Type R-4 and R-5. From 1961 to the 1970's barrier Types R-11 and R-12 were constructed. In 1967, a solid section G.M. (Type 1) barrier was constructed. The G.M. (Type 1) barrier height was 32 inches and the widths of the cross-section were 16 inches at the base and 6 inches at the top. In 1976, the G.M. type barrier was replaced with New Jersey Type 2 and in 1977, the New Jersey Type 3. In 1982, the cross-section of the New Jersey Type 2 and 3 was modified and new standard design configurations (Types 4 and 5) were developed (Staton and Knauff 1999).

Current construction practice of "slipform" casting of concrete barriers started in 1972. Prior to 1972 bridge barriers were always form-cast. In slipforming a very low slump concrete is placed as a steel form is slowly moved, generating an extruded concrete profile. Little or no vibrating is performed in order to retain the limited workability of concrete. Presently, form-casting of barriers is often the most common and essential way. This is because of the recent use of textured surfaces to enhance barrier esthetics which necessitate form-cast construction to accommodate the architectural form lines.

This study was designed in order to further review earlier observations and to develop a comprehensive understanding of the barrier life cycle, from construction to repair or replacement.

1.1 BACKGROUND

This research project "Causes and Cures for Cracking of Bridge Barriers," was sponsored by the Michigan Department of Transportation and performed by the Center for Structural Durability, a collaborative effort between Wayne State University (WSU) and Michigan Technological University (MTU). The findings of Michigan Technological University (MTU) are described in the report titled, "*Causes and Cures for Cracking of Concrete Barriers*" by Van Dam et al. (2003). The WSU work focused on the development of a crack management procedure for the bridge barriers as detailed in this report.

1.2 OBJECTIVES AND METHOD

The work on this project was initiated on May 1, 2002. The objectives of this project were as follows:

1. Investigation of the causes of concrete bridge barrier deterioration with the goal of developing strategies for corrective action.
2. Examination and determination of the barrier distress states and mechanisms, then development of material selection, mixture design, and/or construction strategies to prevent rapid barrier deterioration.

This project consists of eight tasks. Sections of this report corresponding to the project tasks are described below:

Task 1: Literature review includes, a review of relevant MDOT design, construction, and maintenance practices concerning bridge barriers, a historical review of bridge barrier design, material specifications and construction practices, and a review of literature and reports dealing with cracking distress and its impact to bridge barrier service life.

Task 2: A nation wide survey on bridge barrier deterioration was prepared and distributed to State Bridge Engineers in all 50 states. The results of this survey was used to assess the type of concrete barrier distress and deterioration problems observed nation wide, as well as their material specifications, design, construction practices, acceptance parameters, and acceptance tests.

Task 3: Selected bridge barriers were visually inspected at arms length for documenting the visible distress types, extend, and progression. Representative categories of barrier distress were established from pre-inspection field observations. These categories were used for selecting barriers for coring. The barriers inspected showed both good and poor performance, representing a range of age, material types, and construction methods. The inspection was performed visually using an assessment tool. The distress types were identified and classified according to their significance on barrier durability performance. Extensive digital photographs of each inspection site were obtained to document the observed conditions.

Task 4: Eight of the barriers were cored for laboratory investigations. Sampling included two core specimens for petrographic examination and one for permeability testing, and some additional specimens depending on the visual assessment.

Task 5: Bridge barrier constructions during deck and/or full structure replacement projects were monitored and compliance with the Michigan Department of Transportation (MDOT) - Standard Specifications for Construction was appraised. Of special interest was the monitoring of slipformed barrier construction, with particular interest in the casting as well as the curing procedures.

Task 6: Laboratory testing of the specimens obtained from existing and new barriers was performed. The petrographic and distress characterization tests are reported separately in a report by the Michigan Technological University (Van Dam et al. 2003). Mechanical property testing of standard specimens was performed to assess the material properties related to cracking and durability. Gas permeability, water permeability, and porosity measurements were made on core specimens and standard specimens.

Task 7: Barrier performance parameters were established from the results of the literature review, visual inspections, monitoring of new construction, and laboratory analysis. Additional performance parameters were defined contributing to the service life of concrete bridge barriers. The recommendations include modifications to the construction procedures, curing practices, material specifications, and a crack management procedure.

Task 8: Quarterly reports and a final report were prepared for submittal to the Research Advisor Panel. The final report summarizes the entire project and includes recommendations for changes to bridge barrier design practices, materials specifications, and construction procedures in order to increase bridge barrier service life.

2 STATE-OF-THE-ART LITERATURE REVIEW

2.1 OVERVIEW

In order to have an understanding of barrier condition, an initial survey was performed for identifying common distress states. Fourteen bridges with NJ barriers were randomly selected in the Detroit Metropolitan area for distress identification. The distress was recorded in photographs. The data collected was used for defining the scope of the investigation.

The photos taken that recorded the condition of bridge barriers and the associated distresses were in consensus with Staton and Knauff (1999) and documented that the primary distress types were spalling or disintegration, delamination, horizontal cracking, corrosion, efflorescence, vertical cracking (termed as transverse cracking), map cracking, and, popouts (Table 2-1).

Table 2-1. Types of Distress Observed on Reinforced Concrete Barriers

Distress Type	Description	
Spall or Disintegration	A fragment, usually in the shape of a flake, detached from a larger mass; a small spall shape is roughly circular or oval or in some cases elongated, is more than 0.8 in. in depth and 6 in. in greatest dimension.	
Delamination	A separation along a plane parallel to a surface.	
Horizontal cracking	Cracks that develop parallel to the length of a member. Also referred to as longitudinal cracking.	

Table 2-1. Types of Distress Observed on Reinforced Concrete Barriers

Distress Type	Description	
Corrosion	Destruction of rebar by chemical, electrochemical, or electrolytic reaction with its environment.	
Efflorescence	A deposit of salts, usually white, formed on a surface, the substance having emerged in solution from within concrete and subsequently been precipitated by evaporation.	
Vertical Cracking	Cracks that develop at right angles to the longitudinal direction of the member. Also referred to as transverse cracking.	
Map Cracking	Intersecting cracks that are near the concrete surface.	
Popouts	The breaking away of small portions of a concrete surface which leaves a shallow, typically conical, depression; small popouts leave holes up to 0.4 in. in diameter, medium popouts leave holes 0.4 to 2 in. in diameter, large popouts leave holes greater than 2 in. in diameter.	

In the literature review, topics were discussed among the pertinent articles in the order of type of distress, causes of distress, and the corrective actions taken to mitigate bridge barrier distress.

2.2 TYPE OF DISTRESS

2.2.1 Vertical (Transverse) Cracking

Vertical or transverse cracking that is observed at early ages is often the predominant type of distress in a given barrier segment. The vertical cracking is identified as the leading cause of other distress types. These cracks lead to ingress of moisture, which results in early initiation of corrosion. Though epoxy coat is used to prevent/delay reinforcement corrosion, recent studies revealed that epoxy coating debonds with moisture and reinforcement is still susceptible to corrosion (see Section 2.2.3). The causes of vertical cracking on newly constructed barriers were examined on Vachon Bridge, Montreal Canada. Vertical cracks were observed within a few days after concrete placement. The observed vertical crack spacing was approximately 0.8 times the height of the barrier. Cracking forms on barriers primarily under restrained volume change. Barriers can be assumed to be fully restrained at the base (Cusson and Repette 2000). Additional work focusing on restrained cracking of concrete members shows that most vertical cracks occur within a few days after concrete placement (Cusson and Repette 2000, Al Rawi and Kheder 1990, Kheder et al. 1994, Carlson and Reading 1988 and Wiegrink et al. 1996). Al Rawi and Kheder (1990) observed vertical cracking on fully base restrained unreinforced concrete walls at a spacing of 1.24 times the wall height. According to ACI 207 (2001), fully base-restrained unreinforced concrete walls ultimately attain full-length vertical cracks spaced between one to two times the height of the wall.

2.2.2 Map Cracking

Map cracking is a common defect observed on the barrier surface. The cause of map cracking is attributed to factors related to material properties and construction practices. Map cracking of barriers is often related to early thermal shrinkage cracking due to hydration of cement, an exothermic process. The center of the barrier retains sufficient heat to hydrate; meanwhile, the surface is subjected to water loss and subsequently thermal stresses, and causes map cracking on the surface (Baradan et al. 2002 and Kovler 1995). In another case, rapid loss of water from the surface during the hydration process also can cause map cracking.

2.2.3 Corrosion

Corrosion influences the long-term performance of reinforced concrete, particularly in aggressive environments. Corrosion causes spalling of concrete cover and leads to loss of integrity and strength. Corrosion initiation and propagation reduces the area of steel and the bond between concrete and steel (Stewart and Rosowsky 1998). There is an agreement among concrete researchers that cracking can accelerate the ingress of corrosive agents and accelerate corrosion. However, ACI 207.2R (2001) states that cracking transverse to the reinforcing bar does not accelerate corrosion.

Corrosion of steel in a concrete member can develop within a few years after placement of concrete, depending on concrete design, construction practices, and exposure level. Corrosion does not remain limited to the initiation zone, especially in cyclic wetting-drying areas of structures. Uncontrolled corrosion leads to severe deterioration and very rapid loss of capacity of concrete members (Alampalli and Owens 2000, Costa and Appleton 2002, and Montemor et al. 2002).

Epoxy coating is used to inhibit the corrosion of reinforcement. In a study for assessing the effectiveness of epoxy-coating, 18 bridge decks in Virginia was evaluated. The bridge decks were 2 to 20 years old at the time of investigation. A total of 250 concrete cores were extracted from these bridge decks. In all cases except one bridge, adhesion loss of epoxy coating to the steel surface was detected. Chloride, moisture, and oxygen ingress through the debonded coating initiate corrosion. Additional investigations performed in Minnesota and Florida confirms the finding of the Virginia study. Virginia study documented that epoxy coating debonds in as little as 4 years (Pyć et al. 2000).

2.2.4 Horizontal Cracking

Horizontal cracks can be related to the consolidation of plastic concrete into the voids that remained in the slipforming process. Additionally, bridge deck vibrations under traffic can be a factor in generating further consolidation of plastic concrete and should be examined (Cusson and Repette 2000). Plastic settlement cracking can be the source of continuous horizontal cracks

(Baradan et al. 2002). Additional local cracks, which are observed with the continuous horizontal cracking, can be related to corrosion expansion (Allan 1995).

2.2.5 Spalling and Disintegration

Spalling and disintegration are characteristics of the ultimate stages of distress in reinforced concrete structures. With vertical and horizontal cracking, increased volume of moisture with deicers and other deteriorative agents ingress into concrete initiating and accelerating reinforcement corrosion. Further, the freeze-thaw effect with the presence of moisture rapidly increases the length and width of the cracks. Spalling is typically defined as the loss of sizable chunks of intact concrete. Disintegration is usually the loss of small particles and individual aggregate particles due to freeze-thaw effects, chemical attack, or poor construction practices (EM 1110-2-2002 1995).

2.3 ANALYSIS OF CAUSES OF DISTRESS

Primary causes of early-age cracking are identified as drying shrinkage, thermal loads, and restraint of concrete. Volume change in concrete is mainly due to autogenous shrinkage, drying shrinkage, and thermal loads. Autogenous shrinkage occurs due to self-desiccation of cement paste. Autogenous shrinkage is primarily observed in mass concrete. Drying shrinkage occurs while water on the concrete surface evaporates. As drying continues, water in the concrete mass is removed either by evaporation or by hydration, and volume change occurs. In principle, uniform evaporation of free water causes little or no shrinkage in concrete (Neville 1995).

Hydration of cement is an exothermic process, thus generates heat. The amount of heat generated and the age of concrete at which the maximum temperature difference occurs between the interior and exterior of the concrete element depends on the geometry of the structure, ambient temperature at the time of casting, and the concrete mix. The temperature difference between the interior and exterior of the element causes restraint volume change and consequently stresses within the barrier section (ACI 207.1R 2001 and Siew et al. 2003).

2.3.1 Structural

Cracking due to restraints on a reinforced concrete member has been discussed in ACI 207.2R (2001). Barriers can be assumed to be fully base-restrained continuous members. Cracking due to volume change is a result of the combined effects of strain due to volume change and restraint. When strain due to volume change of a concrete component exceeds the concrete cracking strain accompanied by sufficient stress due to restraint effects, concrete will crack (Al-Rawi and Kheder 1990 and Hossain et al. 2003). Once the crack initiates, it can extend at a much lower tensile stress than that which was required for initiation. A propagating crack will increase the tensile stress at every section along the uncracked plane. The full base restraint will cause crack propagation up to 0.2 or 0.3 times the height of the section. At that point, the crack is free to propagate to the full section without any shrinkage due to the unbalanced stress between the cracked and uncracked portions of the section (ACI 207.2R 2001).

Studies on restrained shrinkage described differential shrinkage between the base material and the barrier. This differential shrinkage creates stress that exceeds the tensile strength of immature concrete, and results in shrinkage cracking (Cusson and Repette 2000, Al Rawi and Kheder 1990, Kheder et al. 1994, Carlson and Reading 1988, and Wiegrink et al. 1996). Vertical or transverse cracking is often observed on flat structures such as decks and walls. The formation time of cracks depends on free shrinkage, tensile strength and elasticity modulus, and creep (Wiegrink et al. 1996). Another factor involved in shrinkage cracking is the concrete member geometry, especially during thermal shrinkage at an early-age (Schutter and Taerwe 1996). The exposed-surface to volume ratio of the member affects the rate of shrinkage and heat losses.

2.3.2 Environmental Effects

The environmental condition such as ambient temperature, relative humidity, and wind velocity has a significant influence on the properties of fresh and hardened concrete. The rate of water evaporation and shrinkage strain increases with increased ambient temperature and wind velocity and decreased relative humidity. Elevated temperature exposure decreases the compressive strength of concrete. Environmental exposure conditions influence the concrete pore volume and hence the porosity. Concrete having large pore sizes and a hollow structure allows greater penetration of moisture at an elevated temperature than concrete at a lower temperature. Concrete placed at an elevated temperature is often less durable (Almulsallam 2001 and ACI 207.2R 2001).

Concrete shrinks when it is exposed to a drying environment. The amount of shrinkage depends on material properties, ambient temperature, relative humidity, and the age and size of the structure. Non-uniform moisture distribution, steel embedded in section, or aggregate can cause differential drying shrinkage. When concrete is placed, it is exposed to the ambient air. Water movement inside concrete occurs by diffusion. Therefore, the moisture content in concrete varies in both space and time, and moisture distribution in a given concrete section becomes non-uniform. Non-uniformity and internal drying shrinkage vary through the depth of the section; thus stress induced by differential drying shrinkage may cause surface cracks (Kim and Lee 1998).

2.3.2.1 Methods for Estimating Shrinkage Strain

2.3.2.1.1 Overview

The available shrinkage prediction models for concrete are mostly based on the ACI 209 and CEB-FIP models. Other models are derived from these models by incorporating additional parameters and modifications (Hani et al. 2003). Researchers have tried to evaluate shrinkage prediction models with different curing procedures. Mokarem et al. (2003) used crushed limestone gravel and diabase in concrete mixes in order to see the effects of aggregate type on shrinkage. For all types of aggregate, CEB-FIP 90 is found to be a better predictor than ACI 209, Gardner-Lockman, and the Bazant B3 model. The Gardner-Lockman and Bazant B3 models are found to be the second best prediction models. ACI 209 is found to be the least accurate in predicting drying shrinkage. On the other hand, Hani et al. (2003) found that ACI 209 is a good predictor for 28-day shrinkage and is more conservative for later ages. It was also established that the CEB-FIP 90 model is a good shrinkage prediction model for concrete at very early ages. Researchers state that the Bazant B3 model is a good prediction model for long-term shrinkage estimation.

The mathematical formulations for prediction models are very similar. Each model has an ultimate shrinkage value that predicts total possible shrinkage for given strength and cement type. Ultimate shrinkage is a constant only in ACI 209 for all types of cements and concretes. The shrinkage time function component is a time-dependent function varying based on drying time of concrete, which starts from the time of concrete placement. The shrinkage process stops at any time, at about 100% relative humidity. The shape of components is incorporated in the formula as volume to exposed-surface ratio. The relative humidity component is related to the average humidity of the environment.

2.3.2.1.2 Prediction Model Recommended in ACI 209

ACI 209 is an empirical approach to calculate free total shrinkage of concrete:

$$\varepsilon_{sh}(t) = \left[\frac{(t - t_c)}{35 + (t - t_c)} \right] \cdot \varepsilon_{sh\,u} \qquad \text{(2-1)}$$

where t is the age of concrete at which the shrinkage strain is to be predicted, t_c is the age of curing or the age at which concrete starts drying, and $\varepsilon_{sh\,u}$ is the ultimate shrinkage (in./in.).

In ACI equation the ultimate shrinkage value is constant and equal to 780μ strain for all types of concrete and curing conditions. Ultimate shrinkage is corrected based on a humidity function:

Table 2-2. The Relationship between Humidity Range and Correction Formula

Humidity range	Humidity function (γ_{RH})
40% < RH < 80%	1.40 – RH/100
80% < RH < 100%	3.00 – 3*RH/100

$$\varepsilon_{sh\,u} = 780 \text{x} 10^{-6} \text{ x } \gamma_{RH} \qquad \text{(2-2)}$$

The constant shrinkage approximation reduces the accuracy of predictions. Time (t) and age of curing (t_c) are the variables of the prediction function. The ACI formula is based on the assumption of a 7-day wet curing period upon placement. The shrinkage prediction becomes more accurate with increased curing duration.

2.3.2.1.3 Bazant B3 Model for Shrinkage Prediction

The Bazant B3 formulation was developed from an analytical statistical evaluation (Bazant 1995). Shrinkage at time *t*, (ε_{sh}(t)), is calculated considering the type of cement, curing, water content, and 28-day standard strength:

$$\varepsilon_{sh}(t) = \varepsilon_{shu} \times k_h \times S(t) \qquad (2\text{-}3)$$

Ultimate shrinkage (ε_{shu}) is formulated as follows:

$$\varepsilon_{shu} = \alpha_1 \alpha_2 \left(26 \left(w\right)^{2.1} \left(f'_c\right)^{-0.28} + 270 \right) \times 10^{-6} \qquad (2\text{-}4)$$

Ultimate shrinkage is a function of cement type (α_1), curing conditions (α_2), water content (w), and 28-day strength (f'_c).

Table 2-3. Coefficients α_1 and α_2 used in Bazant B3 Model

ASTM C 150 type cements	α_1
Type I	1
Type II	0.85
Type III	1.1

Curing coefficient	α_2
Steam cured concretes	0.75
Water cured or RH 100 %	1.0
Sealed specimens	1.2

The component k_h is related to the humidity of the environment, and the relationship is given in Table 2-4:

Table 2-4. Humidity Relation used in Bazant B3 Model

Relative humidity	k_h
for $h \leq 0.98$	$1 - h^3$
for h = 1	-0.2
for $0.98 \leq h \leq 1$	Use linear interpolation

The time function of shrinkage S(t) is a function of concrete age (t), duration of wet cure (t_c), and shrinkage time coefficient (τ_{sh}).

$$S(t) = tanh\sqrt{\frac{t - t_c}{\tau_{sh}}} \quad (2\text{-}5)$$

Shrinkage time coefficient τ_{sh} is size dependent as given below:

$$\tau_{sh} = k_t (k_s \times D)^2 \quad (2\text{-}6)$$

$$k_t = 190.8 (t_c)^{-0.08} (f'_c)^{-0.25} \quad (2\text{-}7)$$

k_s is cross section shape factor (for a rectangular section, it is 1) and *D=2*Volume/Surface* ratio (in.).

2.3.2.1.4 CEB-FIP 90 Model for Shrinkage Prediction

The European Concrete Committee formulation is described as one of the most accurate prediction models (Shah et al. 1996). The formulation of CEB-FIP 90 is as follows:

$$\varepsilon_{sh}(t) = \varepsilon_{shu} \times \beta_s (t - t_c) \quad (2\text{-}8)$$

$$\varepsilon_{shu} = \varepsilon_s \times \beta_{RH} \quad (2\text{-}9)$$

where ε_{shu} is the ultimate shrinkage and β_{RH} is the coefficient that incorporates the effect of relative humidity on the ultimate shrinkage, ε_s and $\beta_s(t-t_c)$ are given by Eq. 2-11 and Eq. 2-12, respectively.

Table 2-5. Humidity Relation used in CEB-FIB 90 Model

Relative humidity	β_{RH}
40% $\leq$ RH $\leq$ 99%, stored in air	-1.55 x β_{sRH}
RH $\geq$ 99%, immersed in water	0.25

$$\beta_{sRH} = 1 - \left(\frac{RH}{100}\right)^3 \quad (2\text{-}10)$$

The effect of concrete strength on shrinkage is incorporated with the following equation:

$$\varepsilon_s = \left[160 + 10 \times \beta_{sc}\left(9 - \frac{f'_c}{1450}\right)\right] \times 10^{-6} \qquad (2\text{-}11)$$

The cement type factor is given according to European cement types but it is also defined for the ASTM type cements as shown in Table 2-6.

Table 2-6. Cement Type Coefficient used in CEB-FIP 90 Model

Type of cement	βsc
Low heat development cements (Type II and Type V)	4
Rapid heat development cements Type I and Type III	5

The development of shrinkage with time is defined in the β_s coefficient and described in Eq.2-12:

$$\beta_s(t - t_c) = \sqrt{\frac{(t - t_c)}{\left[350 \times \left(\frac{h_0}{3.937}\right)^2 + (t - t_c)\right]}} \qquad (2\text{-}12)$$

where h_0 = *2x(cross sectional area/ perimeter of the member in contact with the atmosphere)*.

The CEB formulation is similar to the Bazant B3 prediction formula. It has an ultimate shrinkage and humidity coefficient as well as a time-dependent shrinkage function.

The European committee formulation is an accurate method for predicting shrinkage, since it incorporates almost all factors that may affect shrinkage. The results of this formulation are in close agreement with the Bazant B3 formulation in the long term (Mokarem et al. 2003 and Hani et al. 2003). In addition, the CEB-FIP 90 model is quite accurate for early-age predictions since it was developed for specimens cured in short durations (Hani et al. 2003).

2.3.2.1.5 Gardner – Lockman Model for Shrinkage Prediction

The Gardner-Lockman model was proposed in 2001 (Gardner et al. 2001). This prediction model is also known as the Gardner model. Formulation is given in Eq 2-13:

$$\varepsilon_{sh}(t) = \varepsilon_{shu} \times \beta(h) \times \beta(t) \tag{2-13}$$

where β(h) is described in Table 2-7 and ultimate shrinkage (ε_{shu}) and β(t) are given by Eqs 2-14 and 2-15, respectively.

Table 2-7. Humidity Relation used in Gardner-Lockman Model

Relative humidity	β(h)
h < 96%	$1\text{-}1.18h^4$
h ≥ 0.96	0.0

$$\varepsilon_{shu} = 1000 \times K \times \sqrt{\frac{4350}{f'_c}} \times 10^{-6} \tag{2-14}$$

$$\beta(t) = \sqrt{\frac{t - t_c}{t - t_c + 97(V/S)^2}} \tag{2-15}$$

where *V/S* is the *volume / surface ratio* and K is a function of cement type and given in Table 2-8 for ASTM type cements.

Table 2-8. Cement Type Coefficient used in Gardner-Lockman Model

Type of cement	K
Type I	1.00
Type II	0.70
Type III	1.15

Recent research on this model indicates that the formulation is accurate when estimating shrinkage of concrete containing low heat pozzolans (fly ash, slag, etc) (Mokarem et al. 2003).

2.3.2.2 Evaporation of water from fresh concrete

Heat of hydration in fresh concrete, amount of plastic shrinkage, and plastic shrinkage cracking depend to a great extent on the rate of evaporation from fresh concrete, which influences the strength and durability of the concrete. It is recognized that the level of evaporation from the surface of fresh concrete depends on the prevailing ambient temperature, relative humidity, wind speed, and the temperature of the fresh concrete. The evaporation rate depends on climatic conditions because at the beginning of casting, water exists at the surface of the concrete (bleeding water). Additionally, the moisture movement within the solid is rapid enough to maintain a saturated condition of the surface. The drying process of porous media can be divided into two periods referred to as initial and terminal drying period. In the case of concrete, initial drying period consists of two evaporation stages. The mechanism of moisture removal during the first stage is equivalent to evaporation from a liquid water surface. The evaporation-drying rate of concrete can be calculated from heat transfer relationship (Razek and Enein 1999):

$$R_c = h_c (T_{da} - T_w) / L \qquad (2\text{-}16)$$

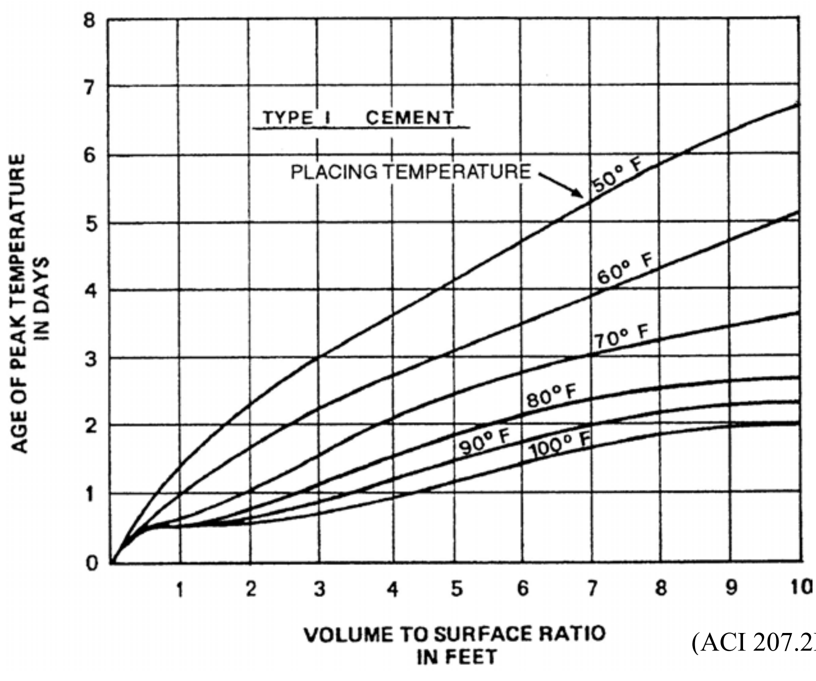
where, R_c is the evaporation drying rate (lb/ft^2/hr), h_c is convection heat transfer coefficient (Btu/ft^2/hr/K) and can be calculated by relation $\mathrm{h_c} = 4.28 + 0.375\mathrm{v}$ (v is the wind velocity in ft/s), T_{da} is the dry bulb temperature (ambient air temperature) (K), T_w is the wet bulb temperature (K) and can be calculated from ASHRAE Psychrometric Chart No.6 (ASHRAE Handbook 1992) by knowing relative humidity and dry bulb temperature of air, and L is the latent heat of vaporization of water (Btu/lb).

2.3.2.3 Hydration Temperature Stresses

Hydration of cement is an exothermic process that causes a temperature rise within a concrete mass. This initial temperature rise and expansion induces no residual compressive stresses in concrete when changing from a plastic state to a solid state. This is because of the extremely low modulus of elasticity of the concrete at this plastic-to-solid state. When the concrete reaches its peak temperature, it has also solidified. Subsequently, the hardened concrete begins to cool to ambient temperature. During the cooling process, the fixed base of the barrier restrain the shrinkage. This phenomenon will in turn cause tensile stresses and cracking of the barrier. The magnitude of thermal shrinkage in the barrier depends on the difference between the peak concrete temperature and the temperature of the supporting deck at that time. Unlike drying shrinkage, which may take over a year, thermal shrinkage is more rapid and loads the concrete over a short period (a few days). Thus, concrete creep properties cannot fully engage to relax the concrete and mitigate cracking (Purvis et al. 1995).

2.3.2.4 Hydration Temperature

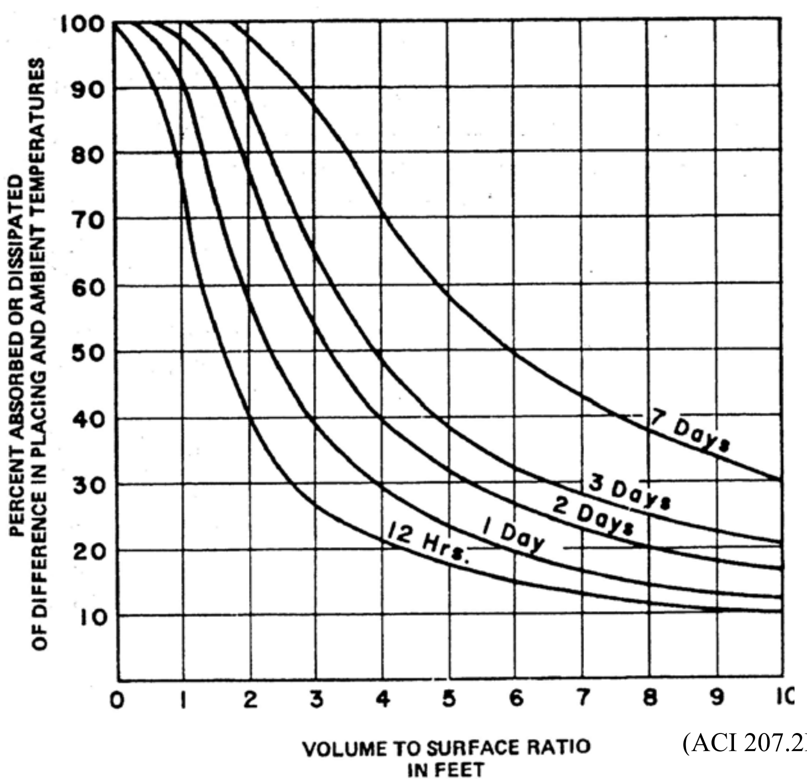
The rate and magnitude of concrete temperature rise during the hydration process depend on several factors. These factors include cement composition (cement type) and fineness, amount of cement per unit volume of concrete, ambient temperature, concrete temperature during placement, and amount of heat lost or gained during hydration process. The exposure conditions and volume to exposed-surface area ratio of bridge barrier governs the amount of heat lost or gained.



(ACI 207.2R 2001)

Figure 2-1. Effect of concrete placing temperature and volume to surface (exposed to environment) ratio on age at peak temperature for Type 1 cement

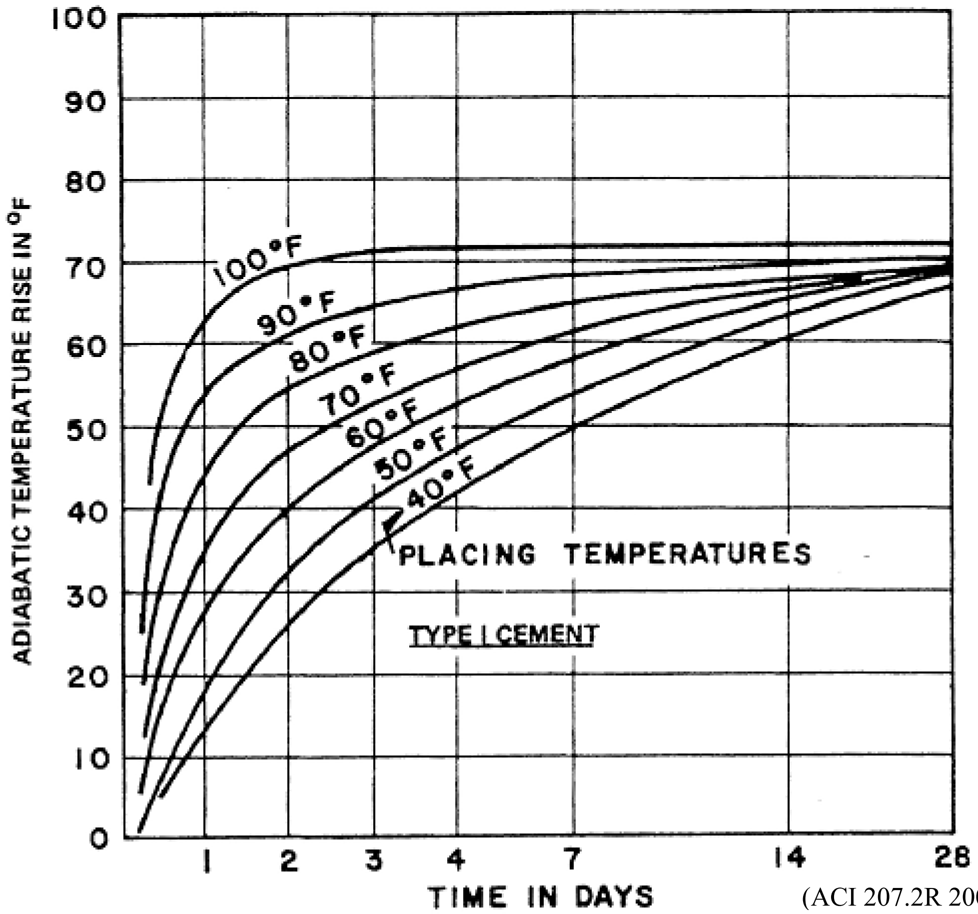
If the concrete placing temperature and volume/surface ratio of bridge barrier are known, the time of achieving peak temperature is determined from Figure 2-1, provided Type 1 cement is used. During the hydration process there is a temperature difference between the concrete forming the barrier and the ambient air. After determining the peak temperature from Figure 2-1, Figure 2-2 can be used to compute the percent absorbed or dissipated heat between placing and ambient temperature.



(ACI 207.2R 2001)

Figure 2-2. The effect of volume to surface ratio and age at peak temperature on percent absorbed or dissipated of difference in placing and ambient temperature

New Jersey bridge barrier geometry gives a volume to exposed-surface ratio (V/S) of 0.42 feet. Consequently, from Figure 2-2 the rate of heat absorbed or dissipated approaches 100%. Thus, the effective concrete temperature during placement approaches ambient temperature. Concrete temperature during placement is also a parameter for adiabatic temperature rise (Figure 2-3). The temperature rise within the concrete barrier also depends on the exposed barrier surface condition. As depicted in Figure 2-4, if the exposed concrete surface is kept wet, the temperature rise decreases significantly.



(ACI 207.2R 2001)

Figure 2-3. Effects of placement temperature on adiabatic temperature rise

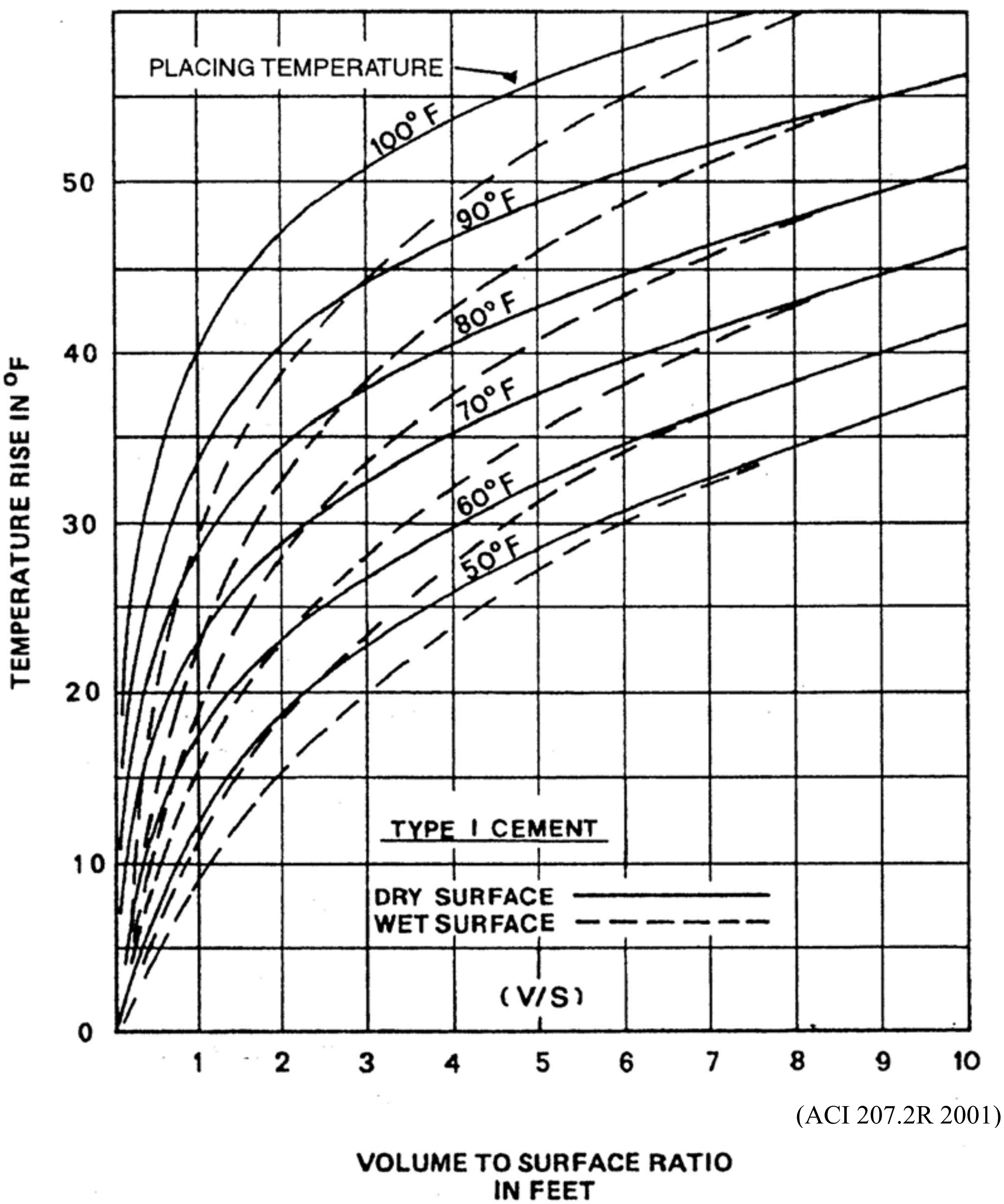
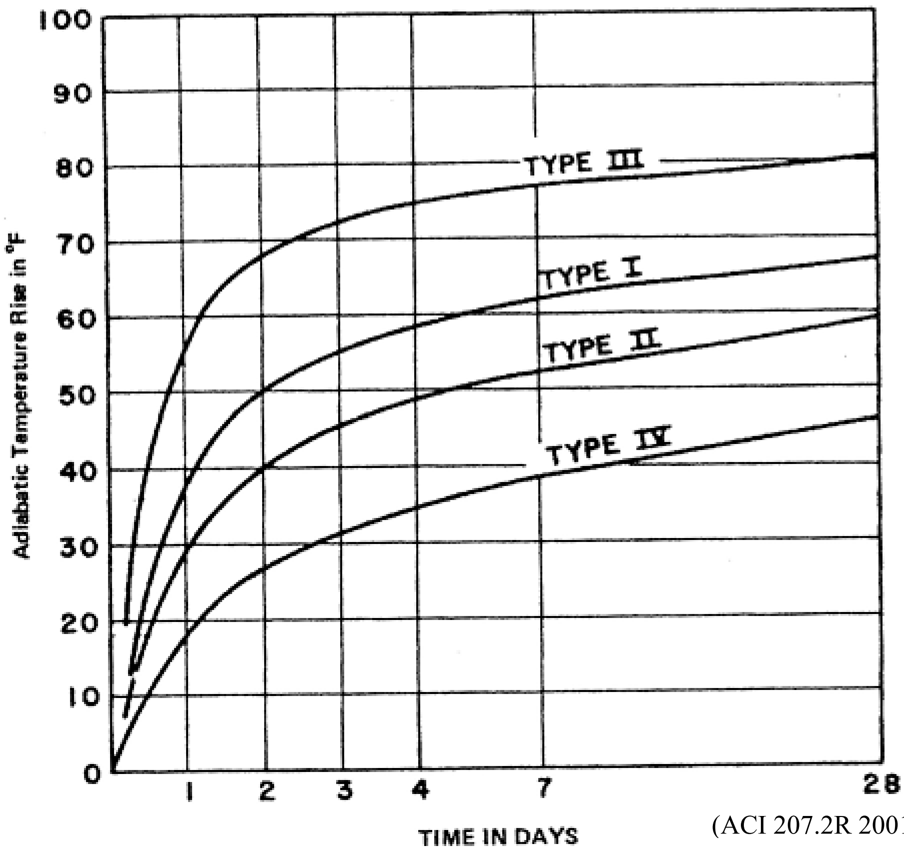


Figure 2-4. The effect of volume to surface ratio and exposed surface condition on temperature rise of concrete members (376 lb/yd^3)

Cement type, content, and its fineness affect the adiabatic temperature rise in a concrete component. Figure 2-5 shows the adiabatic temperature rise for different cement types. In developing Figure 2-5, cement types and the respective average fineness given in Table 2-9 were used. For cements with different finenesses, Figure 2-6 can be used to calculate the correction factors. In summary, the maximum amount of heat generated in concrete is directly proportional to the amount of cement in the concrete mix.

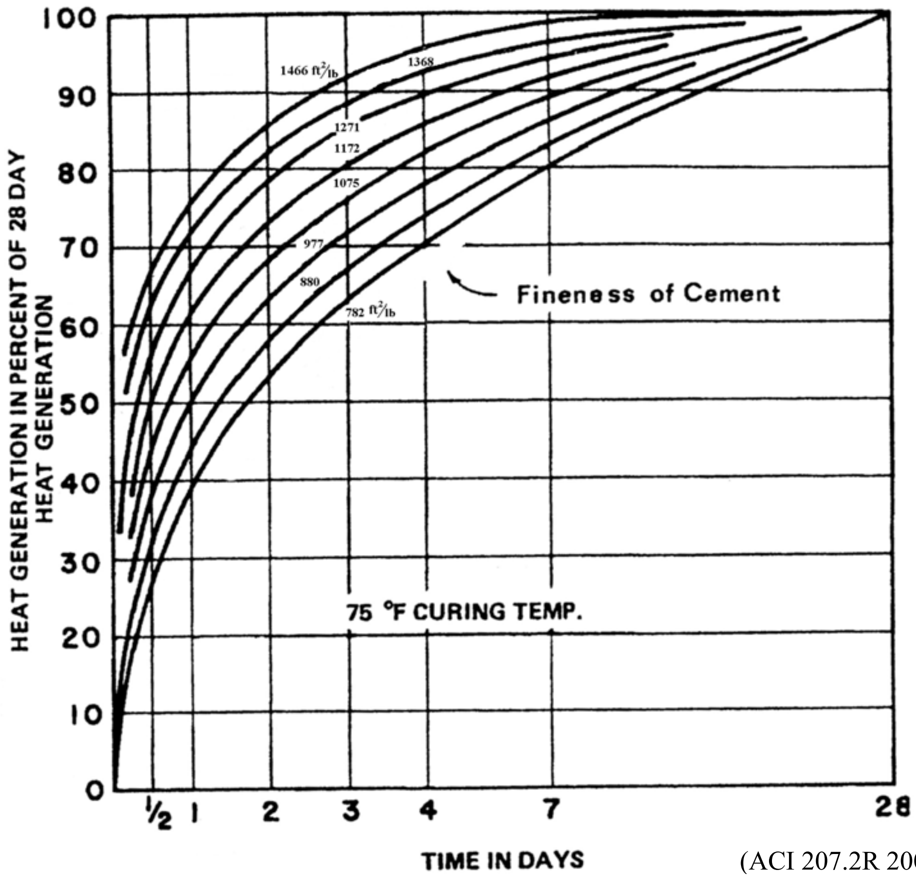


(ACI 207.2R 2001)

Figure 2-5. Variation of adiabatic temperature rise with age of concrete for different types of cements

Table 2-9. Cement Types and Fineness used for Developing Graphs in ACI 207.2R

Cement Type	Fineness ASTM C 115 ft^2/lb
I	875
II	924
III	992
IV	933



(ACI 207.2R 2001)

Figure 2-6. Effects of cement fineness on heat generation

2.3.3 Concrete Construction, Placement, and Curing Errors

A major factor that reduces the service life of a structure is due to errors during placement and curing. Shallow concrete cover, misplaced or missing reinforcements, substandard curing, and other substandard construction practices are often observed in many projects. However, the severity of the effects of the errors on the service life of the concrete structure is controlled by exposure conditions. Mistakes in material selection, construction practices, and post-construction work should be anticipated and immediate corrective measures should be taken during early-age preventive maintenance activities. Specifications should cover possible errors (Jaycox 1982).

The amount of water required for the hydration process should be available if the concrete is to achieve its full potential quality. During the first few hours after concrete placement, specifically prior to start of wet cure, if high wind velocity, high temperature, and low relative humidity conditions exist, excessively rapid drying can cause plastic shrinkage cracking. Permanent strength loss can result if moisture supply is insufficient. Substandard application of the curing compound or insufficient moisture during the first hours and substandard curing following the first few hours is detrimental to the concrete performance at later ages. In addition to the permanent strength loss due to insufficient moisture during the hydration process, other consequences are the increased permeability of concrete and map cracking (RILEM-42-CEA 1981).

The reinforcing bar position and the quality of the surrounding concrete primarily control the bond quality of deformed bars. Upon concrete placement, due to the effect of water gain and sedimentation of coarse aggregates under reinforcing bars, a porous layer of concrete can form under the reinforcement and its ribs as shown in Figure 2-7 (Park and Paulay 1975). Due to this effect, the top rebars of a concrete barrier will have poor bond quality when compared with the bottom rebars. Also, consolidation of concrete immediately upon placement causes a relative downward movement of the concrete surrounding the top reinforcement. This process forms a conduit under the horizontal rebars (Park and Paulay 1975). If vertical cracks occur, water and other contaminants that penetrate through the cracks propagate through the conduit along the length of the rebar, initiating corrosion along the full length of reinforcement. Corrosion of the rebar initiates further cracking along the top horizontal reinforcement.

Slipforming is a widely used construction technique for reinforced concrete bridge barriers. This technique gained high acceptance in the industry and the highway agencies because of its speed. Slipforming is the extrusion of very low slump concrete around the reinforcements to form the barriers. The extruded concrete from the pavement machine is hand floated, broomed, and sprayed with curing compound. No further curing is specified or performed upon the application of the curing compound. If the curing compound does not form a uniform impervious membrane, the barrier is exposed to the ambient air at very early ages. Concrete exposure to the environment at early ages affects compressive and tensile strength gain of concrete as described in RILEM 42-CEA (1981). Additionally, thermal and drying shrinkage strains are amplified. These effects, combined with restraint effects, may create cracking and other distresses. The distress may either be due to loosely consolidated concrete material that inhibits further hydration to restore strength or to the substandard bond between the concrete and reinforcement (RILEM-42-CEA 1981).

In addition to shrinkage and thermal cracking of extruded concrete during first few days, bleeding of mix water during the first two or three hours after placement causes plastic shrinkage of newly placed concrete, leading to premature cracking if constrained. For barriers, the plastic shrinkage may be quite large. These cracks may exhibit some regular pattern (RILEM-42-CEA 1981).

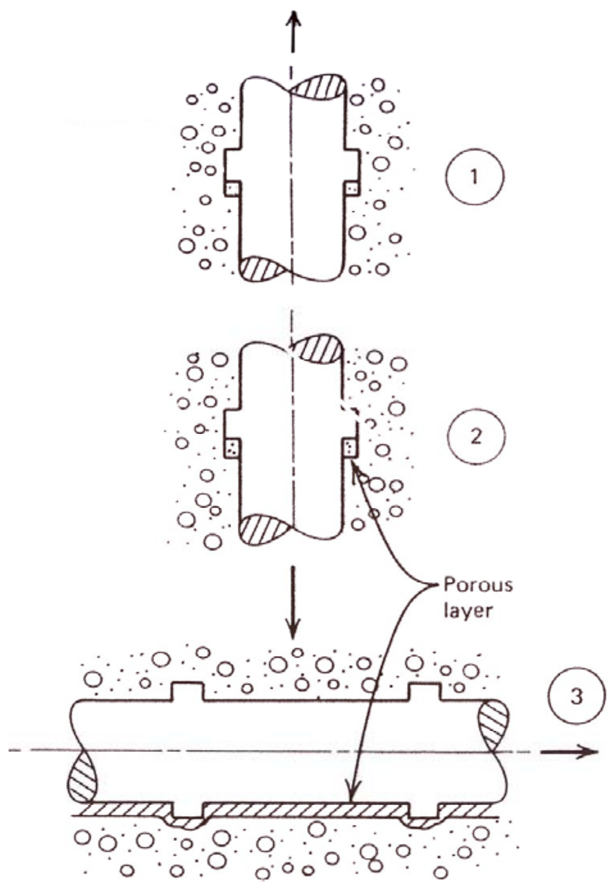


Figure 2-7. The porous layer under the ribs and the rebar upon concrete placement

2.3.4 Material

Material properties are major parameters in predicting service life of bridge components. The research on crack resistance of decks showed that high performance concrete has relatively better performance than ordinary concrete designs regardless of workability, construction practice, or cost (Alampalli and Owens 2000).

In addition to thermal loads, chemical effects, excessive loads, concrete cracking may be due to shrinkage. In the literature, based on the crack formation time, two shrinkage phenomena are described. These are drying shrinkage and autogenous or plastic shrinkage (Li et al. 1999). Drying shrinkage occurs gradually during the first 30 days after placement (Alsayed 1998). The drying shrinkage rate with time is controlled by the rate at which concrete loses moisture. In that respect, curing influences shrinkage rate. The parameters controlling total shrinkage are the amount, strength, and elasticity modulus of the coarse aggregate and the type and amount of cementious materials. Plastic shrinkage of high strength concrete is more than normal strength concrete with respect to cracking under hot and dry weather conditions (Samman et al. 1996 and Wiegrink et al. 1996). High strength concrete also has higher strength than normal strength concrete at early ages. An important parameter in shrinkage cracking of concrete is the cracking strength. Cracking strength is lower than nominal tensile strength at early ages (Altoubat and Lange 2001).

The time to cracking of concrete varies based on the mix properties. With decreasing water-to-binder ratio, cracks occur sooner (Altoubat et al. 2001). Continuous restraint to shrinkage in high strength concrete induces large creep stresses at early ages. A lower water-to-binder ratio could be the cause of higher creep strains (Igarashi et al. 2000).

Cement composition, which governs the properties of cement paste, is one of the most important factors influencing the durability of concrete. Although, a large portion of concrete volume is filled with coarse aggregate, the cement paste is responsible for the overall performance of a given concrete. Cement technology has undergone significant changes during the last two decades. Today's cement contains more C_3S and less C_2S as compared with the old cements, which accelerates the strength gain. Ordinary Portland Cement (OPC) currently being used is much finer in specific surface than the cements of two decades ago. Further, today's cements are different from old cements with respect to concrete mix water demand. They consume less water, but produce a higher heat of hydration, due to their finer specific surface and higher C_3S content (Uzzafar 1992).

Microstructural changes occur in concretes exposed to severe environments. A broad range of analytical techniques is used to document the deterioration in concrete microstructure. With these analytical techniques deicing salt, freeze thaw effects, alkali silicate reaction (ASR), sulfate attack, and other physical causes of concrete deterioration can be identified. Also, Petrographic examinations can be used to determine the cause, depth, and extent of deterioration (Grattan-Bellew 1996).

2.4 PRECAUTIONS

Most of the researchers refer to ACI-207.2R, "Effect of restraint, volume change and reinforcement on cracking of mass concrete". ACI-207.2R examines the effects of heat generation and volume change on the design and behavior of reinforced concrete elements. It particularly focuses on the effects of restrained cracking, placement temperatures, concrete strength requirements, and fineness and type of cement. Likewise, ACI-224.R "Control of cracking in concrete structures" investigates causes of cracking in order to develop appropriate solutions for cracking problems.

2.4.1 Construction Technology

Construction procedure plays an important role in the durability of concrete and cracking of concrete members. Regardless of selected construction procedure, multi-purpose approaches could not be executed to solve problems with complex structures subjected to severe exposures (Brozzetti 2000). As an example, in the construction of the Confederation Bridge, which is in a severe marine environment, a 100-year service life was required. The quality concept was set to a great extent to produce concrete that has the ability to protect embedded steel from corrosion. Applied concrete technology, nondestructive material testing, high standards of quality control, and cooperation of consultants and construction companies allowed the achievement of this performance (Holley et al. 1999).

Available codes and standards, where strength requirements are the primary parameters for acceptance, are not adequate for achieving durability of concrete especially under harsh environmental conditions. The current system does not address the entire spectrum of concrete performance parameters, defining strength, durability, constructibility, and appearance. Local environments and material variations should also be considered to verify concrete performance (Shilstone and Shilstone 2002).

2.4.1.1 Slipforming

A commonly used construction technique for barriers is slipforming. Slipform construction requires specialized equipment, skill, and experience. The slipforming technique has some advantages with regard to speed of construction and disadvantages with regard to workability properties of the concrete mix. The limitations that are imposed on the mix proportions in the slipforming technique impacts concrete workability. For proper implementation of slipforming, special mixes with a well-defined cement content, workability, plasticizers, and optimum total aggregate gradation are required (Neville 1999). During the placement, environmental conditions are also important. Even if all precautions are taken, plastic shrinkage cracking cannot be controlled under windy and dry weather conditions. Proper environmental protection and curing procedures are needed for minimizing cracking potential of slipformed members; curing compounds are suitable for that purpose (Fu 1998). A newly implemented slipforming construction technique, which allows the application of curing compounds and membranes on time, is a cost effective and efficient solution for complex structures. Concreting speed, sliding operations, and curing should be reevaluated and enhanced with respect to concrete performance (Anguelov 1995 and RILEM-42-CEA 1981).

2.4.1.2 Curing of Concrete

Curing has a direct influence on drying shrinkage of concrete members. Type of curing is a factor that must be decided. RILEM-42-CEA (1991) recommends the use of an efficient membrane cover to be immediately placed over the newly placed concrete to prevent early evaporation in order to avoid shrinkage cracking. Early-age thermal cracking can be controlled by reducing the thermal movements or temperature differences. Selection of an appropriate curing process is the most important means of reducing the potential thermal movement by keeping temperature differences within the concrete section to a minimum (RILEM-42-CEA 1991).

Sealing of plastic concrete with a curing compound that forms an impervious membrane could reduce potential for drying shrinkage, but it will not eliminate autogenous shrinkage (Altoubat and Lange 2001). Silane treatment of concrete members in the field results in a decrease in drying shrinkage; this treatment can be considered in curing procedures (Xu and Chung 2000).

2.4.1.3 Quality Control

Quality of a finished construction project should be evaluated by a given structure's functional, physical, environmental, and economical utility (Abdun-Nur 1982). Establishment of a quality system for construction is important; specifications, control charts, and education should support the total quality of system. Variations in understanding of specifications and application differences in the field should be minimized to ensure that the job is done properly. Specifications must be clear and precise to avoid misunderstandings and to minimize effort in the field (Abdun-Nur 1982). Violations in construction procedure should be anticipated, and a designer's review should be implemented. Hiding a questionable condition could be hazardous for a structure and its serviceability. The potential problem points are important to review and should be recorded for further evaluation. For instance, the nuclear concrete industry has many reference documents for use in further evaluation of results (Mayer 1982). The size of the project should not influence the engineering effort and judgement. Obviously, small structures and nonstructural concrete members command less attention than other, more complex applications. The same attention should be paid to all structural elements in terms of construction and curing practices, regardless of size and importance (Jaycox 1982).

2.4.2 Materials

2.4.2.1 Cement

ACI 224-R (1990) states that cement properties have a direct effect on concrete shrinkage. Higher shrinkage of cement does not mean higher concrete shrinkage (Neville 1995). Finer cements generally cause increased shrinkage in concrete, but the increase in fineness is not proportional to shrinkage (ACI 224 1990). An advantage of coarse cement particles is that they give relatively less shrinkage as compared to finer cements; on the other hand, coarser cement requires a longer curing period to avoid development of low strength, larger pores, and high porosity (Bentz et al. 1999).

Cement has less shrinkage if it has lower C_3A/SO_3 ratios, lower alkalinity, and higher C_4AF contents. The choice of cement type is another means of reducing shrinkage. Type II cement has a tendency to less shrinkage when compared with Type I cement (ACI 224 1990).

Industrial by-products such as fly ash, silica fume, and ground granulated blast-furnace slag (GGBS) are mixed with Portland cement in different proportions to develop desirable concrete properties, sometimes in plastic state, but more often in the hardened state. Neville (1995) states that addition of minerals such as fly ash, GGBS, or silica fume can increase concrete shrinkage. Specifically, a higher proportion of a mineral admixture in blended cements leads to higher shrinkage.

The most common cement replacement is by the use of fly ash. The use of fly ash (especially Class F) in concrete has many different advantages such as an increase in long-term strength and reduction in permeability. Fly ash reduces the peak concrete temperature due to heat of cement hydration. Addition of fly ash reduces water demand for constant workability. High carbon content of fly ash affects the workability. Porous carbon particles absorb certain types of air-entraining agents, thus reducing its effectiveness. The initial setting of concrete is delayed due to the retarding effect of fly ash, thus requires longer curing period. The effects of inadequate curing on the water absorption properties are more profound than the effect on the strength of concrete containing fly ash.

Silica fume is used in small amounts (3 – 6 %) as an ordinary Portland cement substitute. Use of silica fume in concrete mix causes high early-strength and low permeability but high heat of hydration. Also causes problems in developing an acceptable air-void system. Curing is very important when silica fume is used. Silica fume reduces bleeding of mix water which can lead to plastic shrinkage under drying conditions unless preventive measures are taken (Neville 1995). The hydration reaction of silica fume has a heat sensitivity that is different from cement hydration (Jensen and Hansen 1999). Silica fume substitution often results in better performance with reduced permeability in severe environments (Sabir 1997).

GGBS is another substitute for ordinary Portland cement. GGBS reduces bleeding of mix water from concrete and retards the hydration process reducing the heat of hydration of cement. GGBS leads to better strength development at the later ages (due to very slow initial hydration). Concrete containing GGBS shows improved durability performance due to reduced water permeability. Since GGBS reduces bleeding of mix water, precautions are needed to prevent plastic shrinkage under drying conditions. In concrete placement at temperatures below 50^{o}F the strength development is poor and the use of GGBS is undesirable (Neville 1995).

Pozzolans can also be used in the formulation of blended cement. They are added to the cement during production. This type of blended cement production is common in Europe and Asia. A number of disadvantages of ordinary Portland cement can be overcome by using additional pozzolans such as fly ash, GGBS, and silica fume. Blended cements give users a chance to enhance cement performance when considering the severe environments to which concrete will be subjected (Malhotra and Hemmings 1995 and Nehdi 2001). Swamy (1989) states that the use of appropriate blended cement and high range water reducer is one of the options for design of durable concrete in severe environments.

2.4.2.2 Concrete

A study on the key parameters controlling concrete shrinkage and the effect of shrinkage on performance based on the distress evaluations of field specimens illustrated that the size of the concrete member significantly affects shrinkage (Bissonnette et al. 1999). Shrinkage decreases with relative humidity in the range of 48–100%. Cement paste volume has a direct effect on amount of shrinkage. Shrinkage compensating concrete mixes are preferred (Altoubat and Lange 2001).

Performance of high strength concrete under different exposure and environmental conditions was examined by the Florida Department of Transportation (Edwards 2000). The severity of exposure is the main criterion in selecting the type of concrete mix design. In moderate and aggressive exposure classes, the use of pozzolans (fly ash, GGBS, and silica fume) in high strength concretes with Type II cement is recommended to reduce chloride ingress. Additionally, construction practices should be tuned to the required construction quality. It is necessary to test all-purpose mixes and the test results have to be interpreted in the light of the exposure conditions (Neville 1995)

2.4.2.3 Water / Cement Ratio

ACI 224 (1990) states that one of the major factors affecting concrete performance is the water content of the mix design. The U.S. Bureau of Reclamation test results showed that any increase in the water content results in an increase in shrinkage. Reduction in concrete shrinkage is made possible by keeping the water content to a minimum and the total aggregate content as high as possible. On the other hand, some recent research findings show that lowering the water content can increase concrete shrinkage (Igarashi et al. 2000, Samman et al. 1996, and Wiegrink et al. 1996). There is a need to better understand the role of water in concrete shrinkage. The optimum water/cement ratio range for shrinkage minimization is between 0.35 and 0.50 (Bissonnette et al. 1999).

2.4.2.4 Admixtures

Gillot and Gillot (1996) studied the durability performance of ground granulated blast-furnace slag cement concrete with respect to freeze-thaw resistance, microstructure, and curing conditions. It could be that the durability of both ordinary cement and ground granulated blast-furnace slag cement is due to air entrainment in mix design. In selecting the admixture(s) for concrete compatibility to construction practices and serviceability of the structure should be considered. Retarding agents and plasticizing admixtures can be used for addressing compatibility concerns to construction practice, but further research may be needed with regard to durability concerns (Ronneberg 1989). Clearly, retarders delay the hydration process subsequently the strength gain in concrete. If moisture evaporation from concrete is not prevented during the hydration process, very early-age cracks will form.

Concrete mix production process and concrete properties are important considerations when selecting the type of air entrainment. Concrete properties can be controlled by the amount of air entrainment agent, but the amount of active air entrainment agent and its reaction with the water reducing agent is important. Another important factor in achieving a specified air entrainment amount is the cement content and its physical and chemical composition. Lower cement content and/or a coarser grind of cement results in better air entrainment. The concrete production process could change the amount and stability of air entrainment. An increased amount of air entrainment will reduce the required fine aggregate content needed to maintain the same workability. Mixing time, mixing and transportation period, and placement are other major factors that should be considered when using air entrainment agents (Rixom and Mailvaganam 1999).

High range water reducers, under hot and dry field and laboratory conditions, increase the long term and early-age shrinkage (Alsayed, 1998). Concrete curing has an important role in concrete shrinkage rate. Delaying shrinkage reduces cracking by allowing concrete to gain a higher tensile strength.

2.5 CONCLUSIONS

The primary distress types identified on bridge barriers are vertical cracking (termed as transverse cracking), map cracking, horizontal cracking, popouts, spalling or disintegration, efflorescence, corrosion, and delamination. Vertical cracking that is observed at early ages is often the predominant type of distress in a given barrier segment. Map cracking is also a common defect observed on barrier surfaces. Map cracking on barriers may be related to early thermal shrinkage cracking due to hydration of cement. Other distresses are the signs of progression of distress initiated by vertical cracking.

Fully base-restrained unreinforced concrete walls ultimately attain full-length vertical cracks spaced at one to two times the height of the wall. Upon concrete placement, due to the effect of water rising to the top and sedimentation under reinforcing bars, a porous layer of concrete can form under the reinforcement and its ribs. Additionally, consolidation of plastic concrete causes relative downward movements (settlement) of the concrete around the top reinforcement. This process forms a conduit under the top horizontal reinforcement. Water and other contaminants penetrate through the vertical cracks and propagate in the conduits formed below the top reinforcement, initiating corrosion along the full length of reinforcement. Recent research revealed that epoxy coating debonds with moisture and reinforcement is still susceptible to corrosion. Corrosion of reinforcement generates internal outward pressure in concrete and leads to further cracking. Corrosion does not remain limited to the initiation zone and propagates, especially in cyclic wetting-drying areas of structures. Upon cracking, further corrosion also causes spalling of concrete cover. Spalling and disintegration are characteristics of the final stages of distress in concrete structures that requires immediate action for repair and rehabilitation.

Major causes of early-age cracking are identified as shrinkage, thermal loads, and base restraint of the concrete barrier. Volume change in concrete is mainly due to autogenous shrinkage, drying shrinkage, and thermal loads. Cracking due to volume change is a result of the combined effects of volume change strains and restraint. When strain due to volume change of a concrete component exceeds the concrete strain capacity accompanied by sufficient stress due to restraint effects, concrete cracks.

A major factor that reduces the service life of a concrete structure is construction quality. Shallow concrete covers, misplaced reinforcement, substandard curing, and other construction errors may be seen in many projects. The impact of construction errors on service life is a function of exposure conditions. The environment has a significant influence on the properties of fresh and hardened concrete. Ambient temperature, relative humidity, and wind velocity can change the properties of concrete. The amount of water required for the hydration process needs to be available if the concrete is to achieve a fully acceptable quality. The effects of substandard curing are only observed with increasing age of concrete. Use of an impervious membrane cover over the newly placed concrete is required to prevent early evaporation and therefore avoid shrinkage cracking. Early-age thermal cracking can be controlled by reducing the thermal movements or temperature differences. Selection of an appropriate curing process is the most important means of reducing the potential thermal movement by keeping temperature differences within the concrete section to a minimum. Silane treatment of concrete members in the field results in a decrease in drying shrinkage; this treatment should be considered in curing practices.

When construction procedures are considered, early form removal without further curing affect concrete durability performance. Substandard curing such as late or uneven application of curing compound exposes the slipformed barrier concrete to the ambient environment immediately. Without moist curing, early overloading by thermal and shrinkage loads combined with restraint effects can cause permanent damage by forming cracks. In addition to shrinkage and thermal cracking, bleeding of mix water causes consolidation of newly placed concrete during the first two or three hours, leading to premature cracking around the top horizontal reinforcement due to the restrain provided by the reinforcement.

The parameters controlling shrinkage are the strength and elasticity modulus of coarse aggregate and type and content of cementitous material. For a given aggregate and cement, cement composition determines concrete properties such as strength, durability, and stability. The aggregate acts as a passive filler material, if unreactive, while the cement paste is responsible for the good and bad properties of a given concrete.

Lower cement content and coarser cement result in better air entrainment. Mixing time, mixing and transportation period, and placement are the other major factors that should be considered when using air entrainment agents. Cement properties have a direct effect on concrete shrinkage and heat of hydration. Finer cements generally increase shrinkage and generate higher heat of hydration in concrete than coarser cements. Coarser cements require a longer curing period to avoid development of larger pores and high porosity.

Ground granulated blast-furnace slag substitution in an amount approximately equal to 50-55% of cement weight could significantly reduce the heat of hydration. Blended cements give users a chance to enhance cement performance considering the severe environments to which concrete will be subjected. The optimum water/cement ratio range for minimizing shrinkage is between 0.35 and 0.50.

The quality concept is the key to improve the concrete performance that has the ability to protect embedded steel from corrosion. Applied concrete technology, nondestructive material testing for quality control and quality assurance, and cooperation of consultants and construction companies is necessary for achievement of required performance. Specifications must be clear and precise to avoid misunderstandings. Potential violations in construction procedures should be anticipated. The same attention should be paid to all structural components in terms of construction, quality, and curing practices, regardless of size and perceived importance.

3 MULTI-STATE SURVEY

3.1 OVERVIEW

The Project Team (PT) and MDOT Research Advisory Panel (RAP) members designed and administered a nation wide survey for documenting the experience of the State Highway Agencies (SHAs) with the problem of cracking of concrete bridge barriers. The survey was sent to SHAs and the District of Columbia in September, 2002. Twenty-six SHAs responded to the survey, giving a response rate of 50 percent. The survey was administered by email, requesting a web submission. There was more than one response from some of the SHAs. The responses were received through email, fax and postal mail. The list of respondents is shown in Table 3-1. The geographic locations of SHAs are shown in the U.S. map in Figure 3-1. The final survey is included in Appendix A.

Table 3-1. List of Respondent States and Media of Responses

Media of Responses	Respondent States
Web Submission	Connecticut, District of Columbia, Florida, Hawaii, Idaho, Indiana, Maryland, Massachusetts, Michigan, Minnesota, Missouri, Montana, Nebraska, North Dakota, New Hampshire, New Mexico, Nevada, New York, Tennessee, Texas, Vermont, Virginia, Washington
Fax	Alabama, Illinois
Postal Mail	New Jersey

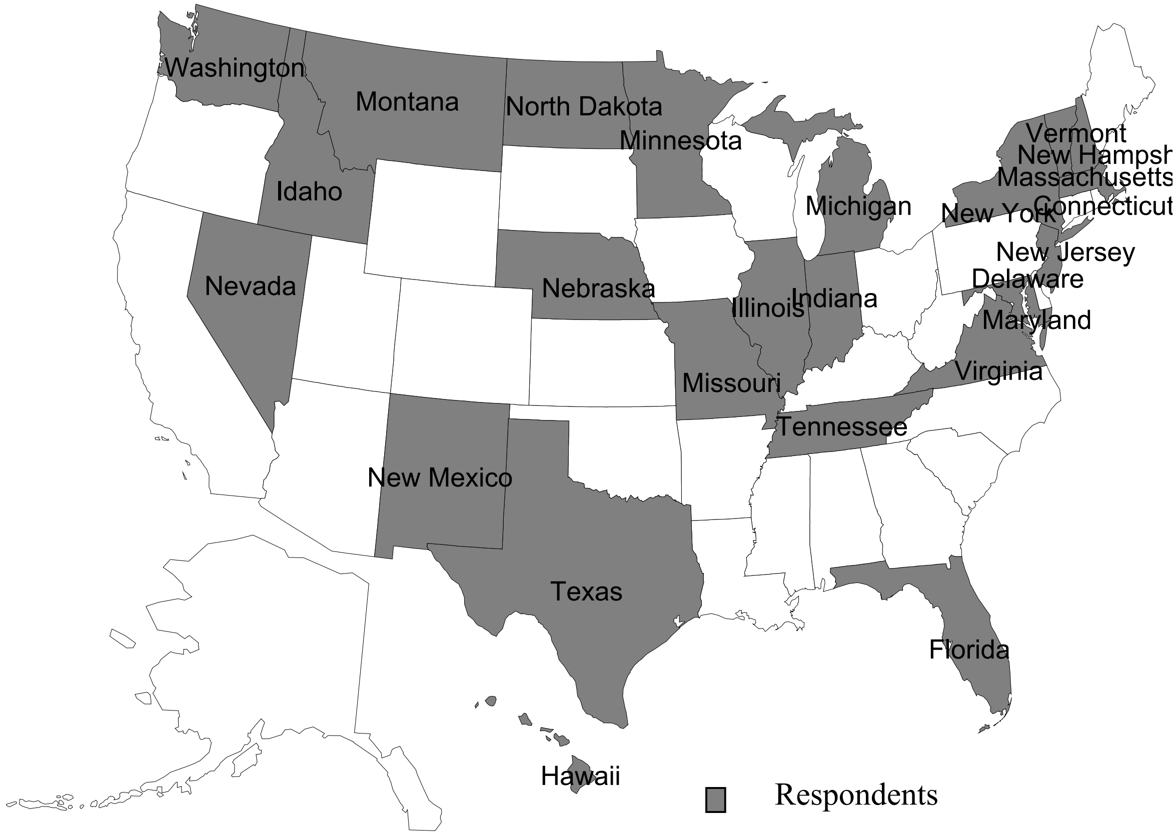


Figure 3-1. Geographic location of State Highway Agencies responded to the survey

3.2 ANALYSIS OF SURVEY DATA

The survey data was compiled and analyzed. The number of responses obtained from some SHAs is more than one. Some of the responses to the same question on the survey questionnaire vary by respondent within the particular SHA. For that reason, the count of responses represents the number of respondents rather than the number of SHAs. Summaries of the responses are presented in the following figures. The codes used to represent the responses are given in Table 3-2

Table 3-2. Codes used for Presentation of Survey Responses

Codes Used	Meaning of the Codes
1	Yes
DN	Don’t know
NR	No response
None	Don't have, did not observe

The first question of the survey questionnaire asked the frequencies (high, medium, low) of various distresses (map cracking, horizontal cracking near joints, etc.,). The respondents identified the frequency of the distresses and those are summarized in the following figures (Figure 3-2 through Figure 3-11).

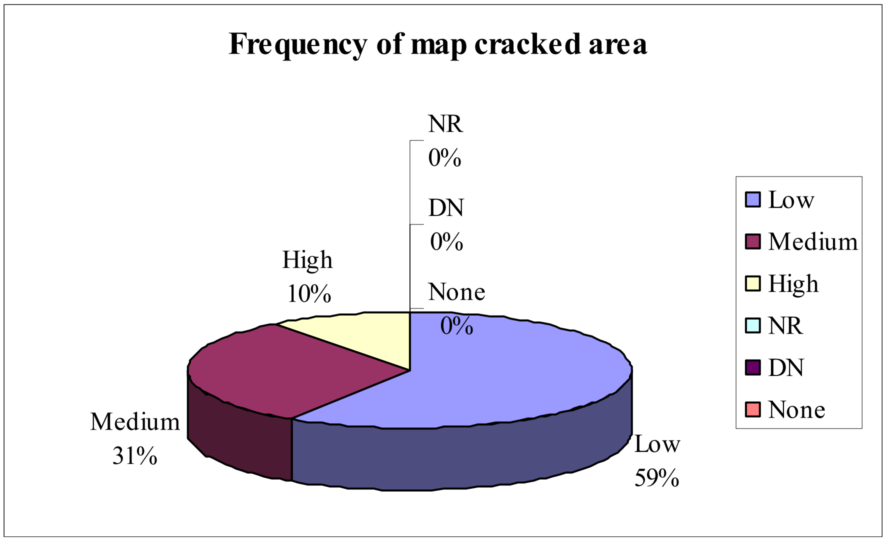


Figure 3-2. Frequency of map cracks observed on barriers

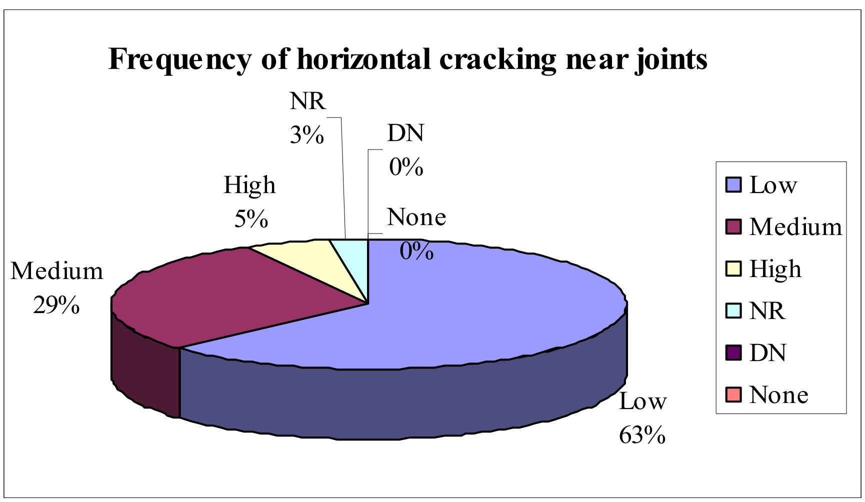


Figure 3-3. Frequency of horizontal cracking observed near joints of barriers

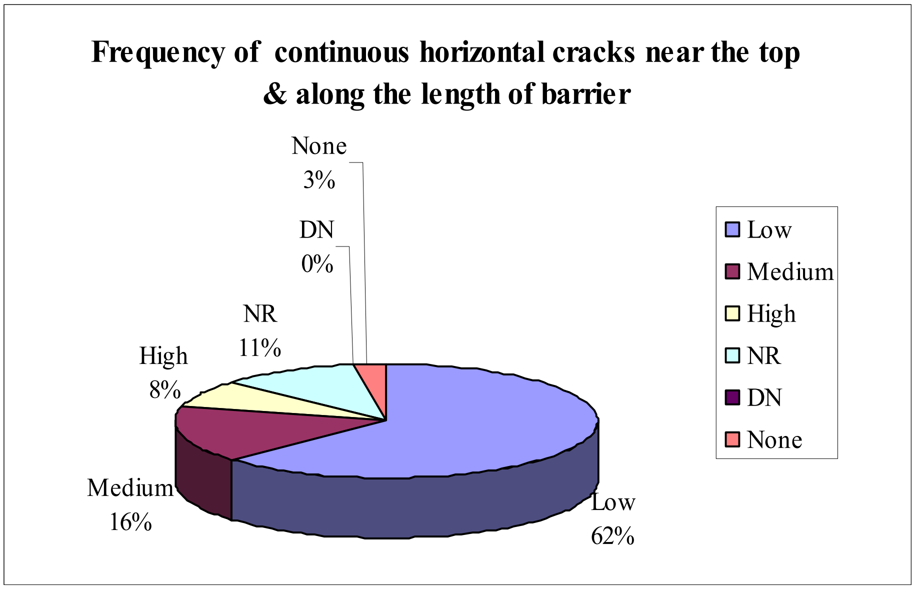


Figure 3-4. Frequency of observed continuous horizontal cracks near the top and along the length of barrier

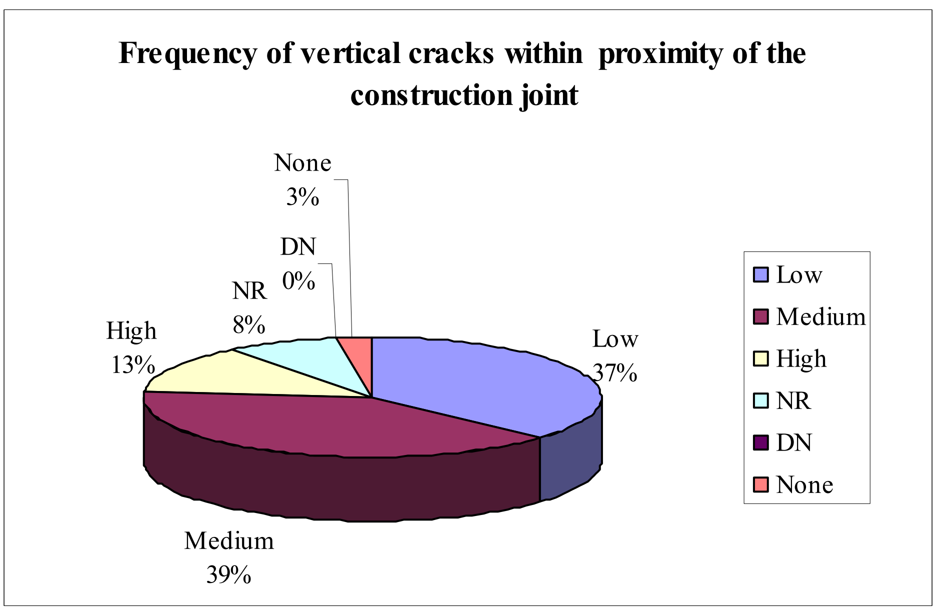


Figure 3-5. Frequency of observed vertical cracks within proximity of the construction joint

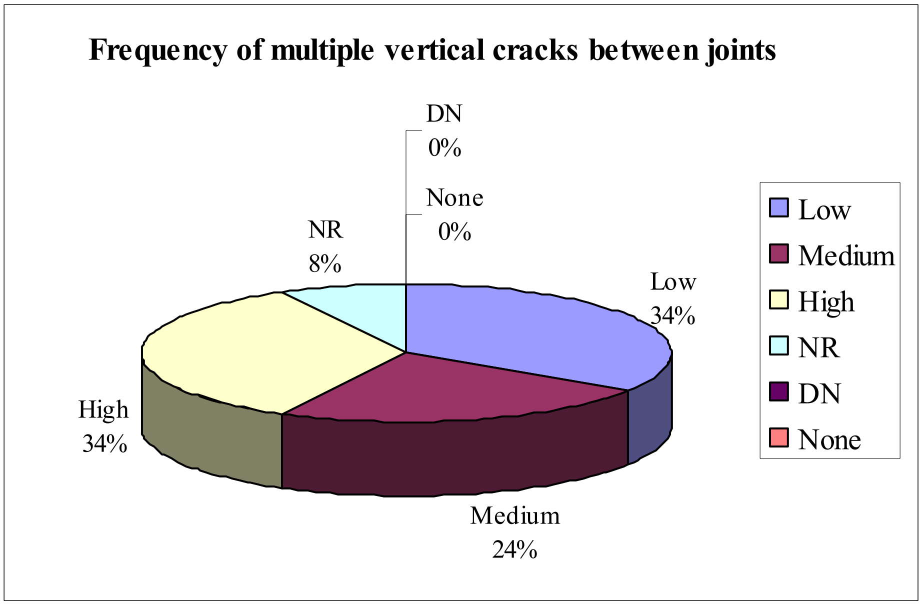


Figure 3-6. Frequency of observed multiple vertical cracks between barrier joints

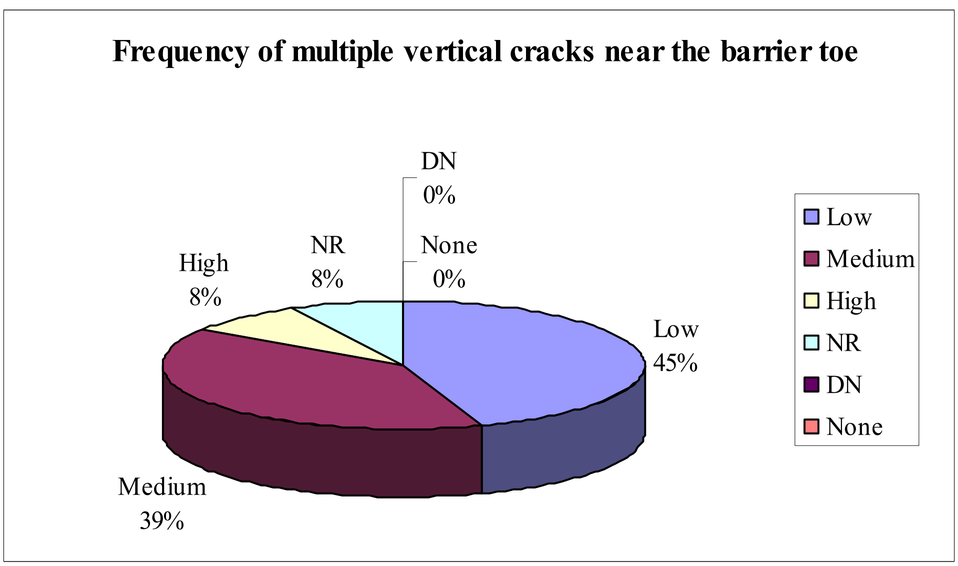


Figure 3-7. Frequency of observed multiple vertical cracks near the barrier toe

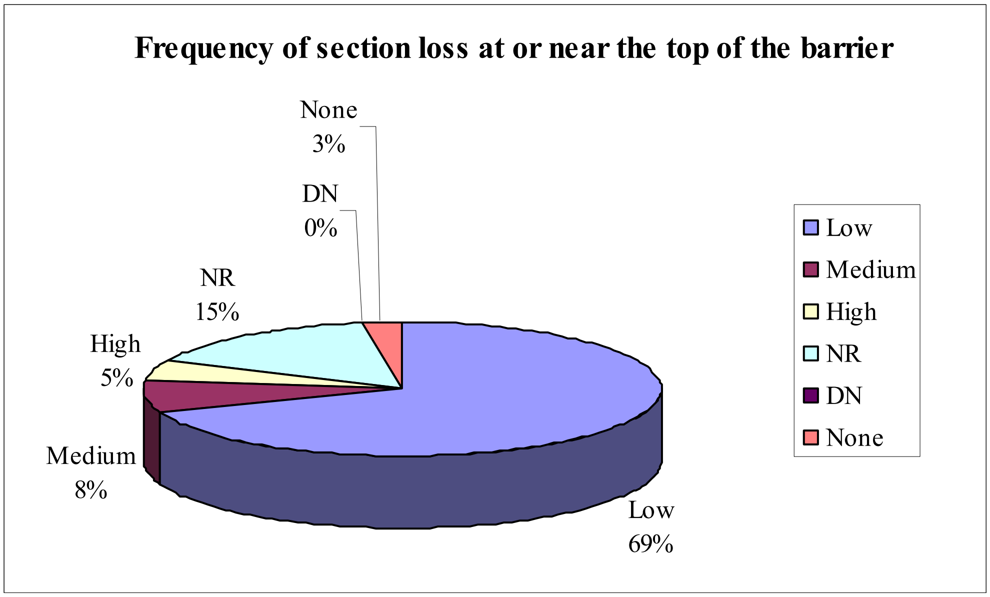


Figure 3-8. Frequency of section loss observed at or near the top of the barrier

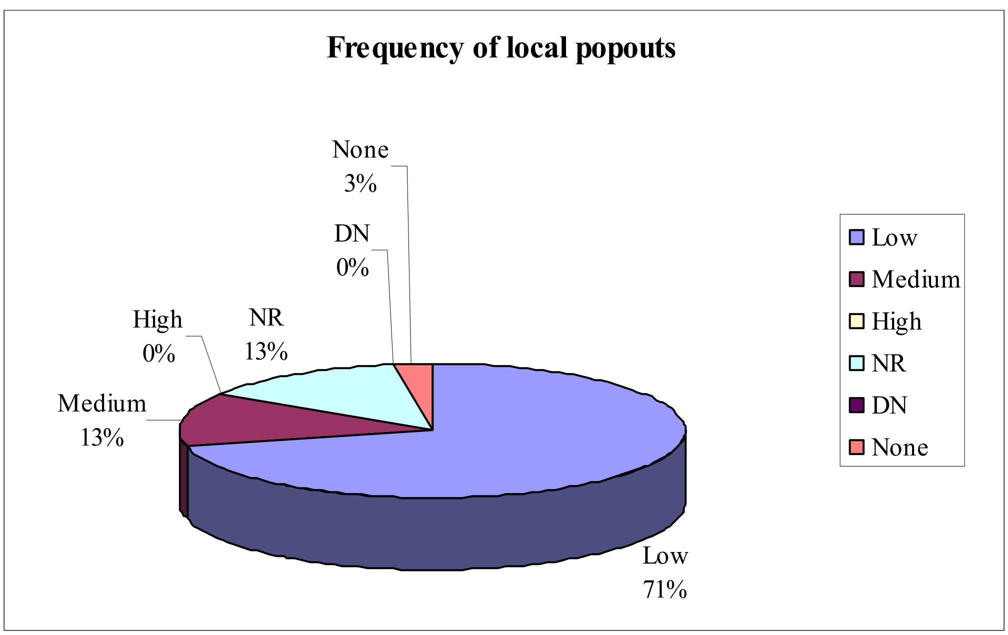


Figure 3-9. Frequency of local pop outs observed

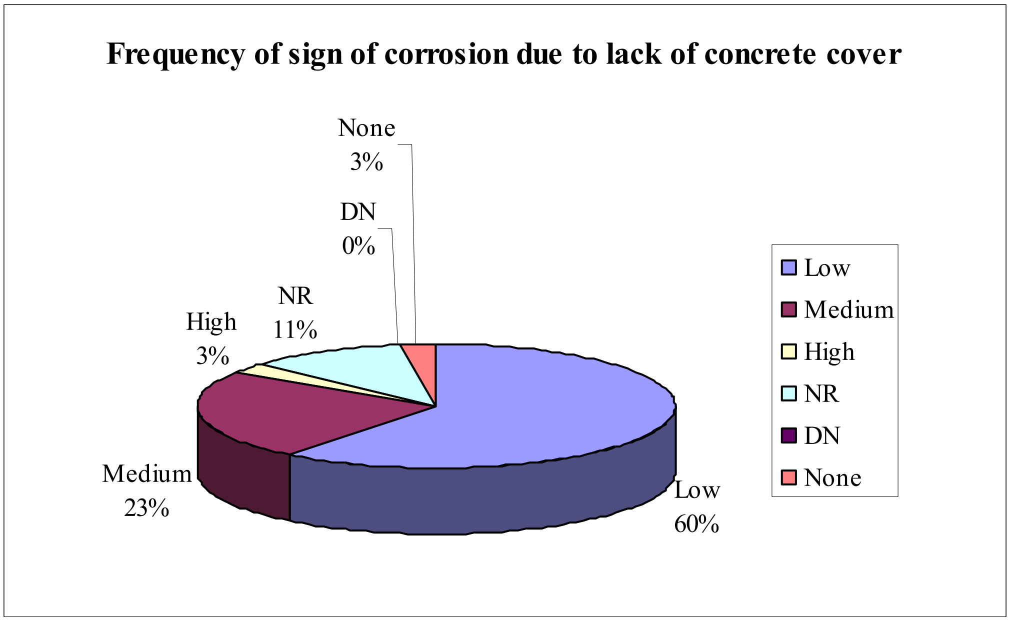


Figure 3-10. Frequency of sign of corrosion observed on bridge barriers

From the responses, the order of prevailing distresses observed on bridge barriers is:

1. Map cracks
2. Horizontal cracking near joints
3. Multiple vertical cracks between joints
4. Multiple vertical cracks near the barrier toe
5. Vertical cracks within the proximity of the construction joints
6. Continuous horizontal cracks near the top and along the length of barrier
7. Signs of corrosion due to lack of concrete cover
8. Local pop out
9. Section loss at or near the top of the barrier

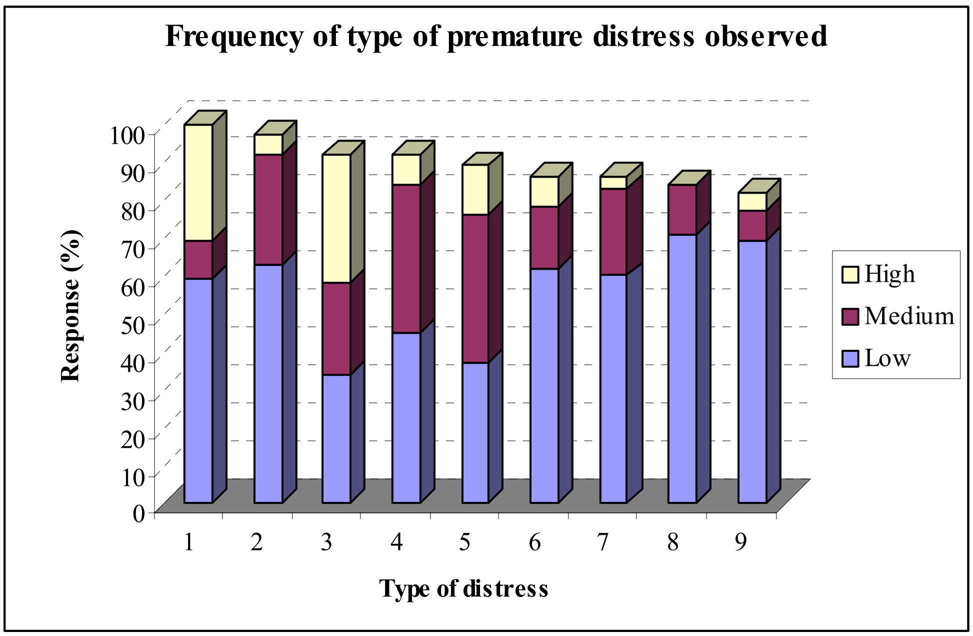


Figure 3-11. Frequency of type of premature distress observed on bridge barriers

A total of 23 (73%) out of 26 SHAs indicated that they do not have an overall durability problem with New Jersey Type 4 bridge barriers. Three SHAs (24%) indicated durability concerns (Figure 3-12)

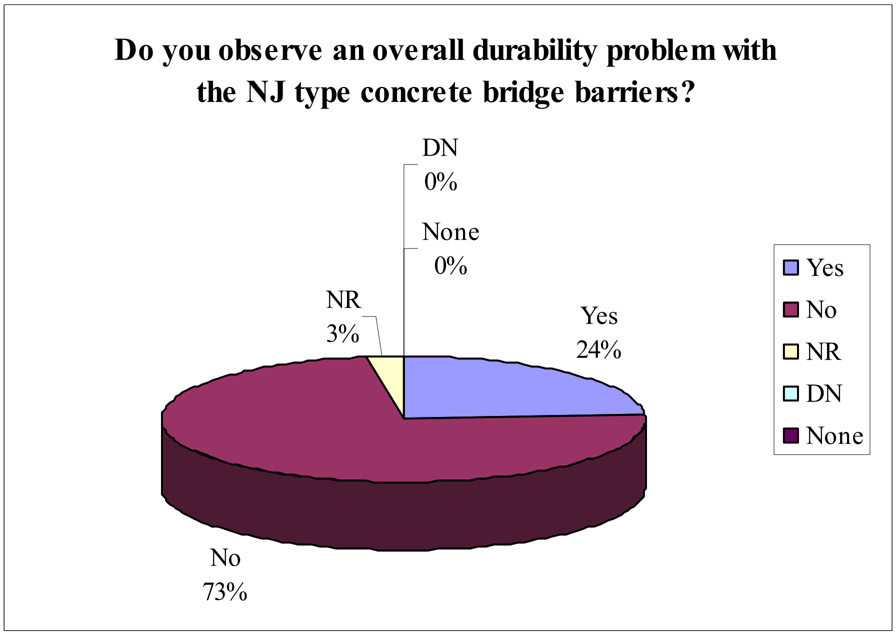


Figure 3-12. Frequency of observed overall durability problems with NJ type barriers

Form-cast and slipformed are the two predominant barrier construction procedures. A few SHAs also use precast barriers. The percentage of usage of different construction procedures among the respondents is shown in Figure 3-13.

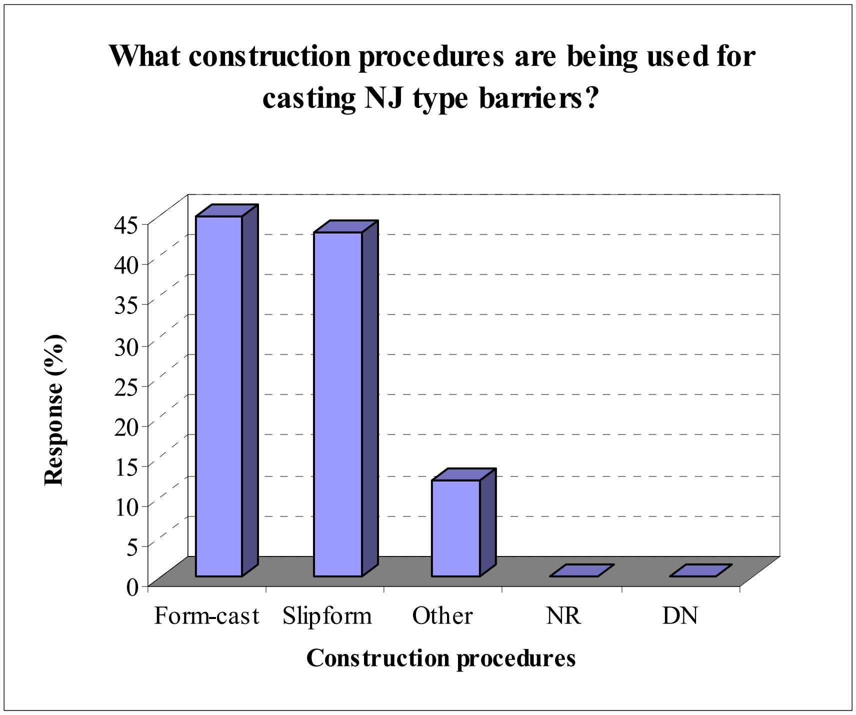


Figure 3-13. Most popular construction procedures used for casting New Jersey Type 4 barriers

The most frequently specified curing procedure for bridge barriers are (Figure 3-14).

1. Burlap
2. Membrane curing
3. Continuous wet curing
4. Other (curing by wet burlene sheets, blankets).

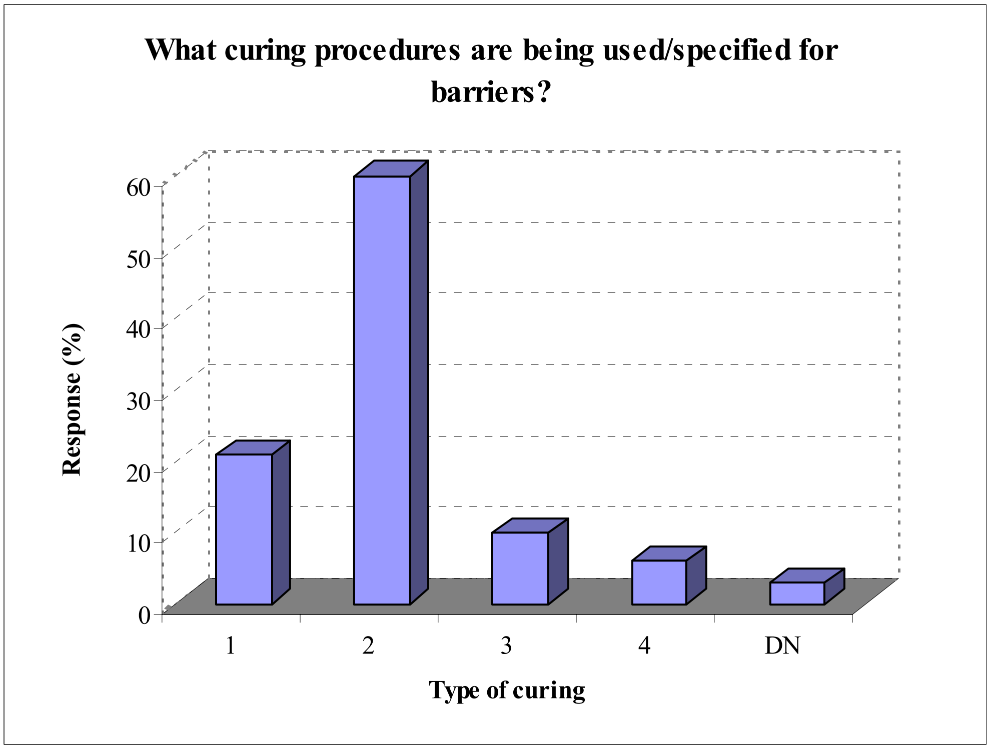


Figure 3-14. Curing procedures used /specified for barriers

Most of respondents (82%) have been using epoxy-coated reinforcements for the barriers (Figure 3-15).

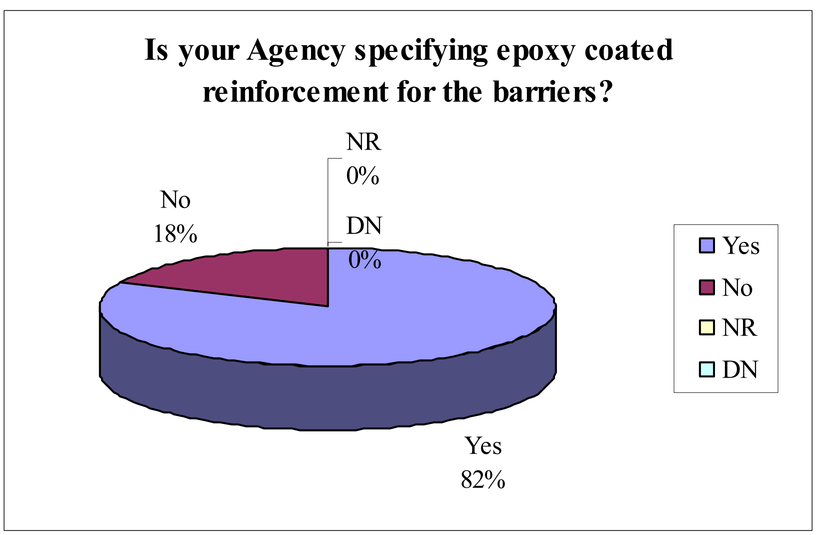


Figure 3-15. Frequency of using epoxy coated reinforcements for the barriers

Fifty-five percent of the respondents reported that coatings/sealants are currently specified for barriers (Figure 3-16).

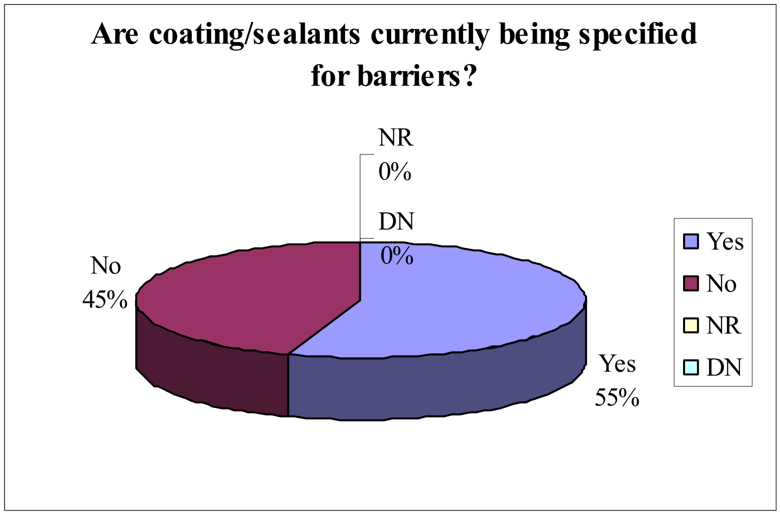


Figure 3-16. Use of coatings/sealants for bridge barriers

Forty-eight percent of the respondents indicated that the same concrete mix is specified for both the deck and the barrier (Figure 3-17).

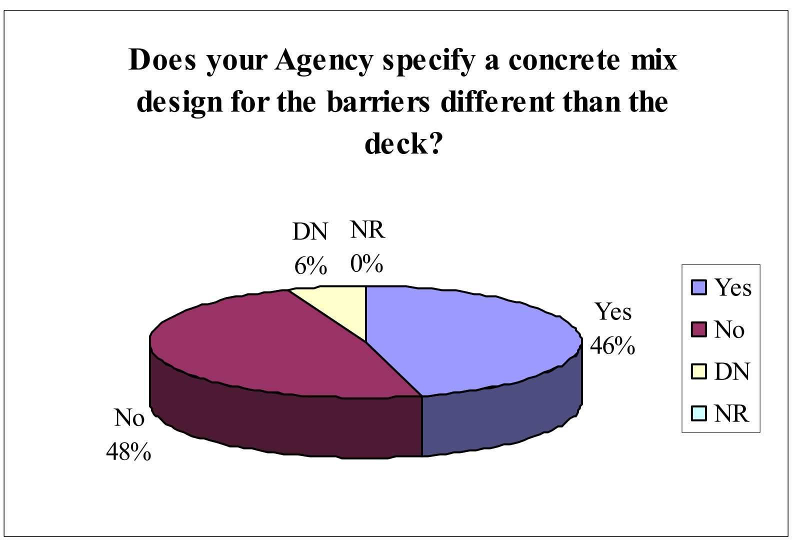


Figure 3-17. Use of different concrete mix design for bridge barriers and decks

Fifteen (58%) respondents indicated that different types of pozzolans are specified in the mix in order to reduce the permeability of the barrier concrete. Most frequently used pozzolans are:

1. Ground Granulated Blast-Furnace Slag (GGBFS)
2. Fly Ash (FA)
3. Silica Fume (SF)

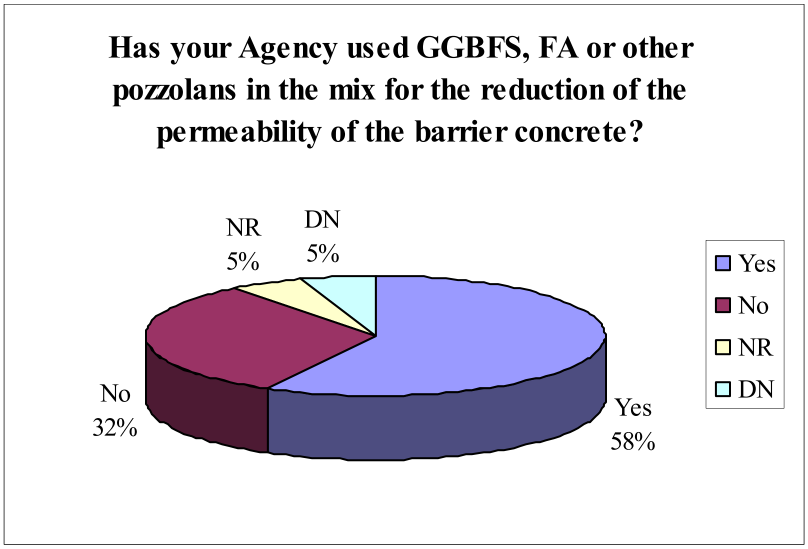


Figure 3-18. Use of different types of pozzolans for barrier concrete

The percentages of usage of different types of pozzolans among the respondents are given in the following Figure 3-19.

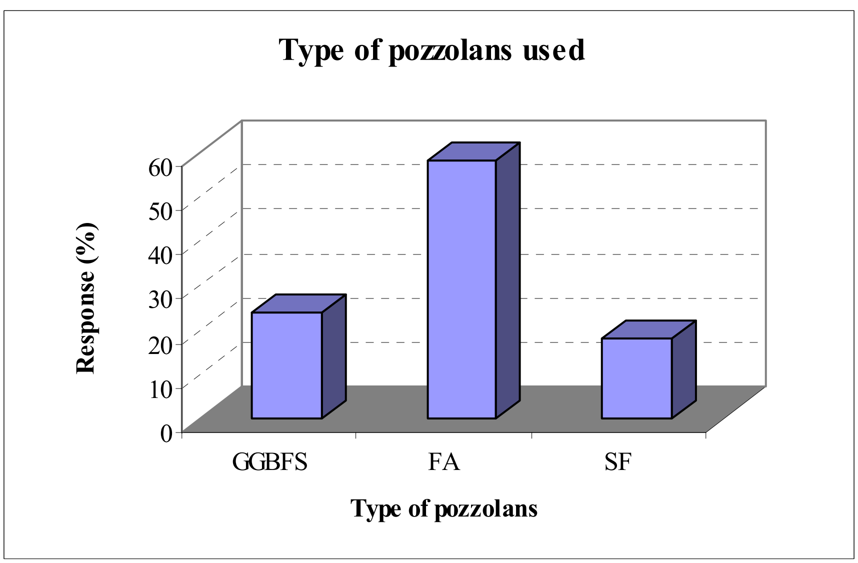


Figure 3-19. The percentage of usage of different types of pozzolans

The most frequently specified methods for barrier concrete surface finishing are:

1. Rubbed
2. Sacked

Other methods stated by the respondents are light brush or broom finishing, grind fins, sprayed coatings, and formed finishing (Figure 3-20).

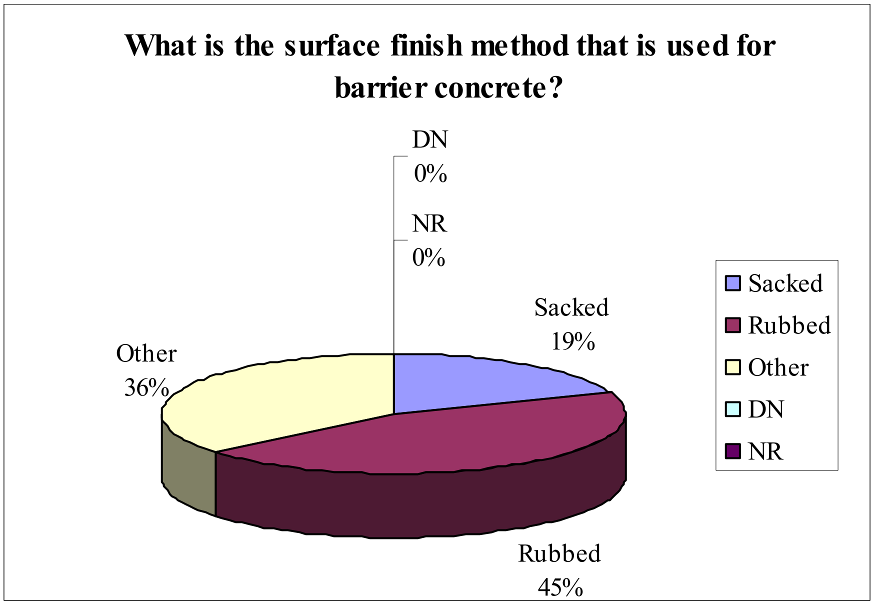


Figure 3-20. Surface finish methods used for barrier concrete

The most identifiable feature is that 64% of respondents did not reply to the question regarding the performance of barriers on rural roads versus trunk line/interstate routes. Twenty-six percent of the respondents indicated that performance differences are observed between barriers on rural roads and interstate routes (Figure 3-21).

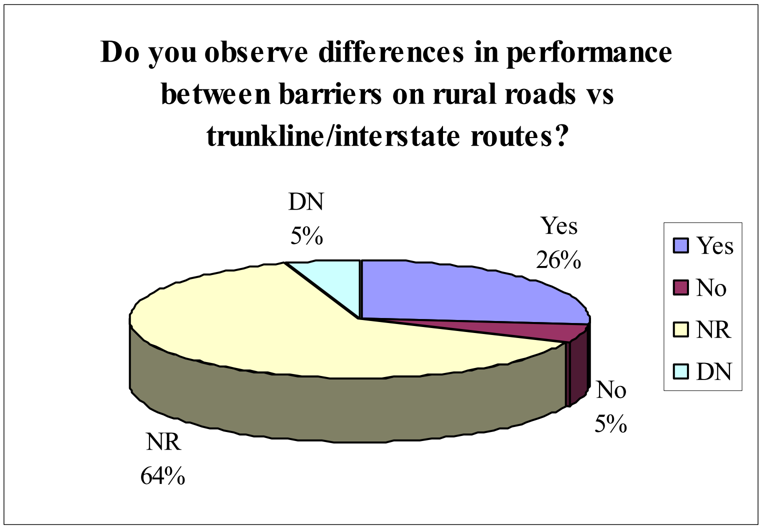


Figure 3-21. Performance differences of barriers on rural roads and trunk line/interstate routes

Many of the SHAs (85%) apply deicers to the bridge deck. Exceptions are Florida, Hawaii, New Jersey, and Nevada. Commonly used deicers are:

1. Different types of salt
2. CMA (Calcium Magnesium Acetate)
3. Others (Urea, heavy salt brine, etc)

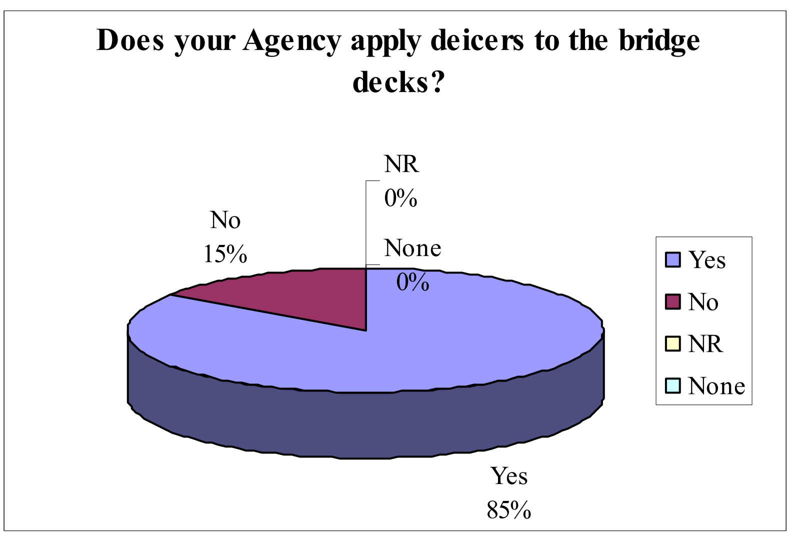


Figure 3-22. Application of deicers to the bridge decks

The percentage of usage of different deicers among the respondents is shown in Figure 3-23.

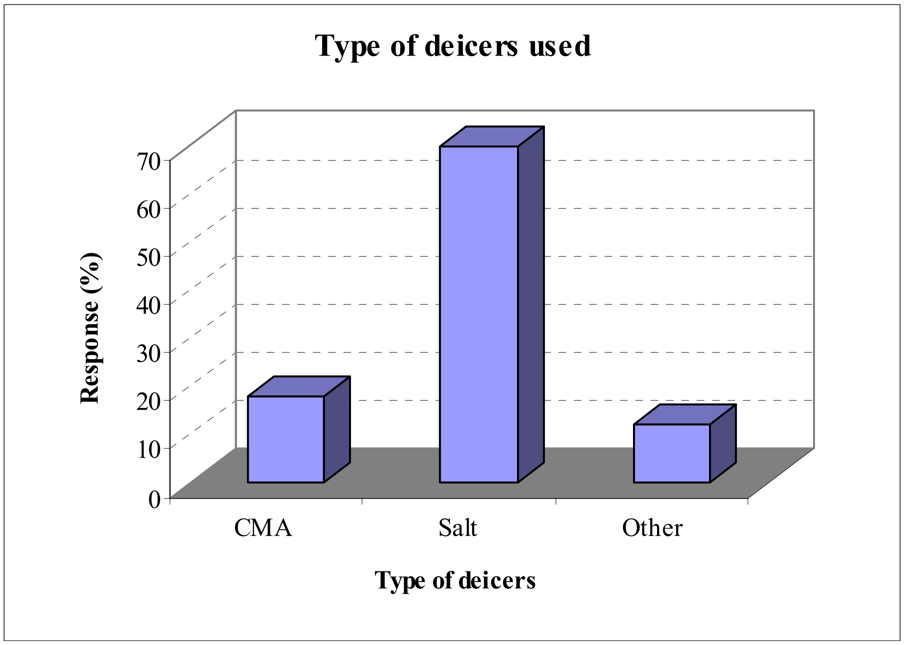


Figure 3-23. The percentage of usage of different deicers

Actions taken by the State Highway Agencies in order to improve the barrier durability were reported as:

1. Changes to mix design
2. Changes to curing procedure

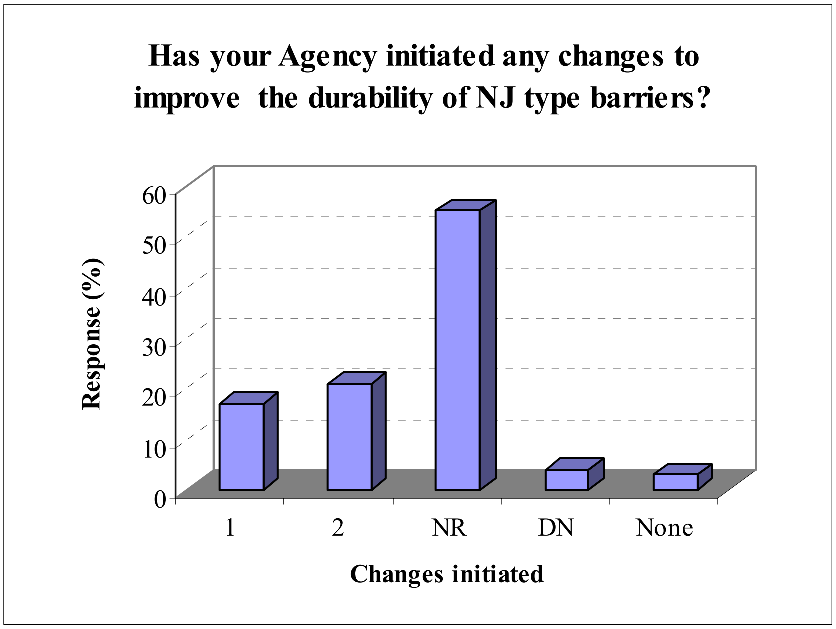


Figure 3-24. Changes made by the State Highway Agencies to improve the durability of New Jersey Type 4 barriers

3.3 CONCLUSIONS

The distresses observed by other State Highway Agencies (SHAs) are similar to those experienced by MDOT. Though all the respondents identified distress similar to MDOT, only Illinois, New Mexico, Vermont, and Virginia acknowledged that they have an overall durability problem with the bridge barriers. Most of SHAs have been form-casting or slipforming the barriers like MDOT. Florida, Maryland, New Hampshire, New York, Texas, Vermont, and Virginia have also been using precast New Jersey Type barriers in addition to the form-cast and slipformed barriers. MDOT's curing practice is the application of curing compound on the slipformed barrier surface. This is the most often specified curing procedure by the respondents. Most of the SHAs have been using epoxy-coated reinforcements like MDOT with the exception of Alabama, Florida, Hawaii, and New Hampshire. Sealants are not currently specified for barriers by Connecticut, Hawaii, Idaho, Maryland, Massachusetts, Minnesota, Missouri, New Hampshire, New Jersey, and Vermont as they are by MDOT. Unlike MDOT, Illinois, Minnesota, New Hampshire, New Mexico, New York, and Washington have been using a modified mix design for the barrier concrete different from the deck. Most of the respondents permitted the use of GGBS and FA in the mix design for the reduction of the permeability of concrete. Massachusetts, New York, and Virginia have been also using SF along with GGBS and FA. MDOT specifies the rubbed surface finish for barrier concrete. This is the common practice of surface finishing among the respondents excluding Alabama, Idaho, Nevada, North Dakota, Virginia, and Washington. Most of the respondents emphasized modifying the mix design and curing procedure for improving the durability of barriers.

4 FIELD INSPECTION

4.1 OVERVIEW

Twenty bridges with New Jersey barriers were identified for inspection. The barriers were categorized into four age groups starting with those constructed during 2001; each age group spans a 5-year period. Hence, the oldest barriers selected for inspection were constructed in the 1980's. The list of barriers inspected along with other inventory information, and the inspection date is shown in Table 4-1.

Table 4-1. Bridges Selected for Barrier Inspection

No	Bridge ID	Const. Year	Location	Average Daily Traffic (% ADTT)	Rail Type[1]	Post Type[2]	Inspection date
1	S02 - 23152	1980	I-96 WB over M-43	22,000 (14)	8	0	08/03/02
2	S01 - 44044	1983	I-69 EB over Clark road	8500 (19)	8	0	09/03/02
3	B02 - 66051	1985	M-26 over Fire Steel river-EB	700 (19)	8	0	09/30/02
4	B01 - 66051	1985	M-26 over Fire Steel river-WB	1600 (19)	8	0	09/30/02
5	S02 - 82194	1986	I-75 NB over Outer drive	42,500 (15)	6	5	06/12/02
6	S04 - 63101	1988	I-696 over Drake road	131,000 (7)	8	0	09/20/02
7	S09 - 63101	1988	I-696 WB over Inkster road	67,000 (7)	8	0	10/23/02
8	S12 - 63172	1988	I-75 over Clintonville road	34,500 (11)	8	4	09/03/02
9	S15 - 63172	1988	I-75 over Clarkston road	29,000 (13)	8	0	09/04/02
10	S08 - 82191	1989	King road over I-75	1971 (10)	8	3	10/25/02
11	S04 - 82022	1993	I-94 over Merriman road	52,000 (6)	8	0	08/30/02
12	S06 - 82022	1993	I-94 over Middle Belt road	54,500 (6)	8	0	08/30/02
13	B01 - 50021	1994	M-59 over Clinton river	37,000 (10)	8	0	10/30/02
14	S24 - 82022	1996	I-94 WB over Outer drive	44,500 (6)	8	0	10/25/02
15	S26 - 82022	1997	I-94 WB over Oakwood Blvd.	58,000 (5)	8	0	10/25/02
16	S12 - 63022	1997	M-102 over Farmington road	12,500 (4)	8	0	09/12/02
17	S28 - 41064	1997	M-6 WB over M-37	14,900 (6)	8	1	09/06/02
18	S27 - 41064	1997	M-6 EB over M-37	16,500 (0)	8	1	09/06/02
19	S04 - 63174	2001	I-75 over 13 mile road	93,500 (7)	6	5	08/30/02
20	S20 - 63174	2001	I-75 NB over Auburn road	62,000 (10)	6	5	08/30/02

1 8 = New Jersey, 6 = open parapet rail

2 0 = unknown, 1 = Type 1, 3 = Type 2, 4 = Type 3, 5 = thrie beam ahead of post

4.2 DISTRESS DOCUMENTATION

Bridge barriers were selected using "Pontis" based on inspectability, sampling ability without lane closure, and barrier age. As a general concept, the geographical distribution of bridges was kept as uniform as possible. Pontis data was not fully accurate and final inspection decisions could only be made after a preliminary site visit. The final list of barriers inspected includes two bridges from the Superior Region, two from the Grand Region, one from the Bay Region, one from the University Region, and the remaining from the Metro Region.

The barrier segment lengths varied between bridges. Barrier segments were divided into quarters for inspection purposes and observations were recorded on the respective inspection templates. Some segments that were very long were also inspected and recorded in quarters. The inspection sheets were prepared for each quarter as if each were a separate segment. Inspectors reviewed the work of one another upon completing the inspection of their assigned segment quarters. Use of this approach ensured the consistency and the accuracy of the inspection sheet.

All visible defects on the barrier segments were recorded. However, the predominant distresses were vertical cracking, horizontal cracking, map cracking, popout, scaling, delamination, spalling, and disintegration. Other observations such as patches, replacements, and repairs were noted.

4.3 COLLECTION OF SPECIMENS

Cores were collected from eight out of twenty inspected bridges in order to investigate the material related distress as well as to establish the concrete properties. The purpose was to document the barrier distress type, state, and extent. An average of 12 core specimens were obtained from each barrier. Half of these core specimens were obtained from distress-free parts of the barriers and the other half was obtained from areas close to the visible distress zones.

4.4 FIELD INSPECTION DATA COLLECTION

The bridges contained a varying number of barrier segments. In order to have a consistent data set, only the first half of the barrier segments in the traffic direction were inspected.

A sample inspection raw data sheet is shown in Figure 4-1. The inspection data was obtained from one barrier segment of bridge S06 of 82022 located in the Metro Region. The bridge carries I-94 WB and the barrier was constructed in 1993. All visual defects are marked on the barrier inspection template as lines and/or zones corresponding to their location on the barrier segment. The crack length and width as well as distance between cracks are noted. Bridge orientation is documented in order to evaluate the exposure effects. Photos taken from the traffic side of the barrier are marked on the inspection template. The condition of the fascia of each barrier is documented with photos taken from the intersecting roadway. The photograph index number is recorded on the inspection template with an arrow showing the photo direction. Selected photos of the inspected barrier are shown in Figure 4-2. The inspection data, shown in Figure 4-1, shows significant cracking.

4.4.1 Inspection Protocol

Bridges with New Jersey barriers and shoulders wider than 12 feet were selected for inspection. This provided the inspection crew with a safe and workable area without any need for lane closure. Topology of segments and all visible distresses were recorded with regard to their location, width, and dimensions. A general bridge view and data was also collected for each bridge.

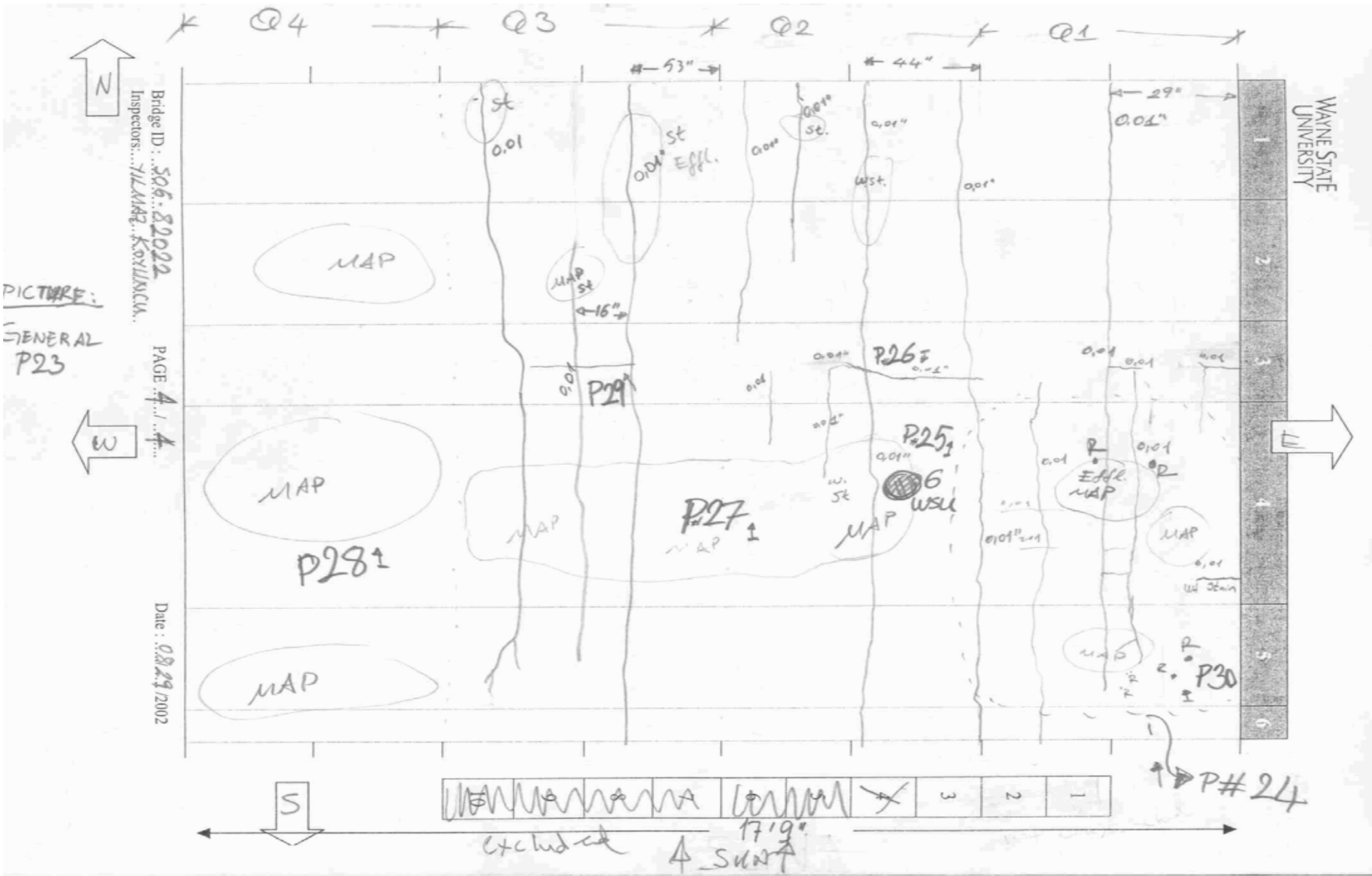


Figure 4-1. Barrier inspection template (S06-82022)

Photograph # 23

Photograph # 24

Photograph # 28

Photograph # 29

Photograph # 25

Photograph # 30

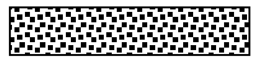
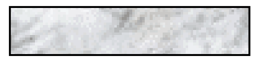
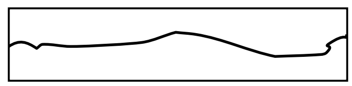
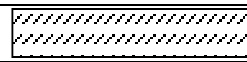
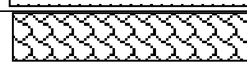
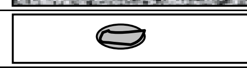
Figure 4-2. Sample photos documenting the visual data documented on the barrier inspection template (S06-82022)

(Photograph numbers refer to the location index shown in Figure 4-1)

4.4.2 Inspection Data Processing

Prior to quantitative analysis, all inspection raw data sheets are presented using graphical symbols. The exposure is defined based on the direction of the traffic bearing side of the barrier (e.g., if the traffic bearing side of the barrier faces north, its exposure is designated as "north exposure"). The legend shown in Table 4-2 is used to represent the data.

Table 4-2. Symbols used to Represent Distress Types

Distress Type	Symbol
Spall	
Delamination	
Crack (Heavier lines signify crack widths > 0.015)	
Map Cracking	
Scaling	
Patch	
Popout	

Figures 4-3 through 4-22 show the barrier condition documented on the inspection templates in processed form.

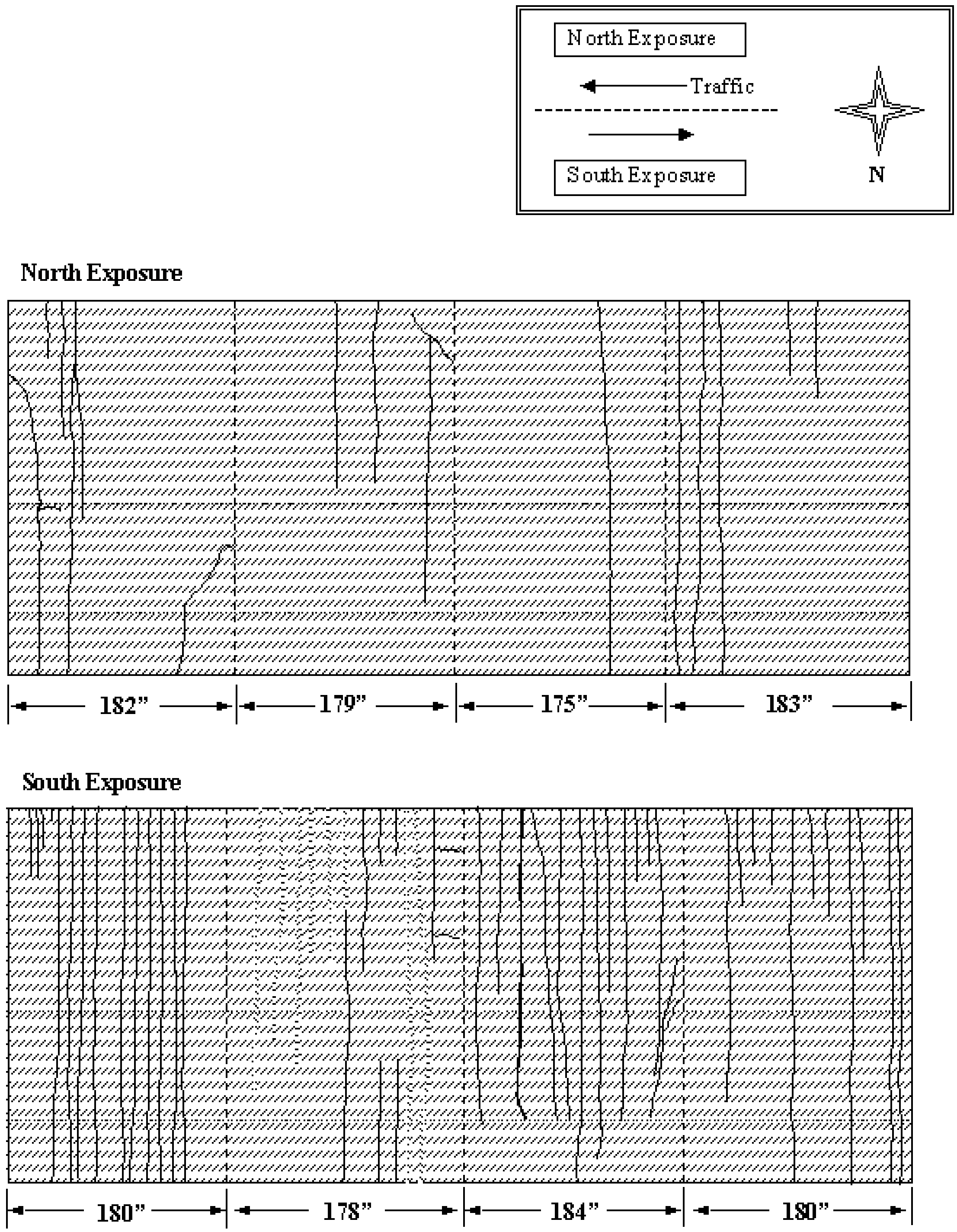
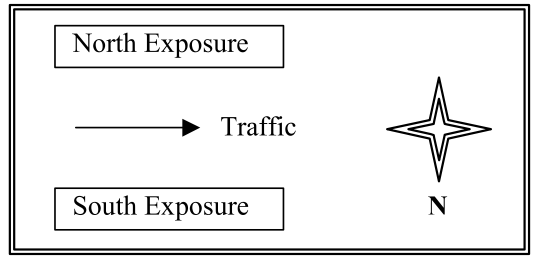
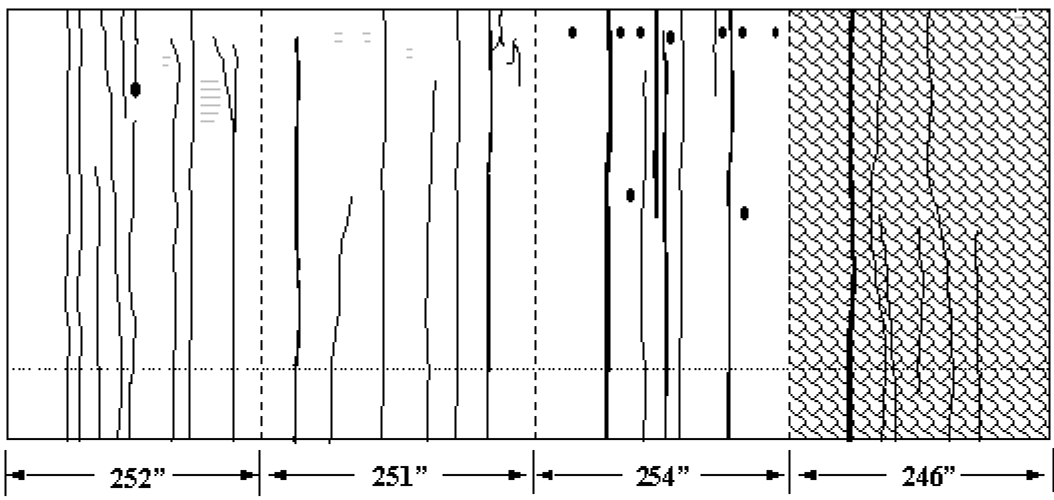


Figure 4-3. Inspection raw data of bridge B01-50021



North Exposure



South Exposure

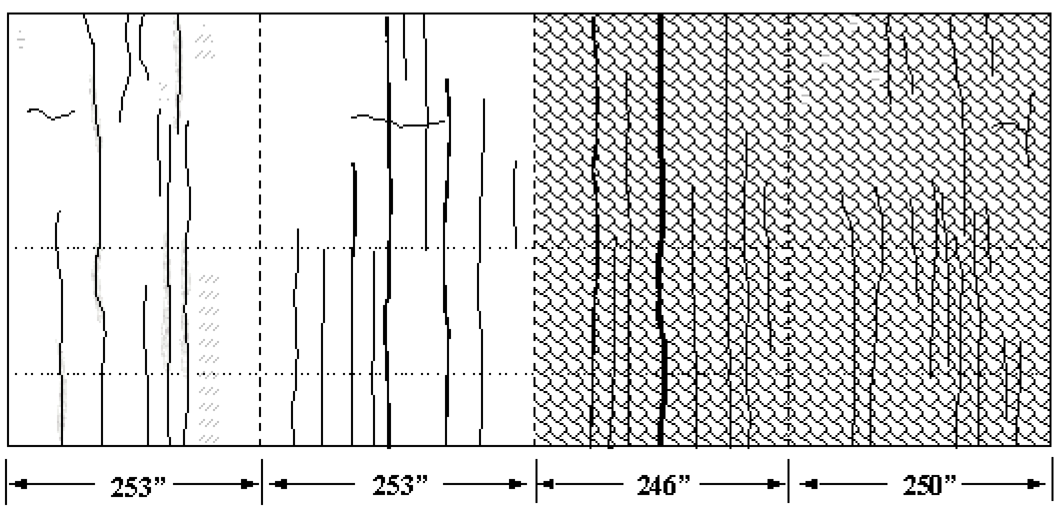


Figure 4-4. Inspection raw data of bridge B01-66051

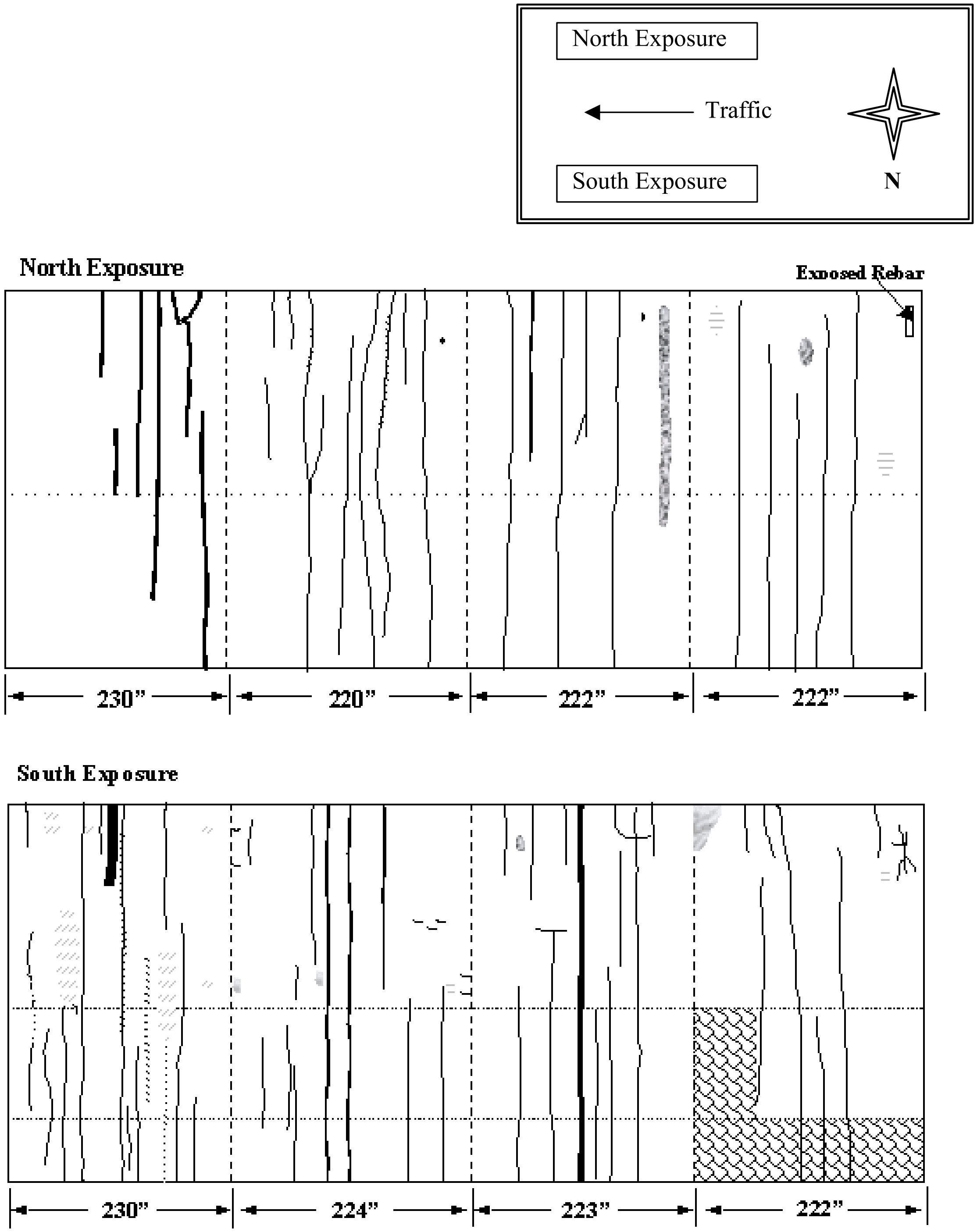


Figure 4-5. Inspection raw data of bridge B02-66051

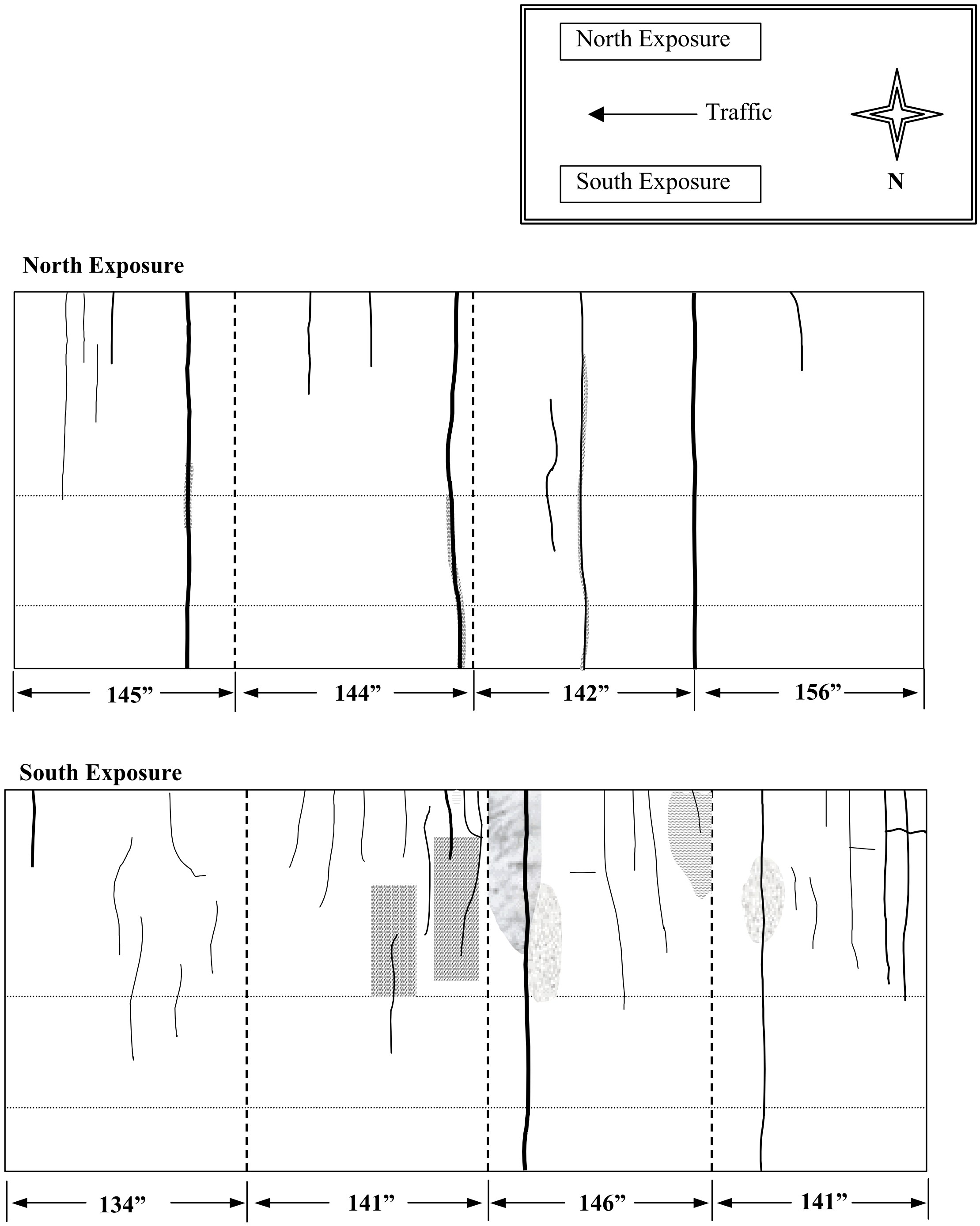


Figure 4-6. Inspection raw data of bridge S01-44044

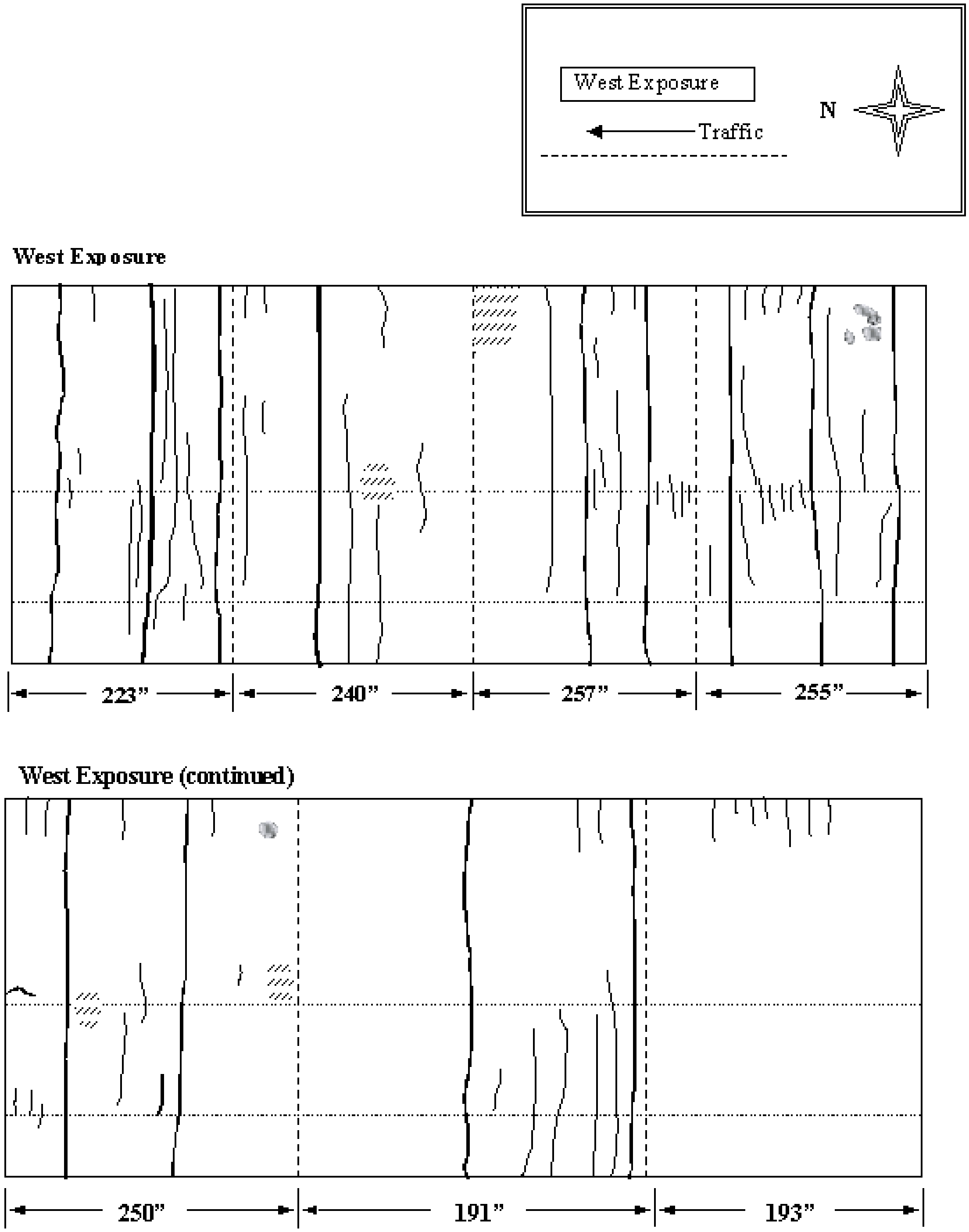
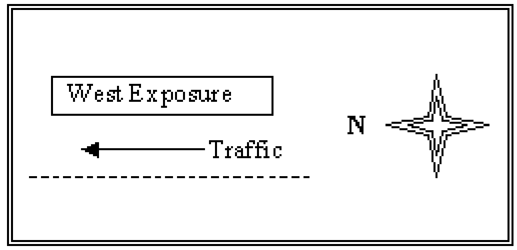
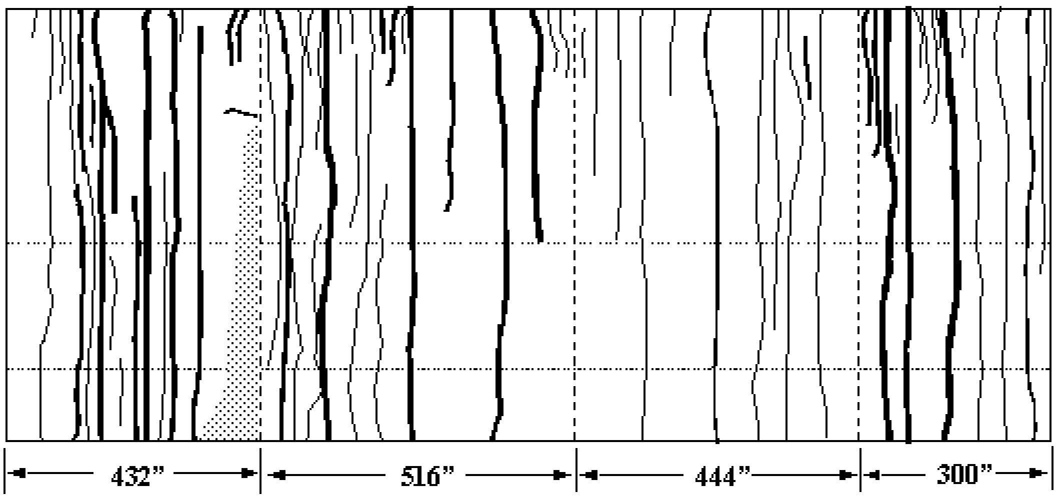


Figure 4-7. Inspection raw data of bridge S20-63174



West Exposure



West Exposure (continued)

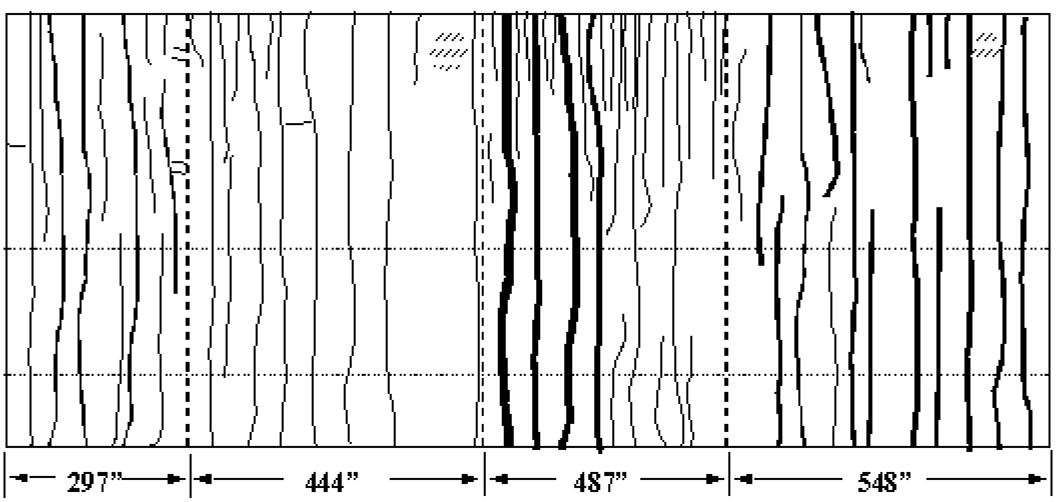


Figure 4-8. Inspection raw data of bridge S02-82194

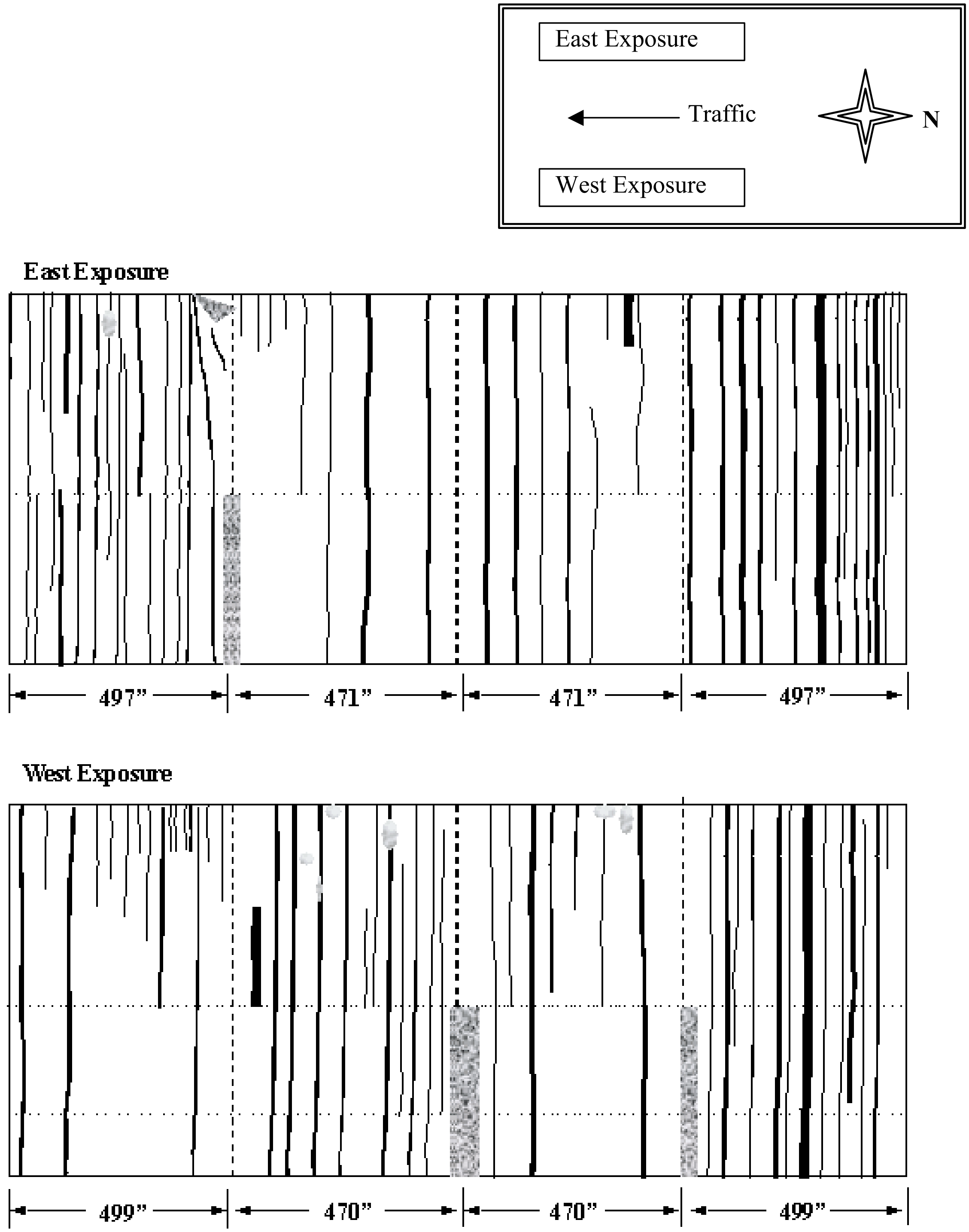


Figure 4-9. Inspection raw data of bridge S02-23152

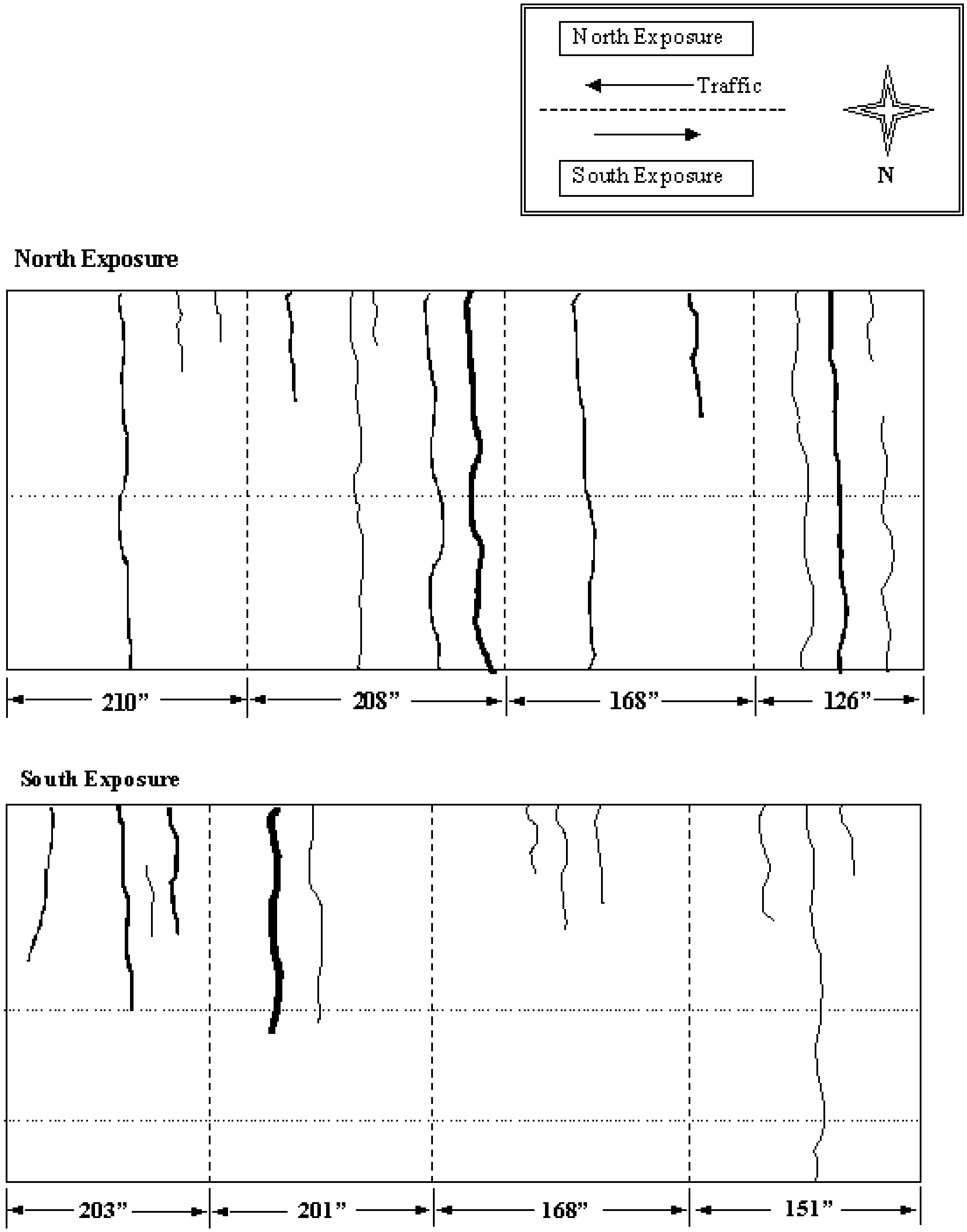


Figure 4-10. Inspection raw data of bridge S04-63101

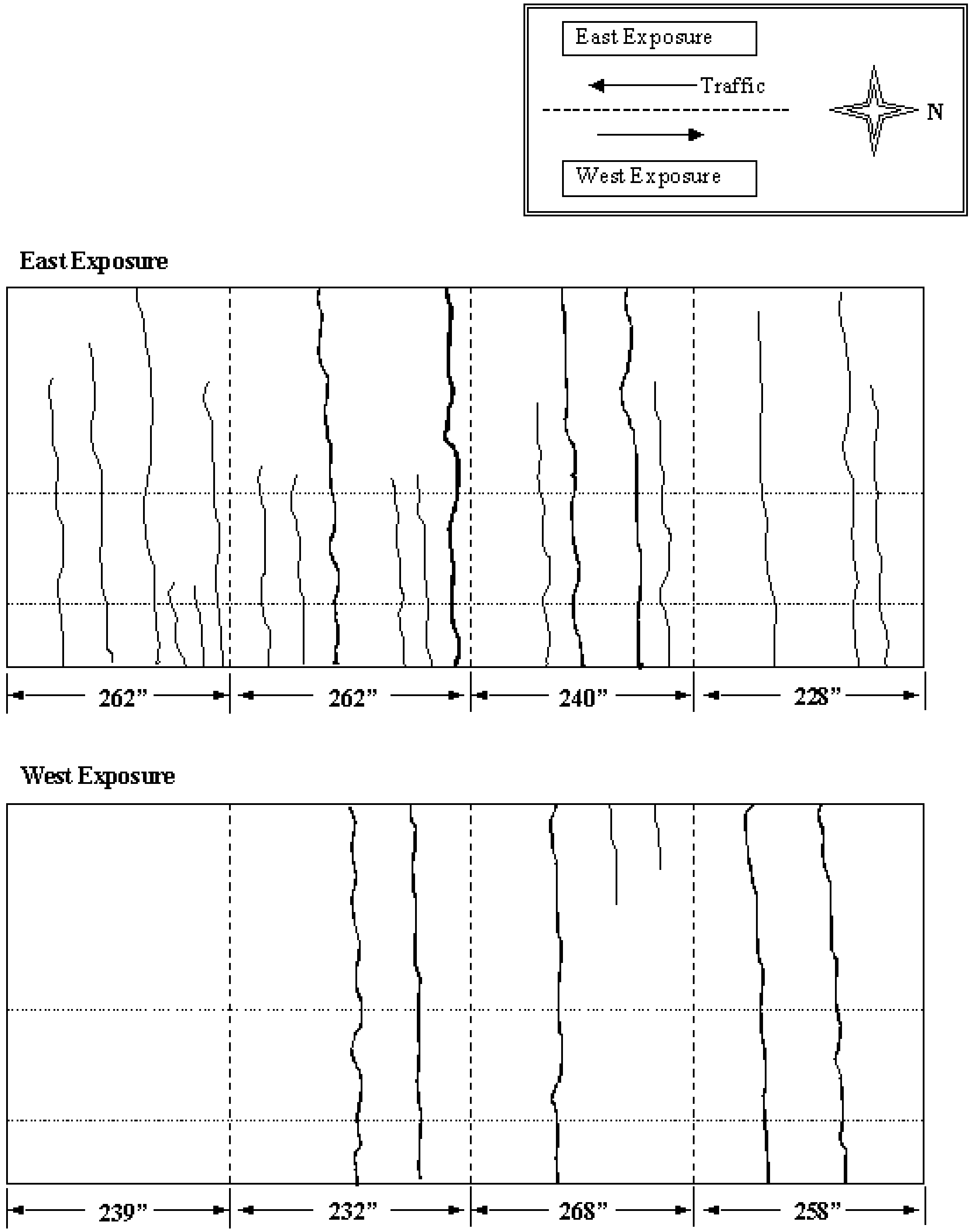


Figure 4-11. Inspection raw data of bridge S04-63174

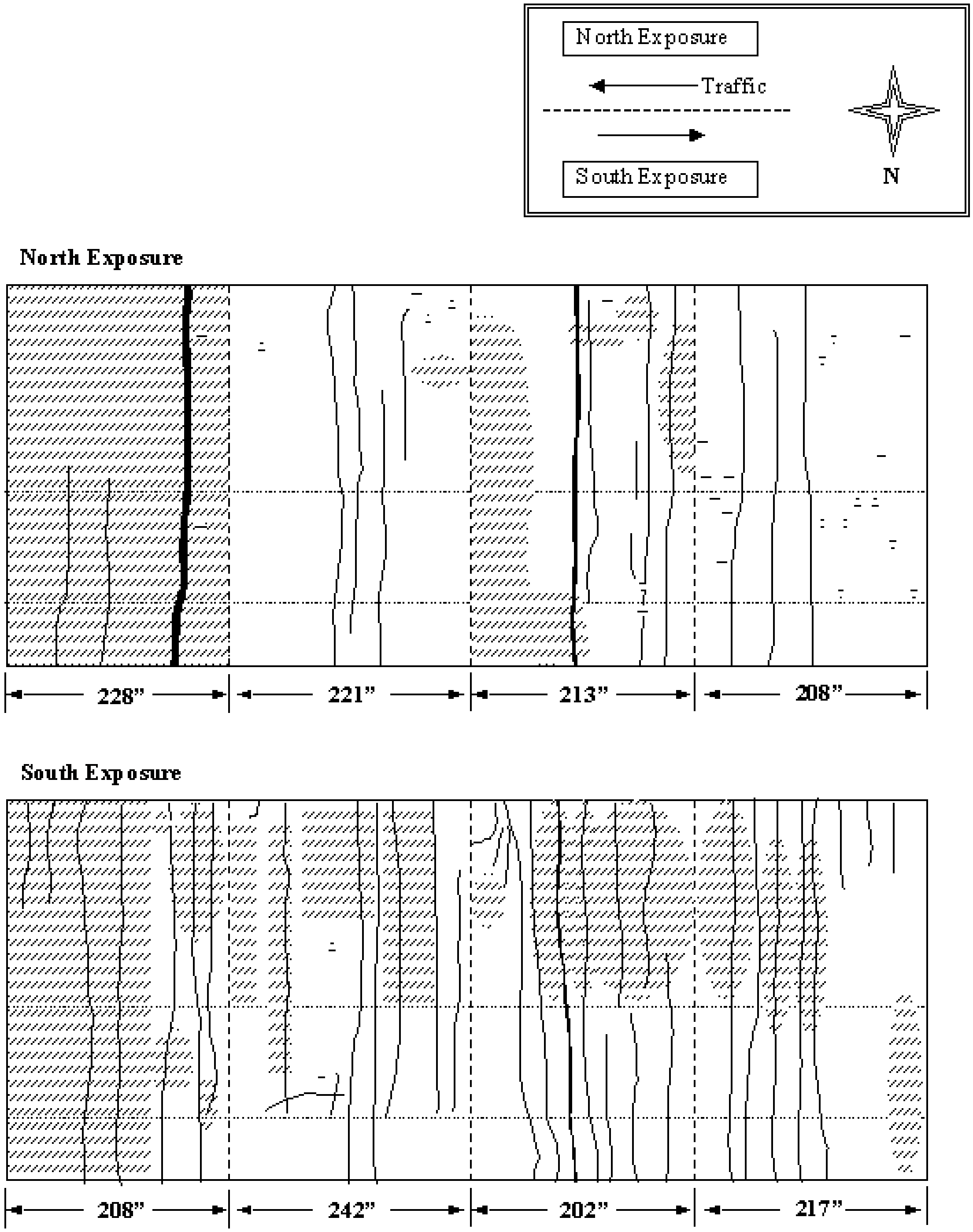


Figure 4-12. Inspection raw data of bridge S04-82022

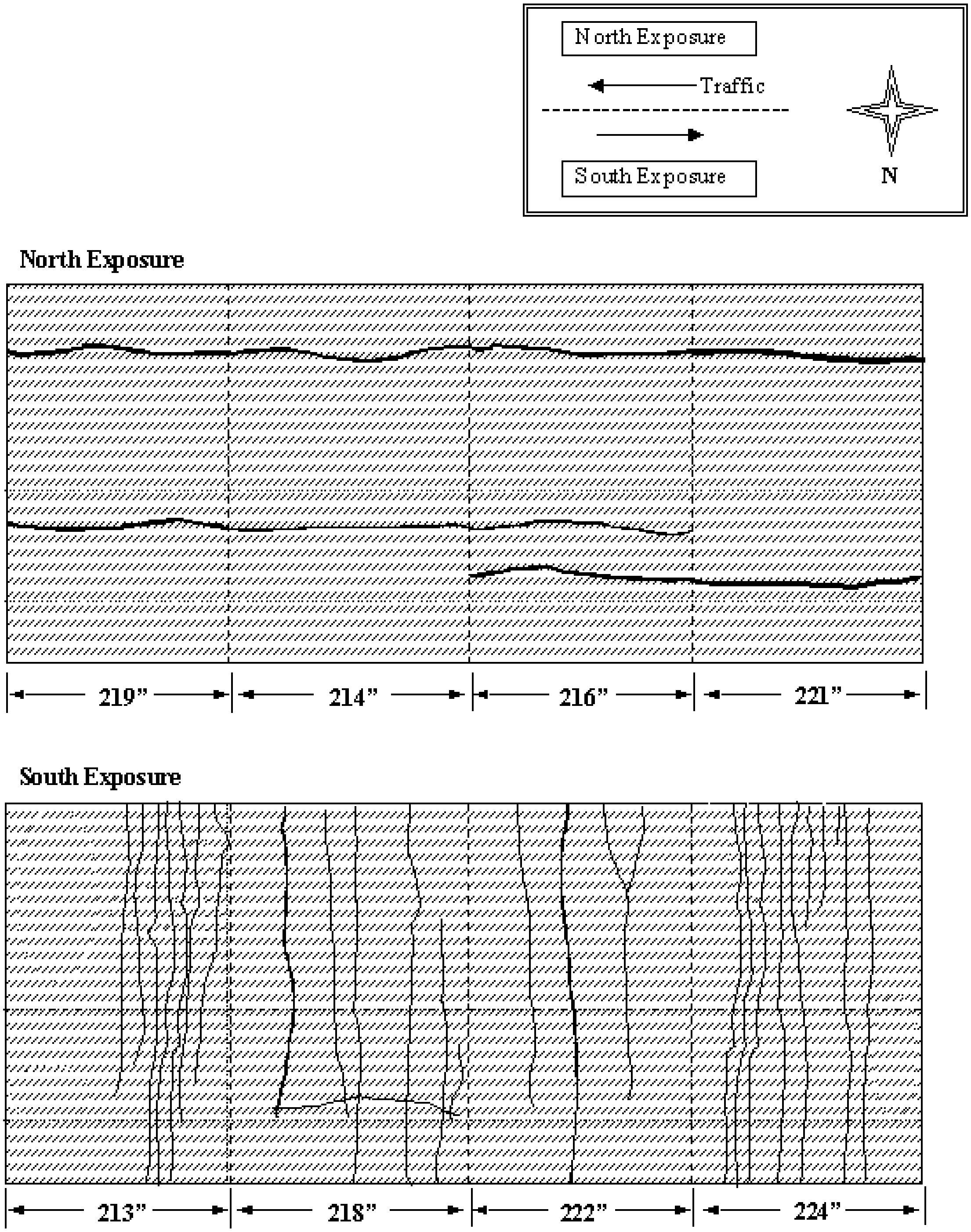


Figure 4-13. Inspection raw data of bridge S06-82022

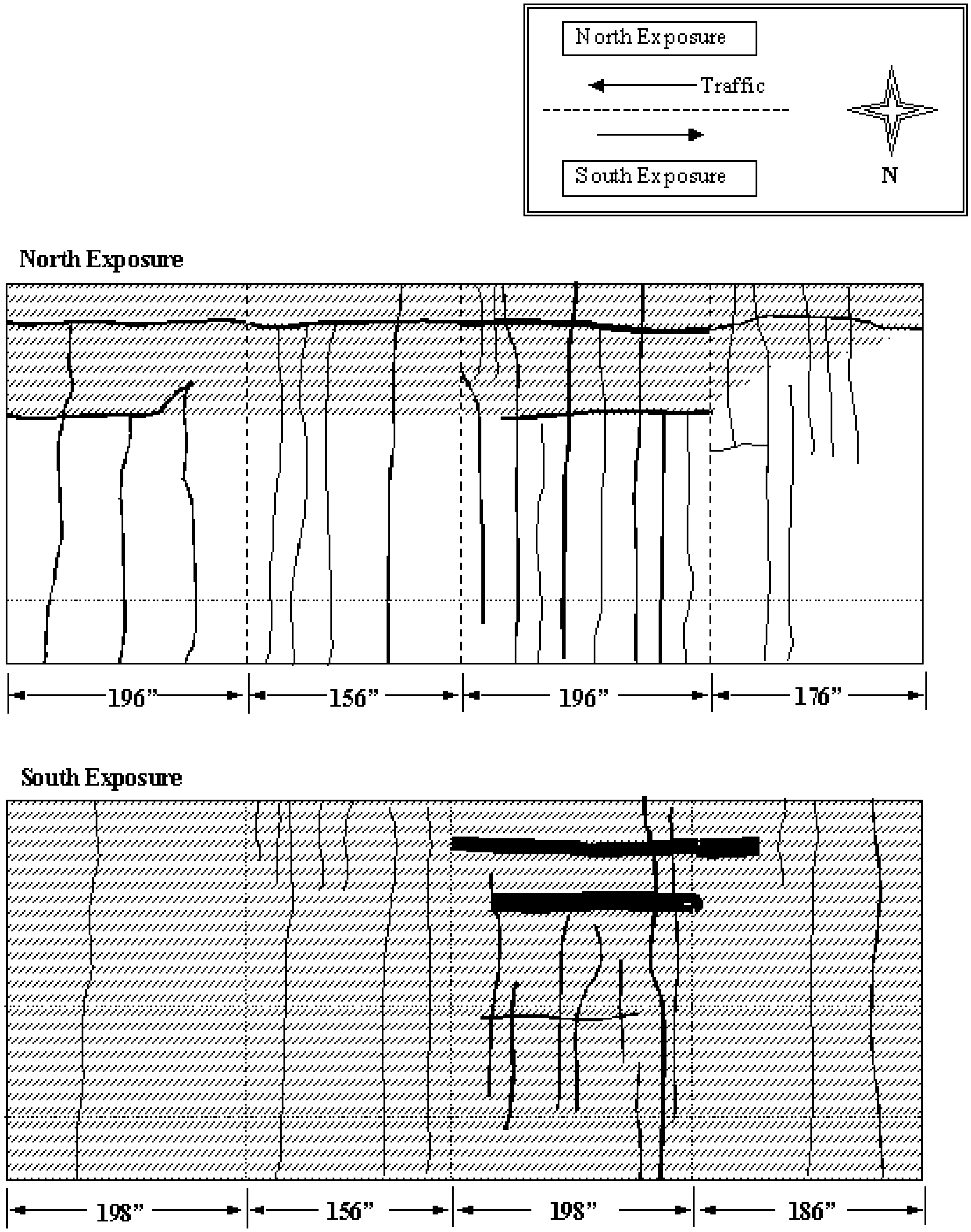


Figure 4-14. Inspection raw data of bridge S08-82191

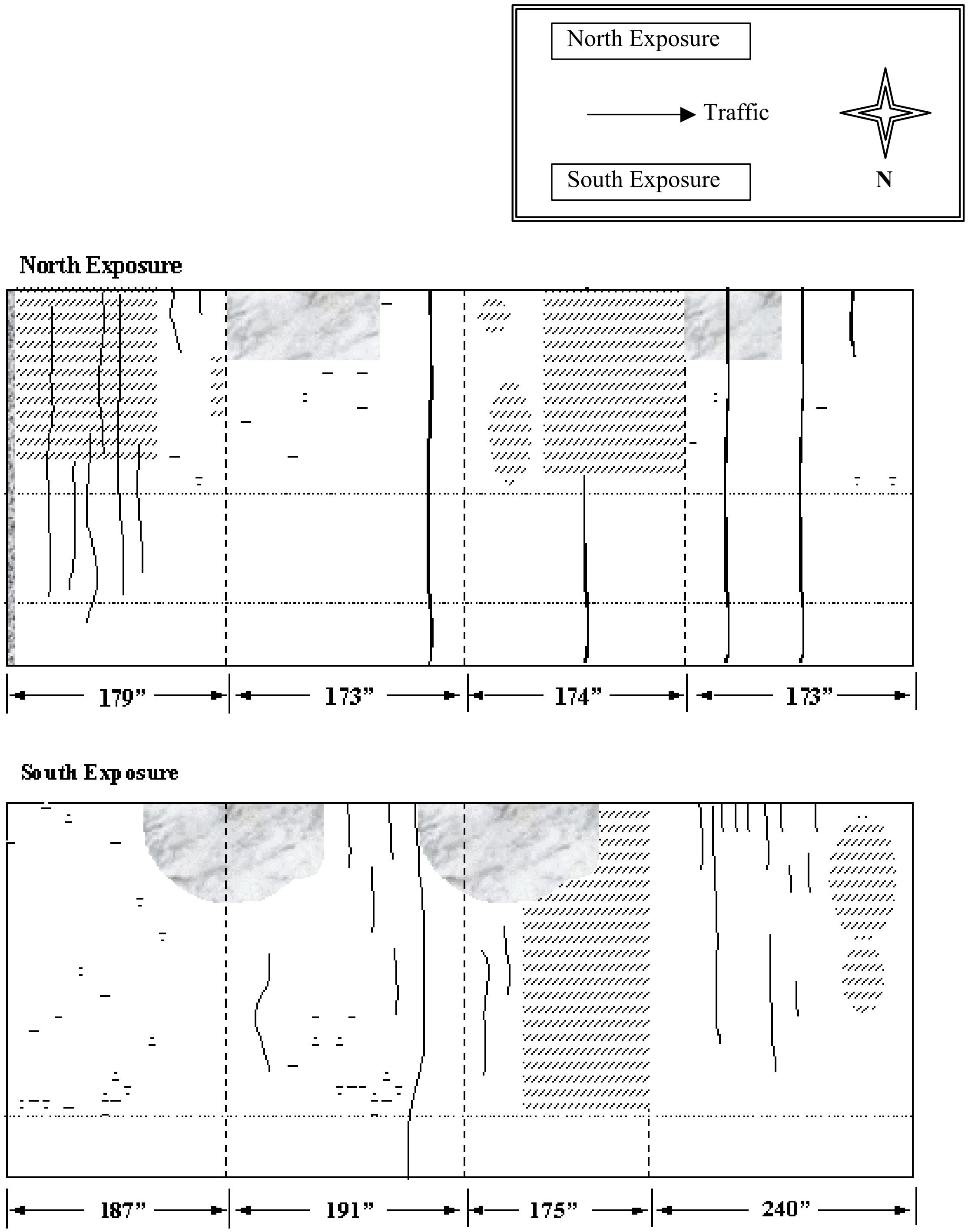


Figure 4-15. Inspection raw data of bridge S09-63101

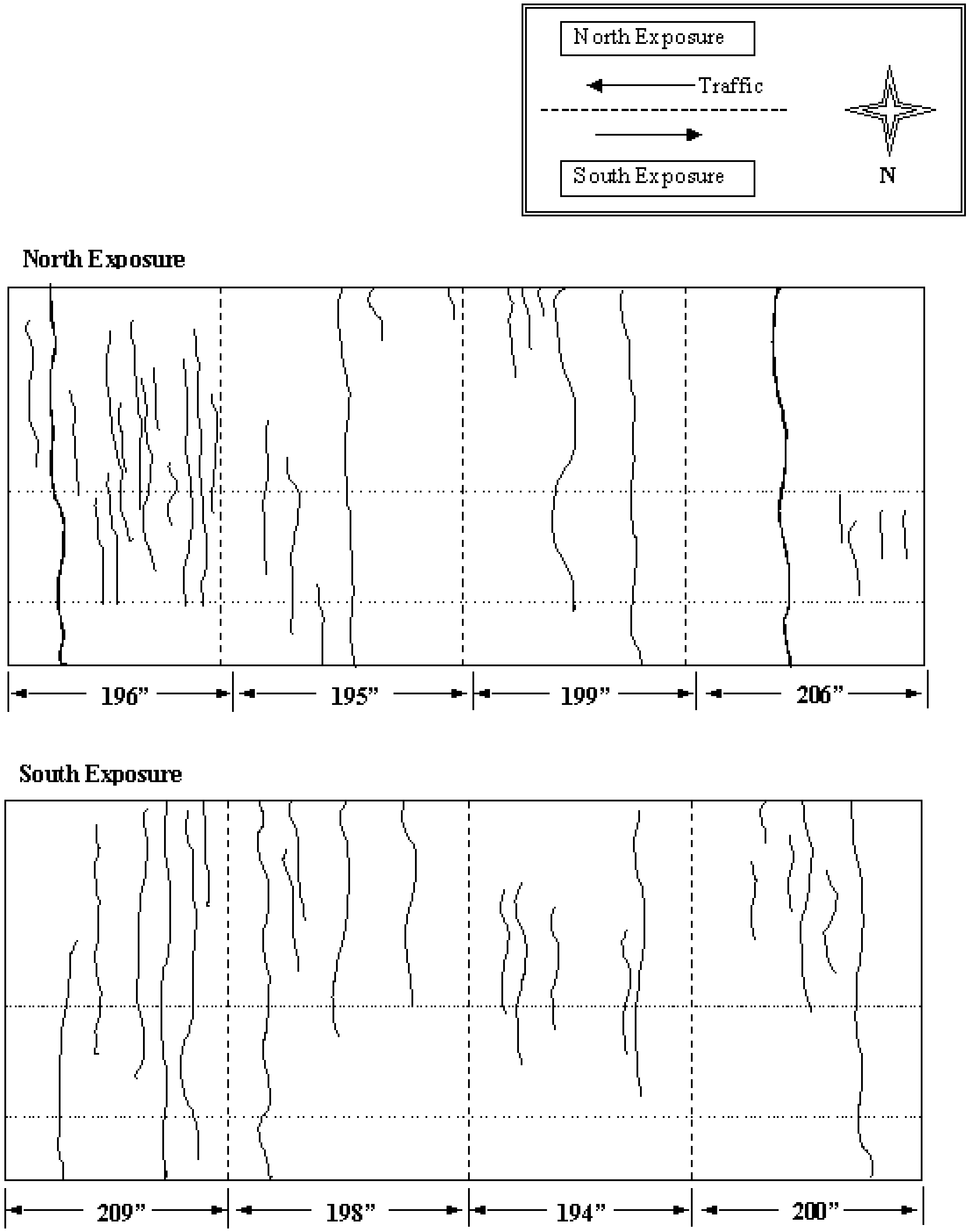


Figure 4-16. Inspection raw data of bridge S12-63022

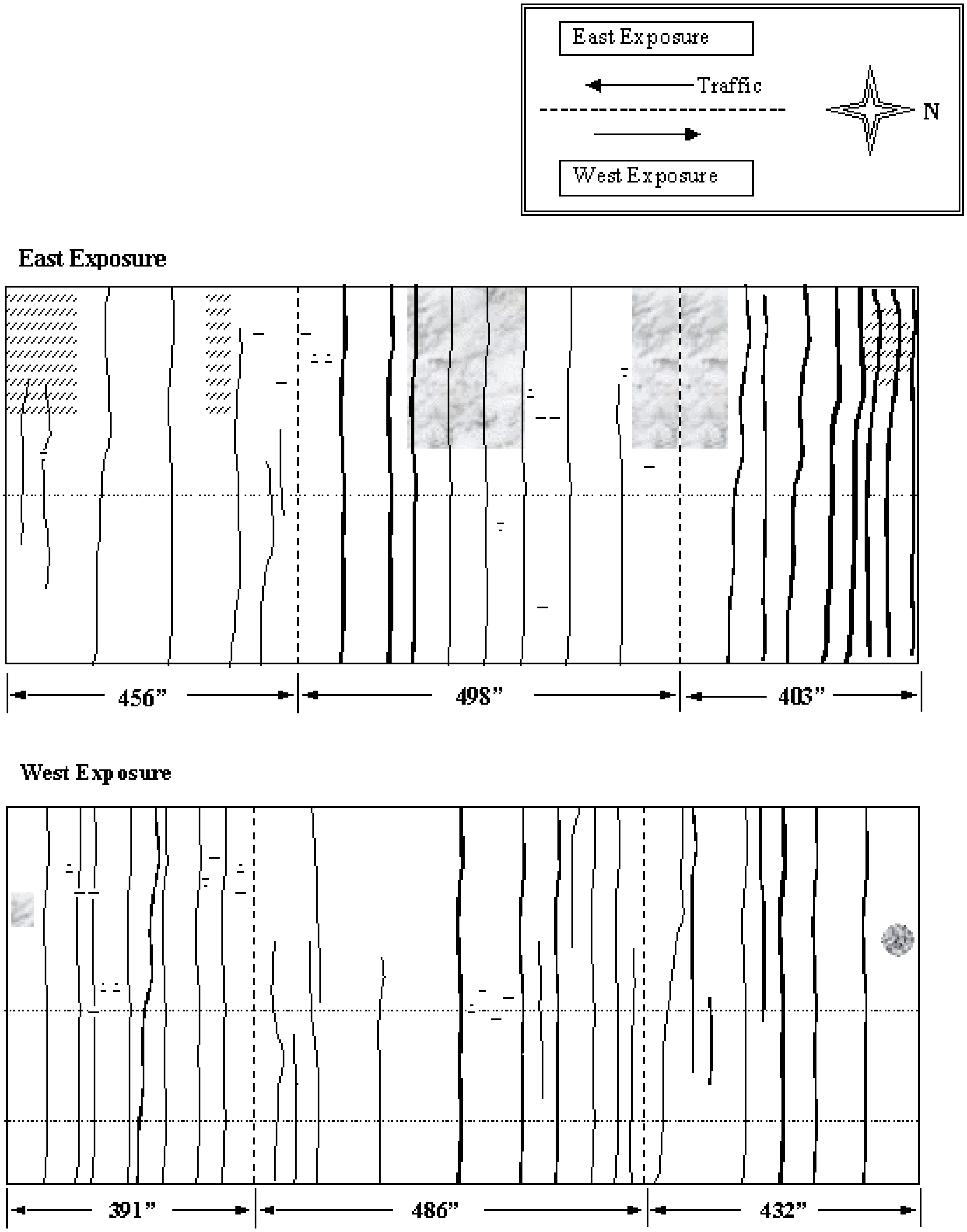


Figure 4-17. Inspection raw data of bridge S15-63172

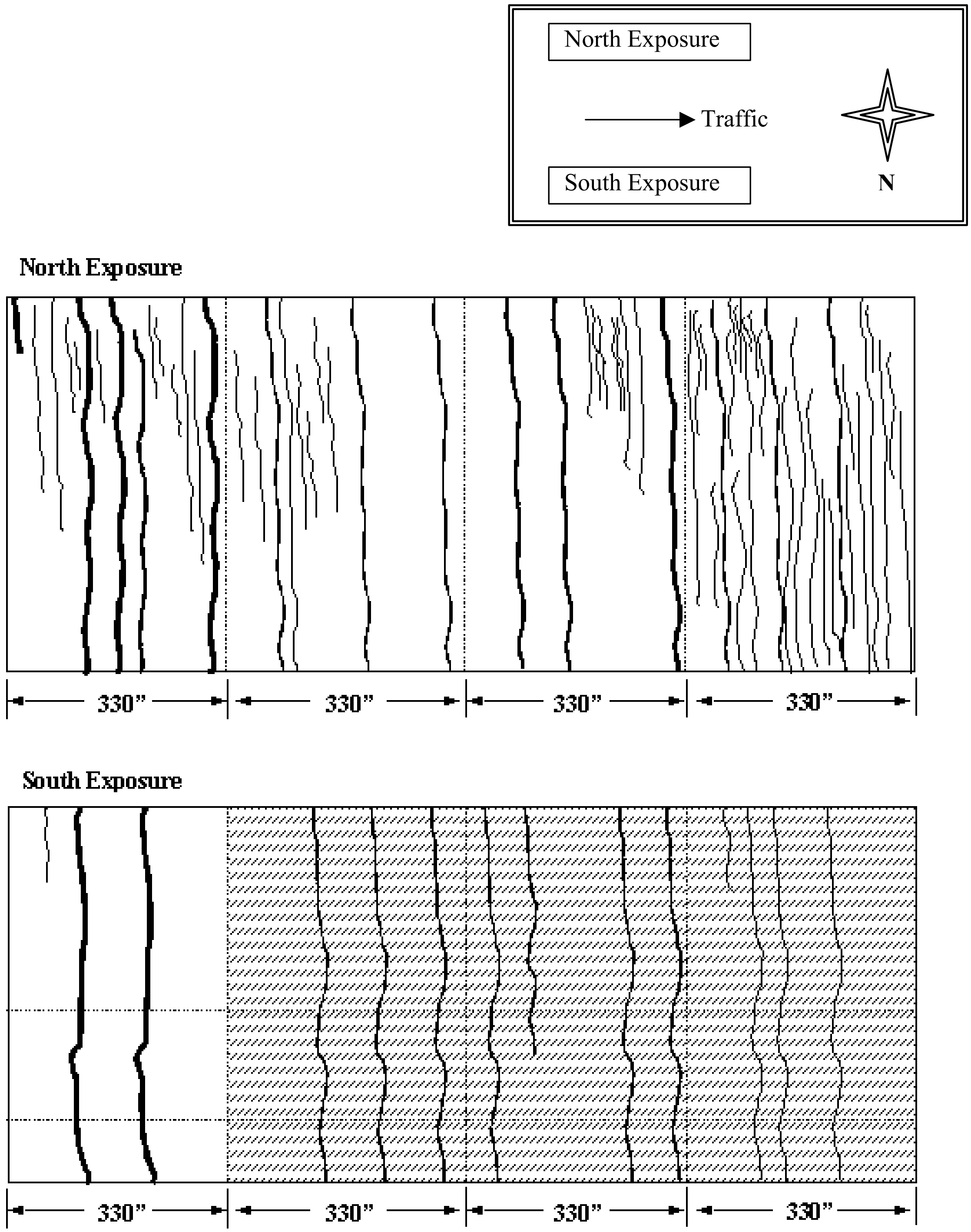


Figure 4-18. Inspection raw data of bridge S24-82022

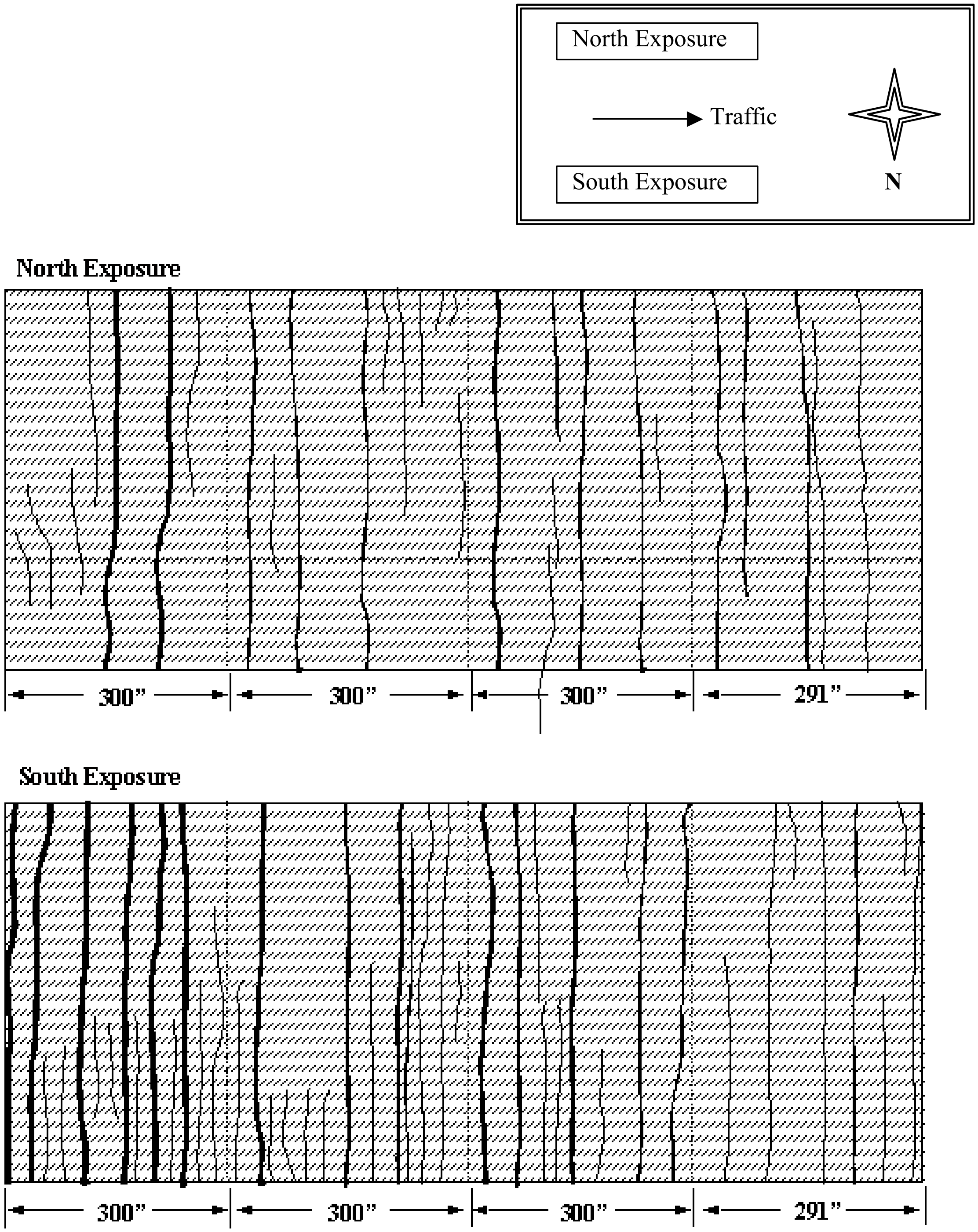


Figure 4-19. Inspection raw data of bridge S26-82022

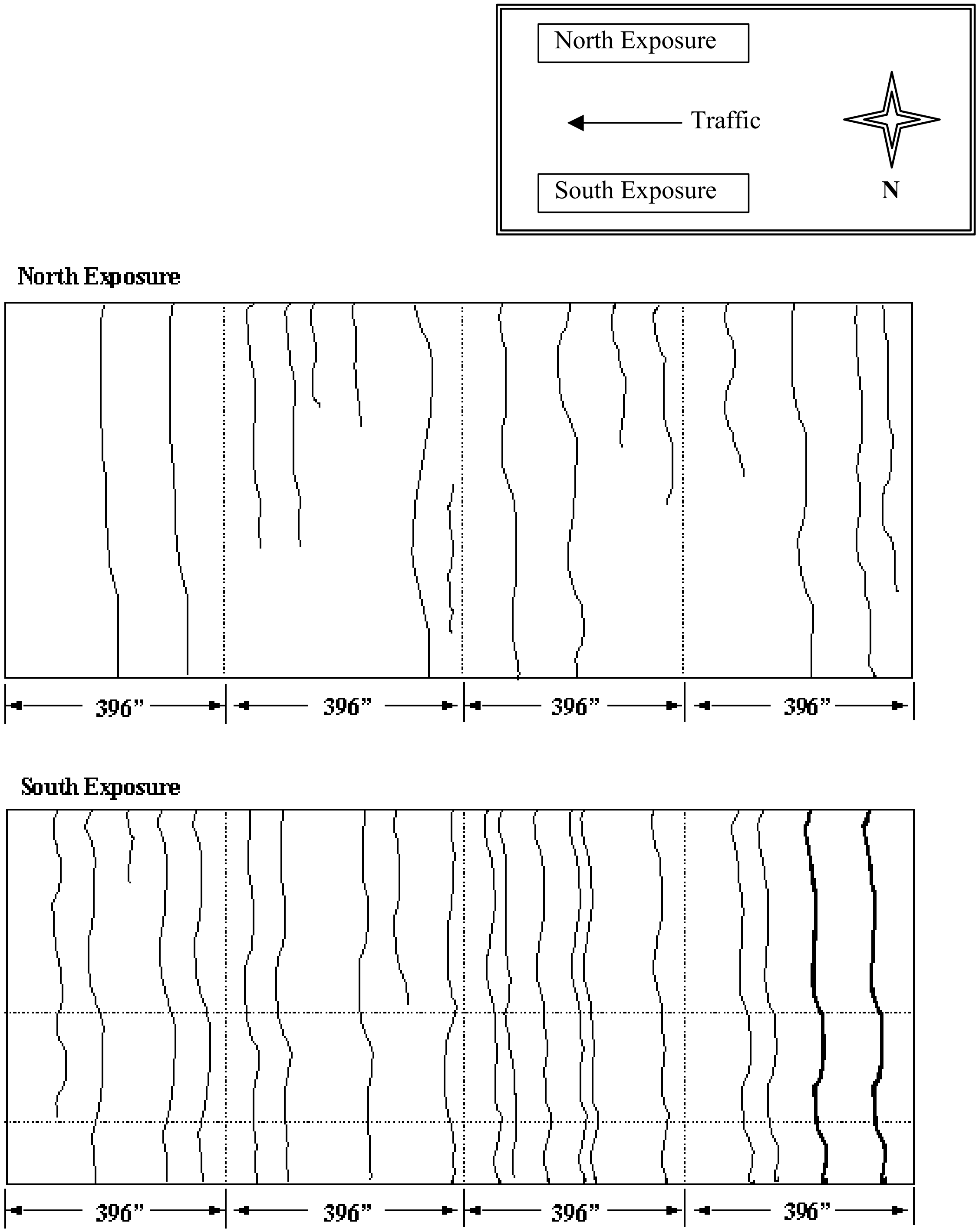


Figure 4-20. Inspection raw data of bridge S27-41064

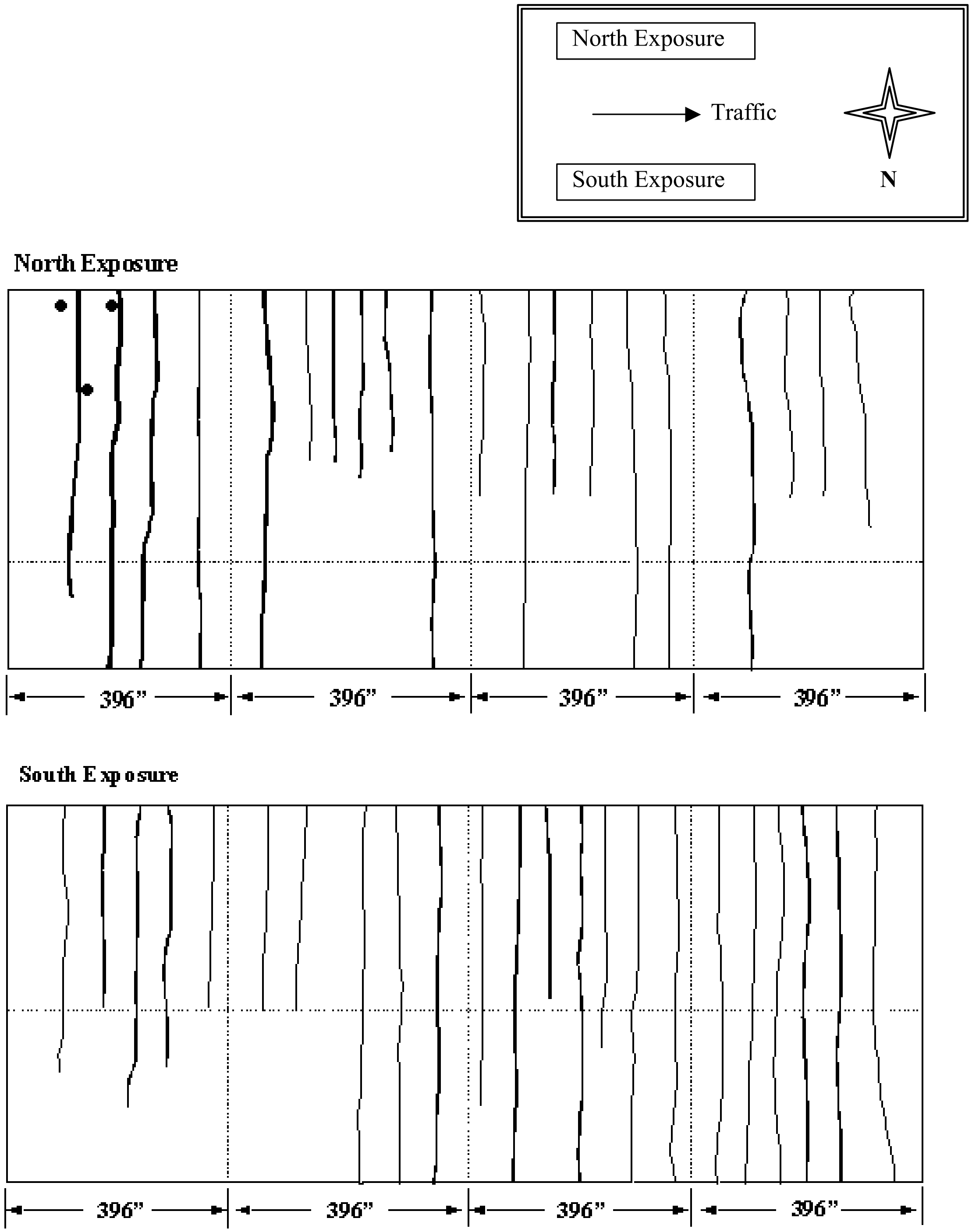


Figure 4-21. Inspection raw data of bridge S28-41064

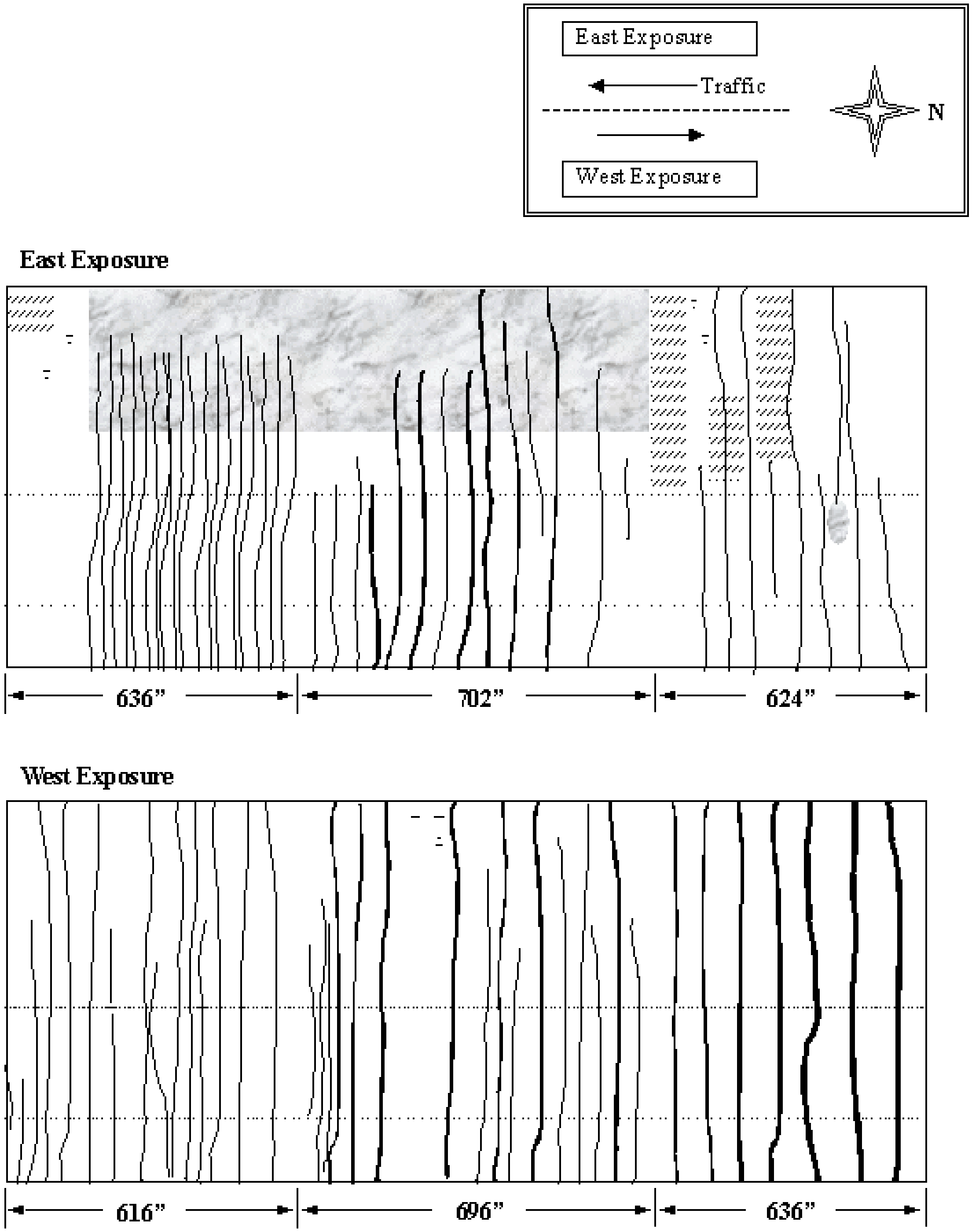


Figure 4-22. Inspection raw data of bridge S12-63172

4.5 COMPILATION OF INSPECTION DATA

The main distress types are compiled from the raw data. The distress types are: vertical cracking, horizontal cracking, map cracking, delamination, spalling and disintegration, patch, and popout. Data is first compiled for each individual barrier segment and grouped under the bridge ID.

Vertical cracking is grouped under different crack widths due to the fact that it is a distress type observed at a very early-age. It is hypothesized that other distresses such as corrosion, horizontal cracking, spall and disintegration proliferate from vertical cracking. Also, map cracking appears at early ages of barriers. However, the cracks within the map cracked area remain near the surface and do not proliferate to other distress types. Table 4-3 through Table 4-22 summarizes the map cracking, vertical cracking, and horizontal cracking data for each barrier segment. Table 4-23 summarizes the data compiled for each bridge.

Table 4-3. Compiled Data on Barrier Distress

Bridge ID B01-50021 (1)	Segment Length (Inches) (2)	Exposure (3)	Map cracking (%) (4)	Total number of vertical cracks (5)	Number of vertical cracks > 0.01 in. (6)	Number of vertical cracks >0.015 in. (7)	Total number of horizontal cracks (8)
Segment 1	182	NORTH	100	4	3	0	2
Segment 2	179	NORTH	100	3	0	0	1
Segment 3	175	NORTH	100	1	1	0	0
Segment 4	183	NORTH	100	5	3	0	0
Segment 5	180	SOUTH	100	15	13	0	0
Segment 6	178	SOUTH	100	12	12	1	2
Segment 7	184	SOUTH	100	11	5	1	1
Segment 8	180	SOUTH	100	13	3	0	0
Total	**1441**			**64**	**40**	**2**	**6**
Average	**180**		**100**	**8**	**5**	**0.25**	**1**

This bridge was constructed in 1994 using PCI girders. The data obtained for evaluation was taken from span two of this bridge.

1. Segment number
2. Segment length measured in inches
3. Direction of exposure of the traffic side of the barrier
4. Percentage of map cracking based on the barrier surface area
5. Total number of vertical cracks on the segment
6. Total number of vertical cracks with crack widths greater than 0.01 inches
7. Total number of vertical cracks with crack widths greater than 0.015 inches
8. Total number of horizontal cracks on the segment

Table 4-4. Compiled Data on Barrier Distress

Bridge ID B01-66051 (1)	Segment Length (Inches) (2)	Exposure (3)	Map cracking (%) (4)	Total number of vertical cracks (5)	Number of vertical cracks > 0.01 in. (6)	Number of vertical cracks >0.015 in. (7)	Total number of horizontal cracks (8)
Segment 1	253	SOUTH	1	10	2	0	1
Segment 2	253	SOUTH	0	10	10	3	1
Segment 3	246	SOUTH	0	8	4	2	0
Segment 4	250	SOUTH	0	7	1	0	1
Segment 5	252	NORTH	0	9	3	1	0
Segment 6	251	NORTH	0	6	5	2	0
Segment 7	254	NORTH	0	7	5	4	0
Segment 8	246	NORTH	0	6	3	1	0
Total	**2005**			**63**	**33**	**13**	**3**
Average	**251**		**0**	**8**	**4**	**2**	**0**

This bridge was constructed in 1985 using steel girders. The data obtained for evaluation was taken from span three of this bridge.

Table 4-5. Compiled Data on Barrier Distress

Bridge ID B02-66051 (1)	Segment Length (Inches) (2)	Exposure (3)	Map cracking (%) (4)	Total number of vertical cracks (5)	Number of vertical cracks > 0.01 in. (6)	Number of vertical cracks >0.015 in. (7)	Total number of horizontal cracks (8)
Segment 1	230	NORTH	8	7	6	1	0
Segment 2	220	NORTH	0	8	2	0	0
Segment 3	222	NORTH	0	5	5	1	0
Segment 4	222	NORTH	0	5	0	0	0
Segment 5	230	SOUTH	2	10	3	0	0
Segment 6	224	SOUTH	0	11	8	3	6
Segment 7	223	SOUTH	0	11	1	1	2
Segment 8	222	SOUTH	0	5	0	0	0
Total	**1793**			**62**	**25**	**6**	**8**
Average	**224**		**1**	**8**	**3**	**0.75**	**1**

This bridge was constructed in 1985 using steel girders. The data obtained for evaluation was taken from span three of this bridge.

Table 4-6. Compiled Data on Barrier Distress

Bridge ID S01-44044 (1)	Segment Length (Inches) (2)	Exposure (3)	Map cracking (%) (4)	Total number of vertical cracks (5)	Number of vertical cracks > 0.01 in. (6)	Number of vertical cracks >0.015 in. (7)	Total number of horizontal cracks (8)
Segment 1	145	NORTH	0	1	1	0	0
Segment 2	144	NORTH	0	2	1	0	0
Segment 3	142	NORTH	0	6	6	1	0
Segment 4	156	NORTH	0	5	2	1	0
Segment 5	134	SOUTH	0	6	1	1	1
Segment 6	141	SOUTH	0	8	4	1	0
Segment 7	146	SOUTH	3	4	0	0	1
Segment 8	141	SOUTH	0	6	0	0	2
Total	**1149**			**38**	**15**	**4**	**4**
Average	**144**		**0**	**5**	**2**	**0.50**	**1**

This bridge was constructed in 1983 using steel girders. The data obtained for evaluation was taken from span one of this bridge

Table 4-7. Compiled Data on Barrier Distress

Bridge ID S20-63174 (1)	Segment Length (Inches) (2)	Exposure (3)	Map cracking (%) (4)	Total number of vertical cracks (5)	Number of vertical cracks > 0.01 in. (6)	Number of vertical cracks >0.015 in. (7)	Total number of horizontal cracks (8)
Segment 1	193	EAST	1	6	5	0	0
Segment 2	191	EAST	0	9	7	2	0
Segment 3	250	EAST	0	11	9	4	1
Segment 4	255	EAST	0	8	8	3	0
Segment 5	257	EAST	1	9	6	2	0
Segment 6	240	EAST	0	7	4	1	0
Segment 7	223	EAST	0	9	3	3	0
Total	**1609**			**59**	**42**	**15**	**1**
Average	**230**		**0**	**8**	**6**	**2**	**0**

This bridge was constructed in 2001 using PCI girders. The data obtained for evaluation was taken from span three of this bridge.

Table 4-8. Compiled Data on Barrier Distress

Bridge ID S02-82194 (1)	Segment Length (Inches) (2)	Exposure (3)	Map cracking (%) (4)	Total number of vertical cracks (5)	Number of vertical cracks > 0.01 in. (6)	Number of vertical cracks >0.015 in. (7)	Total number of horizontal cracks (8)
Segment 1	432	WEST	0	16	15	11	3
Segment 2	516	WEST	0	16	11	9	3
Segment 3	444	WEST	0	12	8	4	5
Segment 4	300	WEST	0	10	9	6	1
Segment 5	297	WEST	0	14	10	3	5
Segment 6	444	WEST	0	15	9	4	6
Segment 7	487	WEST	0	16	11	5	3
Segment 8	548	WEST	0	16	16	14	2
Total	**3468**			**115**	**89**	**56**	**28**
Average	**434**		**0**	**14**	**11**	**7**	**4**

This bridge was constructed in 1986 using steel girders. The data obtained for evaluation was taken from span four of this bridge.

Table 4-9. Compiled Data on Barrier Distress

Bridge ID S02-23152 (1)	Segment Length (Inches) (2)	Exposure (3)	Map cracking (%) (4)	Total number of vertical cracks (5)	Number of vertical cracks > 0.01 in. (6)	Number of vertical cracks >0.015 in. (7)	Total number of horizontal cracks (8)
Segment 1	497	EAST	0	12	10	6	0
Segment 2	471	EAST	0	10	9	5	2
Segment 3	471	EAST	0	7	5	4	0
Segment 4	497	EAST	0	11	10	6	5
Segment 5	499	WEST	0	15	12	9	2
Segment 6	470	WEST	0	9	4	2	4
Segment 7	470	WEST	0	8	7	2	1
Segment 8	499	WEST	0	17	15	3	5
Total	**3874**			**89**	**72**	**37**	**19**
Average	**484**		**0**	**11**	**9**	**5**	**2**

This bridge was constructed in 1980 using steel girders. The data obtained for evaluation was taken from span three of this bridge.

Table 4-10. Compiled Data on Barrier Distress

Bridge ID S04-63101 (1)	Segment Length (Inches) (2)	Exposure (3)	Map cracking (%) (4)	Total number of vertical cracks (5)	Number of vertical cracks > 0.01 in. (6)	Number of vertical cracks >0.015 in. (7)	Total number of horizontal cracks (8)
Segment 1	203	SOUTH	0	4	3	3	0
Segment 2	201	SOUTH	0	2	2	2	1
Segment 3	168	SOUTH	0	3	0	0	0
Segment 4	151	SOUTH	46	3	2	0	5
Segment 5	210	NORTH	0	3	1	1	3
Segment 6	208	NORTH	0	5	4	3	2
Segment 7	168	NORTH	0	2	2	2	0
Segment 8	126	NORTH	0	3	3	1	0
Total	**1435**			**25**	**17**	**12**	**11**
Average	**179**		**6**	**3**	**2**	**2**	**1**

This bridge was constructed in 1988 using PCI girders. The data obtained for evaluation was taken from span three of this bridge.

Table 4-11. Compiled Data on Barrier Distress

Bridge ID S04-63174	Segment Length (Inches) (2)	Exposure (3)	Map cracking (%) (4)	Total number of vertical cracks (5)	Number of vertical cracks > 0.01 in. (6)	Number of vertical cracks >0.015 in. (7)	Total number of horizontal cracks (8)
Segment 1	239	WEST	0	0	0	0	0
Segment 2	232	WEST	0	2	2	2	0
Segment 3	268	WEST	0	3	3	2	0
Segment 4	258	WEST	0	3	3	3	0
Segment 5	228	EAST	0	3	3	0	0
Segment 6	240	EAST	0	4	4	2	0
Segment 7	262	EAST	0	6	4	3	0
Segment 8	262	EAST	0	6	3	2	0
Total	**1989**			**27**	**22**	**14**	**0**
Average	**249**		**0**	**3**	**3**	**2**	**0**

This bridge was constructed in 2001 using side-by-side box girders.

Table 4-12. Compiled Data on Barrier Distress

Bridge ID S04-82022 (1)	Segment Length (Inches) (2)	Exposure (3)	Map cracking (%) (4)	Total number of vertical cracks (5)	Number of vertical cracks > 0.01 in. (6)	Number of vertical cracks >0.015 in. (7)	Total number of horizontal cracks (8)
Segment 1	228	NORTH	22	13	7	0	0
Segment 2	221	NORTH	24	10	5	1	0
Segment 3	213	NORTH	14	6	3	0	0
Segment 4	208	NORTH	29	8	8	0	0
Segment 5	208	SOUTH	100	4	4	1	2
Segment 6	242	SOUTH	26	5	5	1	1
Segment 7	202	SOUTH	7	5	5	1	3
Segment 8	217	SOUTH	5	4	4	1	2
Total	**1739**			**55**	**41**	**5**	**8**
Average	**217**		**28**	**7**	**5**	**0.63**	**1**

This bridge was constructed in 1993 using side-by-side box girders. The data obtained for evaluation was taken from span two of this bridge.

Table 4-13. Compiled Data on Barrier Distress

Bridge ID S06-82022 (1)	Segment Length (Inches) (2)	Exposure (3)	Map cracking (%) (4)	Total number of vertical cracks (5)	Number of vertical cracks > 0.01 in. (6)	Number of vertical cracks >0.015 in. (7)	Total number of horizontal cracks (8)
Segment 1	219	NORTH	100	25	25	0	2
Segment 2	214	NORTH	100	24	24	0	3
Segment 3	216	NORTH	100	25	25	0	5
Segment 4	221	NORTH	100	24	24	0	5
Segment 5	224	SOUTH	26	12	5	0	4
Segment 6	222	SOUTH	0	3	3	1	1
Segment 7	218	SOUTH	68	8	4	1	9
Segment 8	213	SOUTH	24	11	11	0	7
Total	**1747**			**132**	**121**	**2**	**36**
Average	**218**		**65**	**17**	**15**	**0.25**	**5**

This bridge was constructed in 1993 using side-by-side box girders. The data obtained for evaluation was taken from span two of this bridge.

Table 4-14. Compiled Data on Barrier Distress

Bridge ID S08-82191 (1)	Segment Length (Inches) (2)	Exposure (3)	Map cracking (%) (4)	Total number of vertical cracks (5)	Number of vertical cracks > 0.01 in. (6)	Number of vertical cracks >0.015 in. (7)	Total number of horizontal cracks (8)
Segment 1	196	NORTH	0	3	3	3	3
Segment 2	156	NORTH	0	5	4	1	2
Segment 3	196	NORTH	0	10	10	5	3
Segment 4	176	NORTH	0	10	9	1	5
Segment 5	198	SOUTH	1	2	2	2	0
Segment 6	156	SOUTH	16	6	2	0	0
Segment 7	198	SOUTH	0	9	9	9	5
Segment 8	186	SOUTH	0	4	2	2	3
Total	**1462**			**49**	**41**	**23**	**21**
Average	**183**		**2**	**6**	**5**	**3**	**3**

This bridge was constructed in 1989 using steel girders. The data obtained for evaluation was taken from span four of this bridge.

Table 4-15. Compiled Data on Barrier Distress

Bridge ID S09-63101 (1)	Segment Length (Inches) (2)	Exposure (3)	Map cracking (%) (4)	Total number of vertical cracks (5)	Number of vertical cracks > 0.01 in. (6)	Number of vertical cracks >0.015 in. (7)	Total number of horizontal cracks (8)
Segment 1	187	SOUTH	0	3	0	0	0
Segment 2	191	SOUTH	0	12	2	0	1
Segment 3	175	SOUTH	11	15	6	1	3
Segment 4	240	SOUTH	3	11	0	0	1
Segment 5	179	NORTH	18	9	4	0	2
Segment 6	173	NORTH	2	1	1	1	2
Segment 7	174	NORTH	32	1	1	1	0
Segment 8	173	NORTH	0	2	2	2	5
Total	**1492**			**54**	**16**	**5**	**14**
Average	**187**		**8**	**7**	**2**	**0.63**	**2**

This bridge was constructed in 1988 using steel girders. The data obtained for evaluation was taken from span one of this bridge.

Table 4-16. Compiled Data on Barrier Distress

Bridge ID S12-63022 (1)	Segment Length (Inches) (2)	Exposure (3)	Map cracking (%) (4)	Total number of vertical cracks (5)	Number of vertical cracks > 0.01 in. (6)	Number of vertical cracks >0.015 in. (7)	Total number of horizontal cracks (8)
Segment 1	206	NORTH	12	4	1	1	3
Segment 2	199	NORTH	3	4	1	0	0
Segment 3	195	NORTH	5	4	0	0	2
Segment 4	196	NORTH	3	13	3	1	1
Segment 5	209	SOUTH	0	6	1	0	0
Segment 6	198	SOUTH	0	5	1	0	0
Segment 7	194	SOUTH	0	5	4	1	0
Segment 8	200	SOUTH	0	4	1	0	0
Total	**1597**			**45**	**12**	**3**	**6**
Average	**200**		**3**	**6**	**2**	**0.38**	**1**

This bridge was constructed in 1997 using PCI girders. The data obtained for evaluation was taken from span three of this bridge.

Table 4-17. Compiled Data on Barrier Distress

Bridge ID S15-63172 (1)	Segment Length (Inches) (2)	Exposure (3)	Map cracking (%) (4)	Total number of vertical cracks (5)	Number of vertical cracks > 0.01 in. (6)	Number of vertical cracks >0.015 in. (7)	Total number of horizontal cracks (8)
Segment 1	432	WEST	1	9	8	8	3
Segment 2	486	WEST	1	7	7	5	5
Segment 3	391	WEST	5	8	7	6	0
Segment 4	403	EAST	1	9	9	7	0
Segment 5	498	EAST	1	14	6	3	7
Segment 6	456	EAST	0	9	9	1	0
Total	**2666**			**56**	**46**	**30**	**15**
Average	**444**		**1**	**9**	**8**	**5**	**2**

This bridge was constructed in 1988 using steel girders. The data obtained for evaluation was taken from span three of this bridge.

Table 4-18. Compiled Data on Barrier Distress

Bridge ID S24-82022 (1)	Segment Length (Inches) (2)	Exposure (3)	Map cracking (%) (4)	Total number of vertical cracks (5)	Number of vertical cracks > 0.01 in. (6)	Number of vertical cracks >0.015 in. (7)	Total number of horizontal cracks (8)
Segment 1	330	SOUTH	100	3	2	2	0
Segment 2	330	SOUTH	100	3	3	3	0
Segment 3	330	SOUTH	100	5	5	5	0
Segment 4	330	SOUTH	100	7	3	0	3
Segment 5	330	NORTH	17	13	6	5	0
Segment 6	330	NORTH	100	10	4	3	0
Segment 7	330	NORTH	100	10	5	3	0
Segment 8	330	NORTH	8	20	5	3	0
Total	**2640**			**71**	**33**	**24**	**3**
Average	**330**		**78**	**9**	**4**	**3**	**0**

This bridge was constructed in 1996 using box girders. The data obtained for evaluation was taken from span one of this bridge.

Table 4-19. Compiled Data on Barrier Distress

Bridge ID S26-82022 (1)	Segment Length (Inches) (2)	Exposure (3)	Map cracking (%) (4)	Total number of vertical cracks (5)	Number of vertical cracks > 0.01 in. (6)	Number of vertical cracks >0.015 in. (7)	Total number of horizontal cracks (8)
Segment 1	300	SOUTH	100	14	14	6	0
Segment 2	300	SOUTH	46	13	12	4	0
Segment 3	300	SOUTH	46	8	8	5	0
Segment 4	291	SOUTH	46	8	4	1	0
Segment 5	300	NORTH	29	9	4	2	0
Segment 6	300	NORTH	55	9	4	3	0
Segment 7	300	NORTH	46	5	5	4	0
Segment 8	291	NORTH	100	5	4	2	0
Total	**2382**			**71**	**55**	**27**	**0**
Average	**298**		**59**	**9**	**7**	**3**	**0**

This bridge was constructed in 1997 using box girders. The data obtained for evaluation was taken from span one of this bridge.

Table 4-20. Compiled Data on Barrier Distress

Bridge ID S27-41064 (1)	Segment Length (Inches) (2)	Exposure (3)	Map cracking (%) (4)	Total number of vertical cracks (5)	Number of vertical cracks > 0.01 in. (6)	Number of vertical cracks >0.015 in. (7)	Total number of horizontal cracks (8)
Segment 1	396	SOUTH	0	4	4	3	0
Segment 2	396	SOUTH	0	6	6	4	0
Segment 3	396	SOUTH	0	5	5	3	0
Segment 4	396	SOUTH	0	5	5	2	0
Segment 5	396	NORTH	0	2	2	2	0
Segment 6	396	NORTH	0	6	6	3	0
Segment 7	396	NORTH	0	4	4	2	0
Segment 8	396	NORTH	0	4	4	3	0
Total	**3168**			**36**	**36**	**22**	**0**
Average	**396**		**0**	**5**	**5**	**3**	**0**

This bridge was constructed in 1997 using PCI girders. The data obtained for evaluation was taken from span two of this bridge.

Table 4-21. Compiled Data on Barrier Distress

Bridge ID S28-41064 (1)	Segment Length (Inches) (2)	Exposure (3)	Map cracking (%) (4)	Total number of vertical cracks (5)	Number of vertical cracks > 0.01 in. (6)	Number of vertical cracks >0.015 in. (7)	Total number of horizontal cracks (8)
Segment 1	396	NORTH	0	4	4	4	0
Segment 2	396	NORTH	0	6	6	5	0
Segment 3	396	NORTH	0	6	6	1	0
Segment 4	396	NORTH	0	4	4	1	0
Segment 5	396	SOUTH	0	6	3	2	0
Segment 6	396	SOUTH	0	7	5	2	0
Segment 7	396	SOUTH	0	5	5	1	0
Segment 8	396	SOUTH	0	5	4	3	0
Total	**3168**			**43**	**37**	**19**	**0**
Average	**396**		**0**	**5**	**5**	**2**	**0**

This bridge was constructed in 1997 using PCI girders. The data obtained for evaluation was taken from span two of this bridge.

Table 4-22. Compiled Data on Barrier Distress

Bridge ID S12-63172 (1)	Segment Length (Inches) (2)	Exposure (3)	Map cracking (%) (4)	Total number of vertical cracks (5)	Number of vertical cracks > 0.01 in. (6)	Number of vertical cracks >0.015 in. (7)	Total number of horizontal cracks (8)
Segment 1	624	EAST	4	9	7	1	0
Segment 2	702	EAST	0	18	18	9	1
Segment 3	636	EAST	1	17	15	5	0
Segment 4	636	WEST	0	9	9	8	2
Segment 5	696	WEST	0	15	12	7	0
Segment 6	616	WEST	0	16	13	8	2
Total	**3910**			**84**	**74**	**38**	**5**
Average	**652**		**1**	**14**	**12**	**6**	**1**

This bridge was constructed in 1988 using steel girders. The data obtained for evaluation was taken from span three of this bridge.

Table 4-23. Summary of Inspection Data

No	Bridge ID	Year Constructed	Average Daily Traffic	Inspected Barrier Length (ft)	No. of Segments	Map Cracked Area (%)	Total # of Vertical Cracks	# of Vertical Cracks Larger Than 0.015 in	# of Horizontal Cracks	Type of Coarse Aggregate*
1	S02 - 23152	1980	22,000	323	8	0	89	37	19	Natural gravel
2	S01 - 44044	1983	8500	95	8	0	38	4	4	Limestone or dolomite
3	B02 - 66051	1985	700	149	8	1	62	6	8	
4	B01 - 66051	1985	1600	167	8	0	63	13	3	
5	S02 - 82194	1986	42,500	289	8	0	115	56	12	
6	S04 - 63101	1988	131,000	120	8	6	25	12	11	
7	S09 - 63101	1988	67,000	124	8	8	54	5	14	
8	S12 - 63172	1988	34,500	326	6	1	84	38	5	Natural gravel
9	S15 - 63172	1988	29,000	222	6	1	56	30	15	Crushed &Natural gravel
10	S08 - 82191	1989	1971	122	8	2	49	23	21	
11	S04 - 82022	1993	52,000	145	8	28	55	5	8	Slag
12	S06 - 82022	1993	54,500	146	8	65	132	2	36	Slag
13	B01 - 50021	1994	37,000	120	8	100	64	2	6	
14	S24 - 82022	1996	44,500	220	8	78	71	24	3	
15	S26 - 82022	1997	58,000	199	8	59	71	27	0	
16	S12 - 63022	1997	12,500	133	8	3	45	3	6	
17	S28 - 41064	1997	14,900	264	8	0	43	19	0	
18	S27 - 41064	1997	16,500	264	8	0	36	22	0	
19	S04 - 63174	2001	93,500	166	8	0	27	14	0	Limestone or dolomite
20	S20 - 63174	2001	62,000	134	7	0	59	15	1	Limestone or dolomite

* - Data is extracted from the publication made by Van Dam et al. (2003).

4.6 EVALUATION OF INSPECTION DATA

The exposure direction influence was the first evaluation. The observed distress types of map cracking, vertical cracking, and horizontal cracking are grouped with respect to barrier exposure directions (Table 4-24 through Table 4-27). From the data presented in the tables, barriers exposed to north and south directions show significantly more map cracking when compared to east and west exposure directions. There is no observed relation between map cracking percent per segment and the bridge age. The data in the tables also show that average number of vertical cracks per segment is approximately constant and independent of the exposure direction and bridge age. It was hypothesized that the barriers with traffic side exposed to the south will have rapid deterioration due to increased freeze-thaw cycles. However, the data does not support this hypothesis.

Further data evaluation was performed only considering the average number of full-length vertical cracks per barrier segment. This analysis interestingly showed that the ratio of crack spacing to barrier height in Michigan is approximately two (Table 4-28). This table also contains the inspection data gathered from a newly constructed slipformed bridge barrier inspected two days following placement.

The coarse aggregate strength and modulus used in barrier construction is an important parameter in assessing the cracking potential of concrete. The core samples obtained from the barriers of eight inspected bridges indicated that the slag aggregate was used only in two bridges and natural and crushed limestone were used in the other six (Table 4-23). Because of the lack of data, conclusions on the effects of slag aggregate on premature barrier deterioration were not made.

Table 4-24. Distress of Barriers with North Exposure

No.	Bridge ID	Construction Year	Average Length of Barrier Segment (inches)	Average Map Cracking per Segment (%)	Average Vertical Cracks per Segment	Average Horizontal Cracks per Segment
1	S01-44044	1983	147	0	4	0
2	B01-66051	1985	251	0	7	0
3	B02-66051	1985	224	2	6	1
4	S04-63101	1988	178	0	3	1
5	S09-63101	1988	175	13	3	2
6	S08-82191	1989	181	0	7	3
7	S04-82022	1993	218	22	9	0
8	S06-82022	1993	218	100	25	4
9	B01-50021	1994	180	100	3	1
10	S24-82022	1996	330	56	13	0
11	S26-82022	1997	298	58	7	0
12	S12-63022	1997	199	6	6	1
13	S27-41064	1997	396	0	4	0
14	S28-41064	1997	396	0	5	0

Table 4-25. Distress of Barriers with South Exposure

No.	Bridge ID	Construction Year	Average Length of Barrier Segment (inches)	Average Map Cracking per Segment (%)	Average Vertical Cracks per Segment	Average Horizontal Cracks per Segment
1	S01-44044	1983	141	1	6	1
2	B01-66051	1985	251	0	9	1
3	B02-66051	1985	225	1	9	2
4	S04-63101	1988	181	12	3	2
5	S09-63101	1988	198	4	10	1
6	S08-82191	1989	185	4	5	2
7	S04-82022	1993	217	35	5	2
8	S06-82022	1993	219	30	9	5
9	B01-50021	1994	181	100	13	1
10	S24-82022	1996	330	100	5	1
11	S26-82022	1997	298	60	11	0
12	S12-63022	1997	200	0	5	0
13	S27-41064	1997	396	0	5	0
14	S28-41064	1997	396	0	6	0

Table 4-26. Distress of Barriers with West Exposure

No.	Bridge ID	Construction Year	Average Length of Barrier Segment (inches)	Average Map Cracking per Segment (%)	Average Vertical Cracks per Segment	Average Horizontal Cracks per Segment
1	S02-23152	1980	485	0	12	3
2	S02-82194	1986	434	0	14	4
3	S12-63172	1988	649	0	13	1
4	S15-63172	1988	436	2	8	2
5	S04-63174	2001	249	0	2	0

Table 4-27. Distress of Barriers with East Exposure

No.	Bridge ID	Construction Year	Average Length of Barrier Segment (inches)	Average Map Cracking per Segment (%)	Average Vertical Cracks per Segment	Average Horizontal Cracks per Segment
1	S02-23152	1980	484	0	10	2
2	S12-63172	1988	654	2	15	0
3	S15-63172	1988	452	1	11	2
4	S20-63174	2001	230	0	8	0
5	S04-63174	2001	248	0	5	0

Table 4-28. Ratio of Crack Spacing to Barrier Height of Existing Bridge Barriers

Bridge ID (1)	Year Constructed (2)	Average Length of Barrier Segment (inches) (3)	Average Number of Full-Length Vertical Cracks per Segment (4)	Crack Spacing/ Barrier Height (5)
S02-23152	1980	484	5	2
S01-44044	1983	144	1	2
B01-66051	1985	251	2	2
B02-66051	1985	224	1	3
S02-82194	1986	434	6	2
S12-63172	1988	652	5	3
S09-63101	1988	187	1	2
S04-63101	1988	179	1	2
S15-63172	1988	444	3	3
S08-82191	1989	183	1	2
S06-82022	1993	218	1	3
S04-82022	1993	217	2	2
B01-50021	1994	180	2	2
S24-82022	1996	330	3	2
S26-82022	1997	298	4	2
S28-41064	1997	396	2	3
S27-41064	1997	396	3	2
S12-63022	1997	200	1	2
S20-63174	2001	230	2	2
S04-63174	2001	249	1	3

4.7 CONCLUSIONS

A total of 20 bridges consisting of 155 barrier segments were inspected. Only two bridges were on a new road section not yet open to traffic. Thirteen bridges out of 20 were on interstate highways where the average daily traffic ranged from 8,500 to 131,000.

Field inspection data agrees with the findings described in the literature, showing that vertical cracks are most often the originator of other distress. The full-length vertical crack data compiled from the inspection data shows that the ratio of crack spacing to barrier height is equal to two.

Horizontal cracks are classified as either local or continuous. The local and continuous horizontal cracks were mostly observed on the vertical face, at about the level of the top longitudinal reinforcement. Barrier surfaces at the intersections of horizontal and vertical cracks were always in highly distressed condition. This observation was made only on barrier segments noted to be in poor condition. Concrete was spalling and disintegrating above the horizontal cracks on portions of barrier segments. Section loss may create a safety hazard due to falling debris. Hence, the barriers with continuous horizontal cracks require immediate attention.

Exposure of barrier segments is based on the direction of the traffic side of the barrier. Out of 155 inspected barrier segments, 56 segments with a total length of 1,129 ft. were exposed to the north, 56 segments with a total length of 1,140 ft. were exposed to the south, 21 segments with a total length of 655 ft. were exposed to the east, and 22 segments with a total length of 805 ft. were exposed to the west. Data indicated that barriers exposed to north and south directions show increased map cracking. No relationship was found between the barrier age and the amount of map cracking. It was also hypothesized that barriers with south exposure will have increased freeze-thaw cycles, thus more rapid deterioration. The data did not support this hypothesis.

5 CONSTRUCTION MONITORING

5.1 CONSTRUCTION SITES

Four bridge reconstruction projects with New Jersey barriers were monitored. Consideration was given to construction dates and locations, one being a late season project. Formwork and reinforcement cages were inspected and concrete placement was monitored. Following the placement day, the barriers were reinspected and the standard specimens prepared during placement were collected. The list of monitored construction projects is given below in Table 5-1:

Table 5-1. List of Reconstructed Bridges Monitored during Barrier Placement

Bridge ID	Location	Construction Procedure
S05 of 82191	Vreeland Road over I-75	Form-cast
S06 of 82194	I-75over Fort Street	Slipformed
S26 of 50111	I-94 over Metro Parkway	Form-cast
S20 of 50111	I-94 over Little Mack	Slipformed

5.2 PRE-PLACEMENT OBSERVATIONS

5.2.1 Specification Requirements

The Michigan Department of Transportation - Standard Specifications for Construction (specbook@2003) was followed as a reference for observations in the field. Specification requires all structural concrete forms to be mortar tight and sufficiently rigid for concrete placing, consolidating, and curing. Forms are used to classify the concrete surfaces as exposed (Type A) and unexposed (Type B). Any defect on a barrier surface due to either plywood or steel forms is not allowed. In addition, the size, spacing, and dimensions of forms must meet standard dimensions for Type 4 as described in the MDOT Bridge Design Guide. All dust and debris is to be removed from the interior of the forms. In order to get proper surface and service condition, the condition of the forms is required to be maintained until the concrete has sufficiently hardened.

Steel reinforcement is to be epoxy-coated and free from dust, rust, and coating damage and securely tied at all intersections. The Standard Specifications for Construction allows field bending of epoxy-coated bars as provided in the plans or to correct minor errors or omissions in shop bending. If epoxy coating becomes damaged during correction or placement, it must be repaired.

5.2.2 Field Observations

The day before the concrete placement, sites were visited to check the formwork and reinforcement. Removable forms (steel forms on traffic bearing side and wooden forms on the other side) were used in form-cast barrier construction. Figure 5-1 shows the barrier and reinforcement geometry and the specified reinforcement cover for New Jersey Type 4 barriers. Observations show that the reinforcements were not tied at all intersections as required by the specifications (Photo 5-1). In general, fabrication of reinforcement appeared satisfactory, yet some problems with inconsistent cover for the transverse reinforcement were observed (Photo 5-2).

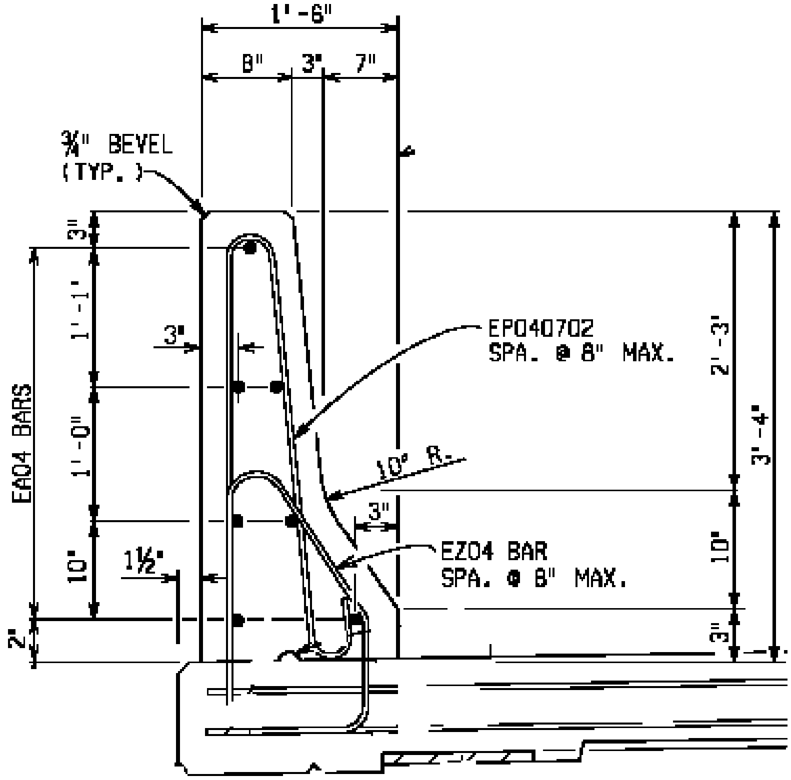


Figure 5-1. Cross-section of New Jersey Type 4 bridge barrier railing

Photo 5-1. Reinforcement ties missing at some rebar intersections (bridge S06–82194)

Photo 5-2. Formwork of barriers of bridge S05–82191

(Note that rebar cover is inconsistent and almost touching the traffic side forms)

5.3 PLACEMENT OBSERVATIONS

5.3.1 Specification Requirements

In the MDOT Standard Specifications for Construction, a slipforming procedure is not described. However, there is a definition for machine finishing. Specification requires a test run to adjust the finishing position of the paver, deflections, and to measure the depth of concrete cover. All necessary corrections are to be made before concrete placement.

The specification indicates that in form-cast barrier construction, form bracings that are temporarily placed into the cross-section of members must be removed before concrete sets. Mechanical high-frequency internal vibration is suggested for consolidation of structural concrete. In using a vibrator, a rubber-coated apparatus must be used to avoid damage to epoxy coated bars. The duration of vibration is required to be adequate to consolidate the concrete, but not to cause segregation. The vibration is to be applied no further apart than twice the radius over which the vibration is visibly effective, and spaced uniformly. Vibration is required to be directly applied to concrete, rather than to forms and reinforcement.

Specifications place weather restrictions during concrete placement based on relative humidity and wind velocity during hot and cold weather. When conditions are outside the acceptable range, a check for evaporation is required. If the evaporation rate is more than 0.2 psf per hour, placement of concrete must be delayed. Maximum allowable concrete temperature during placement is 90 °F and for cold weather conditions minimum air temperature is 40°F.

5.3.2 Field Observations of Barrier Construction

On the placement day, the fresh concrete tests were performed. Standard specimens were prepared according to ASTM C 31 for documenting properties of hardened concrete. The concrete mix design data was obtained from the mix-truck ticket data. The time of casting, batching, delivery, and casting period were calculated using the ticket information. Two of the monitored barrier reconstructions were form-cast and the other two were slipformed (Table 5-1).

5.3.2.1 Form-Cast Construction

Molds were placed aligned with the deck edges and spacing equipment was not used. Rebar cover was less than 1 inch at some locations (Photo 5-2). The form interiors were not clean and contained some sawdust, wood chips, and other debris (Photo 5-3). The outer surface of the barrier mold was textured wood and the traffic side was steel (Photo 5-2 and Photo 5-4). Concrete was placed either using a bucket or directly from the mix-truck chute (Photo 5-5 and Photo 5-6). Concrete placement was continuous with no disruptions during placement. A mechanical internal vibrator was used to consolidate concrete (Photo 5-7). Steel bracing fixtures were used for supporting the barrier forms (Photo 5-8). The bracings were cut during form removal, and the braces remained in the hardened concrete.

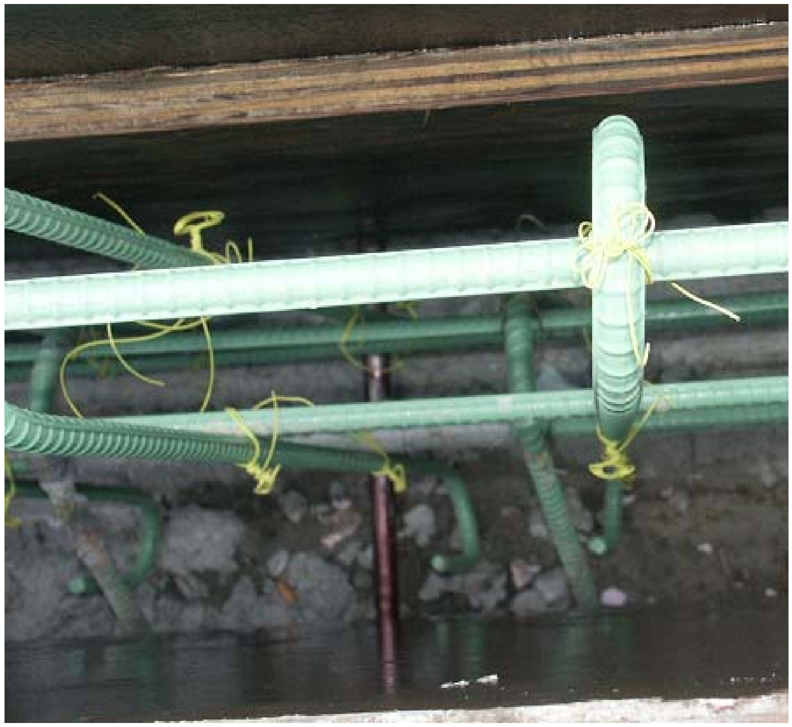


Photo 5-3. Debris inside the forms (S05–82191)

Photo 5-4. Steel molds on the traffic side (S05-82191)

Photo 5-5. Concrete placement with bucket (S05-82191)

Photo 5-6. Concrete placement directly from mix-truck (S20-50111)

Photo 5-7. Concrete consolidation using vibrator (S20-50111)

(a)

(b)

Photo 5-8. (a) Steel bracing connected to barrier reinforcement and (b) steel bracing for the alignment of formwork (S26-50111)

5.3.2.2 Slipformed Construction

Prior to concrete placement, a test run was performed with the paver to adjust the finishing position and to check the reinforcement alignment (Photo 5-9). However, the crew dealt only with the reinforcements that were interfering with the paver path. The reinforcements were realigned by hammering into proper positions (Photo 5-10). After hammering, the potential damage to the epoxy coating were not inspected and repaired. Placement was at a relatively rapid pace considering the low workability of concrete. Vibration was applied externally. The vibration was not sufficient in duration and intensity to consolidate the low slump concrete. Inadequate consolidation observed as rock pockets and cavities and some plastic flow from the molded cross-section were observed. In addition, the surface of the freshly placed concrete was fairly rough and required extensive floating. The required cross-section was achieved by striking as allowed by the Specifications. Final surface finishing was performed with wooden and/or metal floats (Photo 5-11).

Photo 5-9. Test run of the paver (S06-82194)

Photo 5-10. Realigning rebars by hammering (S06-82194)

Photo 5-11. Surface finishing of slipformed barriers (S06-82194)

5.3.3 Curing of Concrete

Specifications only require wet curing of bridge decks. According to specifications, vertical forms must be left in place for at least 15 hours following the completion of concrete placement. The concrete must be cured upon form removal. If the air temperature is below 40 °F, then the low temperature concrete protection procedure is required. If the air temperature is over 40°F, then curing compound should be applied immediately upon completion of concrete finishing. The curing compound (ASTM C 309 Type 2) is applied in two coats. The second coat should be applied within two hours (if the first coat has sufficiently dried) after the first coat. For barriers, curing compound can be applied using a brush, roller or sprayer. The curing compound must form a continuous uniform film without running or sagging. Prior to the application of curing compound, all surface defects (honeycombs, broken corners, cavities, etc.) and all holes more than ¾ inch in diameter must be repaired with mortar.

5.3.4 Curing of Concrete - Field Observations

5.3.4.1 Form-Cast Construction

Forms were removed approximately 18 hours upon concrete placement, even for the late season placement. The surface defects were repaired with a mortar (Photo 5-12). Repair was only performed on the traffic side surface of barrier in order to satisfy MDOT exposed surface regulations. Close inspection of the barrier did not show any vertical, horizontal, map or local cracking upon form removal. The control joints were saw-cut on the traffic side of the barrier (Photo 5-13). Curing compound was not applied anytime during the day after the form removal (Photo 5-14); at that time concrete was exposed to very hot summer temperatures.

Photo 5-12. Repaired surface voids of form-cast barrier surface (S05-82191)

Photo 5-13. Saw-cut joint of a form-cast barrier (S05-82191)

Photo 5-14. Barrier surface without curing compound the day after concrete placement (S05-82191)

5.3.4.2 Slipformed Construction

Following slipforming numerous defects were visible on the concrete surface (Photo 5-15). Portions of the newly slipformed concrete around the expansion joints collapsed and later hand repaired (Photo 5-16). Barriers were cast in a continuous process and later joints were hand-cut while concrete is in a plastic state (Photo 5-17). The joints were made by inserting styrofoam sheets along the barrier sections prior to extrusion. The styrofoam sheet also served as the backing while hand repairing the joint (Photo 5-16). Upon the removal of the styrofoam backing from the joint, honeycombing and cavities were visible in the barrier interior as seen in Photo 5-18.

A single coat of curing compound was sprayed in one pass approximately three hours after slipforming. During that period, concrete was exposed to extremely hot and sunny environmental conditions. The curing compound formed drip marks when applied to the vertical surfaces of the barrier (Photo 5-19) and the coverage was not uniform (Photo 5-19 and Photo 5-20).

A site inspection visit was made two days after placement. Map cracking and vertical cracking on all barrier segments were revealed (Photo 5-20 and Photo 5-21). All control joints had formed full-length vertical cracks. Additional vertical cracks were observed on the barriers segments. It should also be noted that the traffic flow next to the newly cast barrier created significant vibrations (Photo 5-22).

Photo 5-15. Visible surface defects after extrusion in slipformed barriers (S06-82194)

Photo 5-16. Portions of plastic concrete near the construction joint of slipformed barrier broke away and later hand repaired (S06-82194)

Photo 5-17. Hand finish joint of a slipformed barrier (S06-82194)

Photo 5-18. Honeycombing and large voids were visible at the joint of slipformed barrier (S06-82194)

Photo 5-19. Application of the one and only layer of curing compound on barrier surface (S06-82194)

Photo 5-20. Map cracking of barrier surface observed 48 hours after slipforming (S06-82194)

(Note: absence of curing compound)

Photo 5-21. Full-length vertical crack on the barrier observed 48 hours after slipforming (S06-82194)

(Note: the lack of curing compound on barrier surface)

Photo 5-22. Traffic generating vibrations while slipforming of barriers (S06-82194)
(Note: prior to the curing compound application)

5.4 CONCLUSIONS

Construction and curing of concrete bridge barriers are not specifically addressed in the MDOT Standard Specifications for Construction. Concrete was placed often during midday without any attention to air temperatures at the time. With regard to form-cast construction, after concrete placement, the barrier top surface was not covered or protected from direct exposure. Forms were removed approximately 18 hours following construction as allowed by the specifications, but at that time concrete was exposed to hot summer temperatures; and a curing compound was not applied upon form removal. A quick inspection upon form removal did not reveal any obvious cracking.

In slipforming barriers primary concerns were related to consolidation and curing. Though it is difficult to investigate the barrier interior without taking core samples, removal of one-inch thick styrofoam joint bulkhead revealed honeycombing and cavities inside the barrier. The curing compound was sprayed, but curing compound did not form a uniform layer over the barrier surface. Uneven application and drip marks were observed. The non-uniform application and the dripping could be due to the fact that the sprayer adjustments may not be suitable for application of curing compound on vertical surfaces. Uneven application of curing compound and early form removal promotes rapid loss of mix water, and a high temperature gradient between the interior and exterior of the concrete barrier will be generated. This is consistent with observed premature distresses such as map cracking and vertical cracking. The observed full-length vertical cracking is also due to thermal and shrinkage effects, while map cracks are due to plastic shrinkage. The most probable cause of cracking observed near the top of the barrier is perhaps due to the plastic settlement of insufficiently consolidated concrete. Vibration generated by the traffic adjacent to the newly cast barriers could contribute to the settlement of plastic concrete. Based on overall observations, the following conclusions are made with respect to construction practices:

1. The MDOT Standard Specifications for Construction should clearly address barrier construction.
2. Bridge barriers should receive the same attention as other structural components in terms of construction and curing practices (e.g., night casting is the practice for bridge decks in summer but on the same bridge deck barriers are cast during daytime; wet curing is required for bridge decks but only curing compound is for bridge barriers).
3. In order to continue the use of slipforming, consolidation and curing practices should be improved.
4. In form-cast construction, forms should be kept in place for a minimum of 3 days and curing compound should be applied to the exposed top surface immediately upon placement.
5. Curing compound should be applied immediately following form removal with a roller or brush (not with the sprayer as allowed by the specifications).
6. Traffic flow next to the newly cast barriers can be isolated otherwise reduced speed can be imposed until concrete gains sufficient strength.

6 LABORATORY TESTING

6.1 OVERVIEW

The concrete specimens tested consisted of 6-inch and 4-inch diameter standard cylindrical specimens prepared during construction and 4-inch diameter specimens of various lengths cored from existing barriers. Mechanical and other physical properties related to concrete durability are obtained through several standard tests.

Mechanical properties of concrete are obtained from compressive strength and elasticity modulus tests in accordance with ASTM C 39 and ASTM C 469, respectively. The compressive strength test is the most common test performed on hardened concrete. There is a strong correlation between the durability properties of concrete and its compressive strength. This test is used to document material performance and can help to establish the mixture proportion to attain the required strength. The strength test is performed on standard cylinder specimens made from concrete in the field. Elasticity modulus and Poisson's ratio tests provide a value for stress to strain proportionality and a ratio of lateral to longitudinal strain for hardened concrete most commonly at 28 days. The modulus of elasticity and Poisson's ratio values are applicable in stress ranges from 0 to 40% of the ultimate concrete strength. Elasticity modulus is needed for computing the stress from intrinsic strains. In addition, by conducting compressive strength, elasticity modulus, and Poisson's ratio tests at different ages of concrete, the change in mechanical properties with respect to time is established.

The ultrasonic pulse velocity (UPV) test measures the velocity of stress waves propagating through the concrete specimen. The pulse velocity of stress waves in concrete is related to its elastic properties and density. This test method is often used to evaluate the uniformity and relative quality of concrete and to indicate the presence of voids and cracks. It may also be used as evidence to the changes in concrete properties. The elasticity modulus can be calculated from measured ultrasonic pulse velocity. This test is performed in compliance with ASTM C 597.

The rapid chloride permeability test (RCPT) is performed in accordance with ASTM C 1202. This test is useful in determining the electrical conductance of concrete and provides an indication of its resistance to penetration of chloride ions. The absorption and air permeability tests are performed in developing the data required for determining the volume of permeable voids and limits of absorption. Concrete with large or many permeable voids is not desirable due to its lack of ability to protect itself from environmental attacks (i.e. chloride ion penetration); therefore, it is ideal to obtain a low pore-volume ratio for quality concrete. The absorption test is performed in compliance with ASTM C 642. The air-permeability test is performed using an apparatus not yet standardized by ASTM.

The rapid chloride permeability, absorption, gas and water permeability, and ultrasonic pulse velocity tests were performed on the standard specimens made during construction and the core specimens obtained from existing barriers.

6.1.1 Standard Specimens Prepared during Construction

A comprehensive list, which enumerates the tests performed on the standard cylinder specimens and the required number of samples, is given in Table 6-1. Compressive strength, elasticity modulus, Poisson's ratio, rapid chloride permeability test (RCPT), ultrasonic pulse velocity (UPV), air-permeability, and absorption tests were conducted and the test results are shown in Table 6-2, Table 6-3, Table 6-4, Table 6-5, Table 6-6, Table 6-7, and Table 6-8, respectively.

Table 6-1. The Conducted Tests and Required Number of Standard Specimens

Tests	Type of Specimen	Number of Specimens	Test Days					Total Number of Specimens
			3	7	28	56	90	
Compressive Strength (ASTM – C 39)	6 x 12 in	4	x	x	x	x	x	20
Modulus of Elasticity (ASTM – C 469)	6 x 12 in	4	2	2	x	x	x	12
UPV (ASTM – C 597)	4 x 8 in	12	x	x	x	x	x	12
Air-Permeability	4 x 2 in	4			x	x	x	12
RCPT (ASTM – C 1202)	4 x 2 in	4			x	x	x	12
Absorption (ASTM –C 642)	4 x 2 in	4			x	x	x	12

6.1.2 Core Specimens from Existing Barriers

The core samples were obtained from a limited number of barriers. All cored specimens were first processed in the laboratory by cleaning, documenting the state, taking measurements, and photologging. Two core samples from each bridge were taken to Michigan Technological University for petrographic and microstructural investigations. Remaining core specimens were tested for ultrasonic pulse velocity (UPV), air-permeability, rapid chloride penetration (RCPT), and absorption. Test results are shown in Table 6-9, Table 6-10, Table 6-11, and Table 6-12.

6.1.3 Test Procedures for Standard Specimens Prepared during Construction

Standard specimens (minimum thirty two 6-in.x12-in. and twenty four 4-in.x 8-in.) were prepared during the placement of barrier concrete for laboratory testing (Table 6-1). The specimens remained undisturbed at the site for 24 hours before transporting to the lab. Specimens were labeled, cataloged, and wet cured in $70^{o}F$ water bath for 28 days. The specimens were kept in ambient laboratory air after the 28^{th} day. Compressive strength, modulus of elasticity, Poisson's ratio, and ultrasonic pulse velocity tests were conducted at the ages of 3, 7, 28, 56, and 90 days. Rapid chloride permeability (RCPT), absorption, and air-permeability tests were conducted at the ages of 28, 56, and 90 days. The laboratory data sheets are presented in Appendix B.

6.1.4 Test Procedures for Core Specimens

The petrographic test procedures are described in the report titled, "*Causes and Cures for Cracking of Concrete Barriers"* by Van Dam et al. (2003). Remaining cores were labeled, cataloged, and visually inspected. The qualitative observations made during this inspection helped to provide an overall assessment of the concrete. Core samples were photologged for later reference.

Core samples were prepared for UPV, RCPT, air-permeability, and absorption tests. The broken portions of the specimens were reattached with epoxy. Sample ends were saw cut to form straight polished ends. After the UPV test, core samples were again saw cut into two-inch thick portions for air-permeability, RCPT, and absorption tests.

6.2 TEST RESULTS

6.2.1 Standard Specimens Prepared during Construction

Table 6-2, Table 6-3, and Table 6-4 show mean values and coefficient of variance for compressive strength, elasticity modulus, and Poisson's ratio, respectively. RCPT, UPV, air permeability, and absorption test results are given in Table 6-5, Table 6-6, Table 6-7, and Table 6-8, respectively.

All the tests were conducted according to ASTM Standards with the exception of the air permeability test. These Standards are given in Table 6-1.

Table 6-2. Compressive Strength Test Results of Standard Specimens

Bridge ID	Compressive Strength (psi)									
	$f'_{c,3}$	COV (%)	$f'_{c,7}$	COV (%)	$f'_{c,28}$	COV (%)	$f'_{c,56}$	COV (%)	$f'_{c,90}$	COV (%)
S05 of 82191	3685	2.9	4160	4.9	4740	1.0	-	-	5795	3.5
S06 of 82194	2603	0.8	2893	2.9	3380	3.4	4098	4.1	4097	4.3
S26 of 50111	-	-	-	-	6473	1.8	7670	2.6	7920	1.3
S20 of 50111	-	-	5775	-*	7063	2.0	8338	2.4	8931	0.2

- Missing data

-* -Only two readings are available

Table 6-3. Modulus of Elasticity Results of Standard Specimens

Bridge ID	Modulus of Elasticity (ksi)									
	$E_{,3}$	COV (%)	$E_{,7}$	COV (%)	$E_{,28}$	COV (%)	$E_{,56}$	COV (%)	$E_{,90}$	COV (%)
S05 of 82191	5209	-*	5267	-*	5132	-**	-	-	4928	0.4
S06 of 82194	4131	-*	4241	-*	4520	3.9	4174	4.5	3841	2.4
S26 of 50111	-	-*	-	-*	4967	1.6	4793	1.3	4755	2.1
S20 of 50111	-	-*	-	-*	5418	3.4	5386	1.6	5257	1.5

- Missing data

-* - Only two readings are available

-** - Only three readings are available

Table 6-4. Poisson's Ratio Test Results of Standard Specimens

Bridge ID	Poisson's Ratio									
	$\nu_{,3}$	COV (%)	$\nu_{,7}$	COV (%)	$\nu_{,28}$	COV (%)	$\nu_{,56}$	COV (%)	$\nu_{,90}$	COV (%)
S05 of 82191	0.25	-*	0.25	-*	0.24	-**	-	-	0.24	2.0
S06 of 82194	0.24	-*	0.24	-*	0.24	4.4	0.24	4.7	0.22	5.5
S26 of 50111	-	-*	-	-*	0.25	1.7	0.26	8.0	0.24	4.7
S20 of 50111	-	-*	-	-*	0.25	0.1	0.24	3.8	0.25	2.7

- Missing data

-* - Only two readings are available

-** - Only three readings are available

Table 6-5. RCPT Test Results of Standard Specimens

Bridge ID	Test Age (days)	RCPT (Coulombs)					COV (%)
		#1	#2	#3	#4	Mean	
S05 of 82191	**28**	4830	4670	5700	4765	**4994**	10
	56	4770	4370	4940	3700	**4445**	12
	90	5380	9720	8470	6340	**7478**	26
S06 of 82194	**28**	3880	3290	3350	3460	**3495**	8
	56	-	7520	7020	-	**7270**	-
	90	-	-	-	-	-	-
S26 of 50111	**28**	-	9220	-	-	-	-
	56	-	-	-	-	-	-
	90	8130	5850	6410	6780	**6790**	14
S20 of 50111	**28**	-	10660	-	-	-	-
	56	5485	5955	6210	5740	**5850**	5
	90	7700	7020	7800	6380	**7230**	9
S05 of 82025	**28**	-	-	-	-	-	-
	56	5390	9080	7700	7630	**7440**	21
	90	6500	6730	9930	7190	**7590**	21

- Missing data

Table 6-6. UPV Test Results of Standard Specimens

Bridge ID	UPV Speed (in/sec)				
	3 Days	7 Days	28 Days	56 Days	90 Days
S05 of 82191	–	–	194,000	196,000	196,000
S06 of 82194	–	190,000	195,000	196,000	196,000
S26 of 50111	184,000	187,000	190,000	194,000	193,000
S20 of 50111	186,000	190,000	193,000	197,000	197,000

- Missing data

Table 6-7. Air-Permeability Test Results of Standard Specimens

Bridge ID	Test Age (Day)	Intrinsic Gas Permeability (in^2) 10^{-13}					COV (%)
		#1	#2	#3	#4	Mean	
S05 of 82191	28	3.4	3.1	2.3	2.9	**2.9**	15.5
	56	3.1	3.3	3.1	3.6	**3.3**	6.7
	90	7.3	8.5	6.7	7.0	**7.4**	11.0
S06 of 82194	28	1.6	4.7	2.2	1.7	**2.5**	57.4
	56	-	4.3	3.1	1.9	**3.1**	-
	90	4.3	7.3	10.7	2.2	**6.1**	60.4
S26 of 50111	28	2.6	2.9	3.7	2.8	**3.0**	16.0
	56	11.3	8.8	8.5	9.8	**9.6**	13.0
	90	7.8	9.8	9.6	10.5	**9.4**	12.6
S20 of 50111	28	-	2.9	3.6	2.5	**3.0**	-
	56	4.8	4.5	-	-	**4.7**	-
	90	11.0	5.4	3.9	2.9	**5.8**	62.1

- Missing data

Table 6-8. Absorption Test Results of Standard Specimens

Bridge ID	Volume of Permeable Pore Space (Voids), %					
	28 day	COV (%)	56 day	COV (%)	90 day	COV (%)
S05 of 82191	11.5	4.2	12.3	2.3	12.9	4.9
S06 of 82194	11.4	4.8	12.8	3.1	13.6	4.3
S26 of 50111	11.9	8.8	11.8	5.8	13.1	4.0
S20 of 50111	11.4	2.2	12.2	2.9	8.8	8.8

6.2.2 Core Specimens

The tests performed on core specimens were rapid chloride permeability, ultrasonic pulse velocity, air permeability, and absorption. The results of these tests are given in Table 6-9, Table 6-10, Table 6-11, and Table 6-12, respectively.

The petrographic test results and microstructural observations are described in the report titled, "*Causes and Cures for Cracking of Concrete Barriers"* by Van Dam et al. (2003).

Table 6-9. RCPT Test Results of Core Specimens

Bridge ID	RCPT (Coulombs)					
	#1	#2	#3	#4	Mean	COV (%)
S02 of 23152	-	-	-	-	-	-
S01 of 44044	2010	1760	5075	2760	**2900**	52
S12-1 of 63172	1490	1150	1375	1320	**1335**	11
S04-4 of 82022	3030	2020	2750	3120	**2730**	18
S04-2 of 63174	5080	5010	3930	5150	**4790**	12

\- Missing data

Table 6-10. UPV Test Results of Core Specimens

Bridge ID	UPV Speed (in/sec)	
	Mean	COV (%)
S02 of 23152	188,000	3.6
S01 of 44044	189,000	3.5
S12-1 of 63172	193,000	12.7
S15 of 63172	193,000	5.3
S04-4 of 82022	185,000	2.3
S06 of 82022	190,000	1.7
S04-2 of 63174	182,000	2.4
S20-1 of 63174	178,000	1.8

Table 6-11. Air-Permeability Test Results of Core Specimens

Bridge ID	Intrinsic Gas Permeability (in^2) 10^{-13}					COV (%)
	# 1	# 2	# 3	# 4	Mean	
S02 of 23152	-	-	-	1.4	-	-
S01 of 44044	0.8	0.5	0.6	0.6	**0.6**	16.0
S12-1 of 63172	-	-	-	0.5	-	-
S04-4 of 82022	0.7	1.6	-	-	**1.1**	-
S04-2 of 63174	0.6	0.9	2.5	2.2	**1.5**	61.7

- Missing data

Table 6-12. Absorption Test Results of Core Specimens

Bridge ID	Volume of Permeable Pore Space (Voids), %					
	#1	#2	#3	#4	Mean	COV (%)
S02 of 23152	12.5	12.0	10.5	11.0	**11.4**	7.2
S01 of 44044	12.2	11.9	11.5	11.4	**11.7**	3.0
S12-1 of 63172	9.9	10.9	8.9	10.9	**10.2**	9.3
S04-4 of 82022	14.6	13.7	14.0	13.3	**13.9**	3.9
S04-2 of 63174	13.6	13.1	11.9	10.3	**12.2**	12.3

6.3 CONCLUSIONS

The compressive strength and elasticity modulus of standard specimens were measured at 3, 7, 28, 56 and 90 days. Test results of standard specimens showed that two of the bridge barrier concrete possessed 28–day strength in excess of 6000 psi. Concrete with a 28-day compressive strength greater than 6000 psi is typically defined as high-strength concrete that requires special placement and curing procedures. Concrete compressive strength and elasticity modulus test results that were obtained at 3, 7, 28, 56, and 90 days were used to establish the relationship of mechanical property variation with time. The measured compressive strength and elasticity modulus are used in Chapter 7 for predicting concrete cracking stress-strain relationship.

Core samples taken from distress-free zones show air-permeability values lower than the measurements of the specimens obtained from new barriers. Samples taken from the zones of distress displayed excessive leakage that resulted in the out of bound results. The existence of cracks and large voids triggered the leakage. The permeable pore volume measured from the absorption test does not reveal any significant difference between cores and standard samples. UPV measurements on the cores and standard specimens also show similar results. The Coulomb values measured on standard specimens are significantly higher than that of core specimens. The lack of correlation of test results indicates that there is a significant variation in concrete properties obtained from cores and standard specimens as well as within the same bridge barrier.

7 PARAMETERS INFLUENCING BARRIER CRACKING

7.1 OVERVIEW

Field inspections on the condition of New Jersey Type 4 barriers on 20 selected bridges were appraised by field inspection. These barriers were selected to represent a well-distributed sample with ages of up to 22 years for the purposes of identifying the primary distress forms and their progression. Also, newly slipformed barriers of bridge S06-82194 were inspected two days after placement. The inspection data analysis results agree with the findings of the literature review; rapid barrier deterioration is initiated by early-age vertical cracking. Analysis of the inspection data showed that the average full-length vertical crack spacing of barriers is twice the barrier height.

The findings so far indicate that the deterioration of concrete barriers starts with vertical cracking which provides access to deteriorating agents into the concrete interior. The rate of barrier deterioration is a function of concrete consolidation and in general, overall concrete and construction quality. Barrier concrete deterioration rate is the lowest in well consolidated and well cured concrete with the proportions and aggregate quality specified by MDOT Standard Specifications.

Literature review and construction monitoring reveled that there are a limited number of factors that influence early-age cracking of reinforced concrete bridge barriers. The major parameter is the volume change due to shrinkage and thermal loads combined with the base restraint at the deck-barrier interface. As discussed in the previous chapters, vertical cracking that initiates due to volume change is the primary cause of other distress types. Reduction, control, or prevention of vertical cracking will improve barrier service life.

Shrinkage and thermal loads are inherent to concrete construction. Cement hydration is an exothermic process that generates heat and consequently a temperature difference between the concrete mass and the ambient air. As discussed in Chapter 2, several factors control the heat magnitude that develops within the concrete mass. At a specific time after placement, during the hydration process, the concrete mass reaches peak temperature. The concrete interior being warmer than the ambient air, a temperature gradient forms between the interior and the exterior surface of the barrier. As the hydration subsides and concrete interior starts cooling, the barrier starts shrinking and tensile stresses develop at the base of the barrier where the restraint is the highest.

Drying shrinkage initiates as soon as concrete is placed. Factors controlling drying shrinkage are described in Chapter 2. Lack of available restraint when concrete is in a plastic state prevents stresses developing from shrinkage. The maximum shrinkage strain that will develop in concrete is a constant value. The formation of cracks during shrinkage progression is a function of when shrinkage takes place. Often concrete cracks if shrinkage is not controlled during early-ages when cracking strength is low. Cracking will be controlled if wet curing is promptly initiated and shrinkage is delayed. The effectiveness of the curing compound is limited and only if an impervious membrane can be formed.

There are several shrinkage prediction models that estimate the progression of drying shrinkage following wet or moist curing. Although specific models for the prediction of very early-age drying shrinkage are not available, the formulation and discussion given in Section 2.3.2.2 verify the importance of implementing appropriate curing procedures immediately upon concrete placement.

Volume change of the barrier caused by concrete shrinkage and thermal loads in conjunction with restraint effects generate tensile stresses. In order to appraise the cracking potential of bridge barriers, shrinkage and thermal loads need to be estimated. Prediction of the shrinkage strain is based on concrete properties, barrier geometry, and ambient conditions. Thermal loads are predicted based on the concrete mix proportions, curing conditions, barrier geometry, and ambient conditions that prevailed during the time of concrete placement.

In this chapter, the potential full-length vertical crack spacing of concrete barriers under predicted shrinkage and thermal loads in combination with restraint effects is evaluated by the finite element (FE) analysis of a barrier segment. FE analysis models are generated for several barrier length/height (L/H) ratios. The purpose of the multiple models is to determine the length/height ratio at which cracking may initiate. Further, the analysis of longitudinal stress distributions with accompanying strain distributions establishes the vertical crack spacing. The reinforcements are incorporated in these models.

In order to incorporate the shrinkage, thermal effects, and restraint effects on vertical cracking, change in the mechanical properties of concrete with respect to time is required. The compressive strength and elasticity modulus tests of concrete obtained from standard specimens at 3, 7, 28, 56, and 90 days were given in Chapter 6. However, the analysis requires variation of mechanical properties of concrete against time at very early ages (from the time of concrete placement up to 2 days). Hence, various prediction models are investigated to determine the models that are capable of estimating the very early-age compressive strength, elasticity modulus, and tensile strength.

7.2 MECHANICAL PROPERTIES OF CONCRETE

The purpose of developing mechanical property prediction models is to have a continuous relationship describing concrete strength and elasticity modulus (stiffness) against time. Also, the crack initiation requires the direct tensile strength of concrete. A prediction model for direct tensile strength of concrete is also described.

7.2.1 Compressive Strength

The two concrete compressive strength prediction models compared here are described by ACI and CEB-FIP.

ACI Committee 209 recommends a model for moist-cured concrete made with normal Portland cement (ASTM Type I) as given in Eq. 7-1.

$$(f_c')_t = \frac{t}{4 + 0.85t}(f_c')_{28} \tag{7-1}$$

The following relationship is suggested by CEB-FIP Models Code (1990) for concrete specimens cured at 68 ^{0}F (Mehta 1993):

$$(f_c')_t = exp\left[s\left(1 - \left(\frac{28}{t/t_1}\right)^{1/2}\right)\right](f_c')_{28} \tag{7-2}$$

where $(f'_c)_t$ = mean compressive strength at age t days

$(f'_c)_{28}$ = 28-day compressive strength

s = coefficient depending on the cement type, such as s = 0.2 for high early strength cements; s = 0.25 for normal hardening cements; s = 0.38 for slow hardening cements

t = time in days

t_1 = 1-day

Both models require 28-day concrete strength for the formulation of compressive strength variation against time. A complete set of compressive strength test data is available only for S06 of 82194 bridge barrier replacement project (see Chapter 6). CEB-FIP and ACI 209 predictions are compared with the test data of barrier concrete of bridge S06 of 82194. Test data shows high early-age concrete strength. In that case, CEB-FIP Models Code (1990) requires using s = 0.2 in Eq. 7-2. From the comparison of the model predictors with the experimental data given in Figure 7-1 within the time span of test data, CEB-FIP model gives the best fit. This model will be used for estimating very early-age concrete strength.

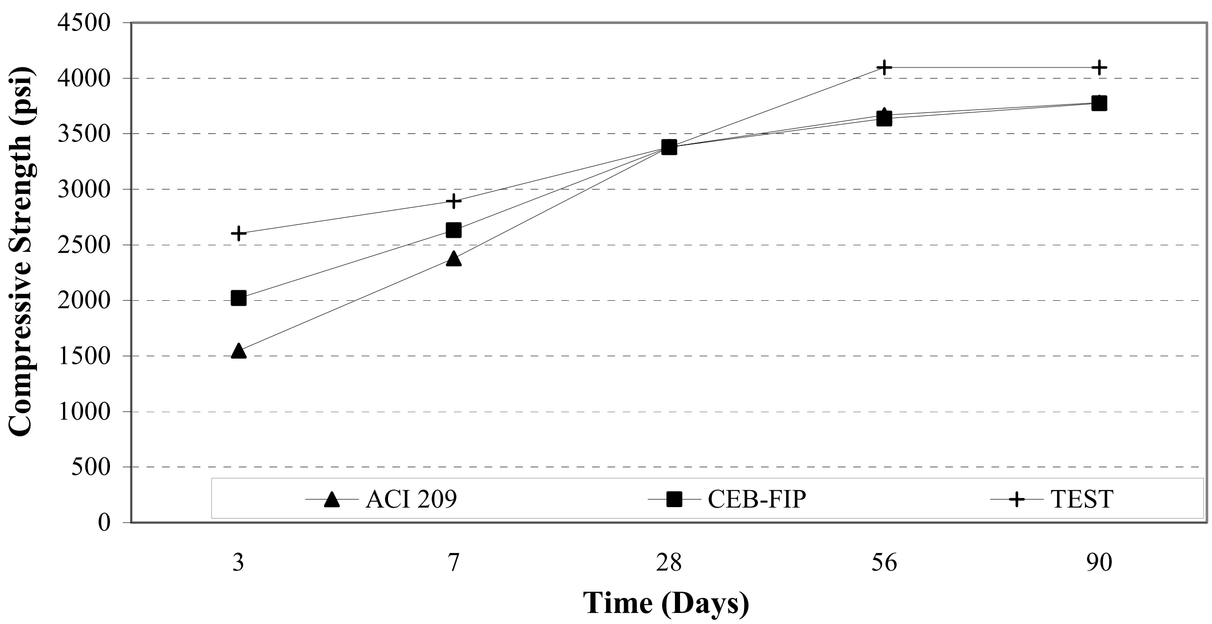


Figure 7-1. Comparison of compressive strength prediction models

7.2.2 Elasticity Modulus

There are several models available for calculating the modulus of elasticity. Two of the models are adopted by ACI and by CEB-FIP.

The ACI Committee 209 recommended model is given as:

$$E_{ct} = g_{ct}\left[w^3 (f'_c)_t\right]^{1/2} \tag{7-3}$$

where w is the unit weight of concrete (pcf) and $g_{ct} = 33$ is a constant.

The model most commonly used in literature and given in ACI 318 is:

$$E_{ct} = 57000\sqrt{(f'_c)_t} \tag{7-4}$$

The following relationship is suggested by CEB-FIP Models Code (1990) (Vincent 2003):

$$E_{ct} = E_c e^{[s/2(1-\sqrt{28/t})]} \tag{7-5}$$

where E_c is the mean modulus of elasticity of concrete at 28-days.

ACI 209 and ACI 318 models require concrete compressive strength for calculating variation of elasticity modulus against time. Similarly, CEB-FIP model requires 28-day mean modulus of elasticity of concrete. The CEB-FIP, ACI 318, and ACI 209 predictions are compared with the test results obtained from barrier concrete of bridge S06 of 82194. Figure 7-2 shows the comparison of the proposed models with the experimental counterparts.

Based on these comparisons, the CEB-FIP model, which closely predicts the early-age elasticity modulus generated in this study, is used for estimating the elasticity modulus.

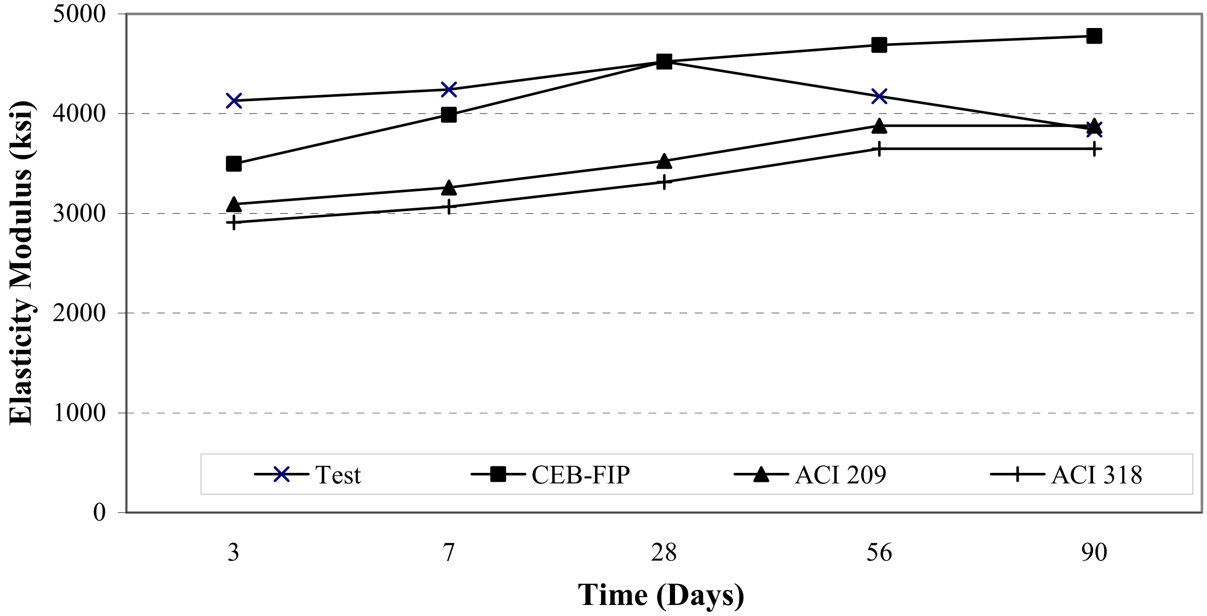


Figure 7-2. Comparison of elasticity modulus prediction models

7.2.3 Direct Tensile Strength

ACI Committee 209 recommends the following equation for computing average values of direct tensile strength ($f_t^{'}$):

$$f_t^{'} = g_t \left[w(f_c^{'})_t \right]^{1/2} \tag{7-6}$$

where w is unit weight of concrete (*pcf*), and g_t is given as 1/3.

7.2.4 Concrete Strain

Assuming concrete behaves as an elastic brittle material in tension, concrete strain at cracking can be calculated using Hook's law from the tensile strength and elasticity modulus.

$$\varepsilon = f_t^{'} / E \tag{7-7}$$

where ε is the strain, $f_t^{'}$ is the tensile stress, and E is the elasticity modulus.

Concrete crack initiates when concrete strain due to volume change exceeds the concrete strain calculated using Eq. 7-7, and with sufficient accompanying restraining stress.

7.2.5 Mechanical Properties and Shrinkage of Barrier Concrete

Concrete mechanical properties calculated using the CEB-FIP model are given in Table 7-1. The concrete shrinkage is calculated using the models described in Section 2.3.2.1. The comparisons of shrinkage strains obtained from various models are shown in Table 7-2. The shrinkage is calculated specifically for MDOT Grade D concrete (ASTM Type I cement, w/c ratio of 0.4, and cement content of 660 lb/yd^3) and assuming 60% relative humidity with two days moist curing.

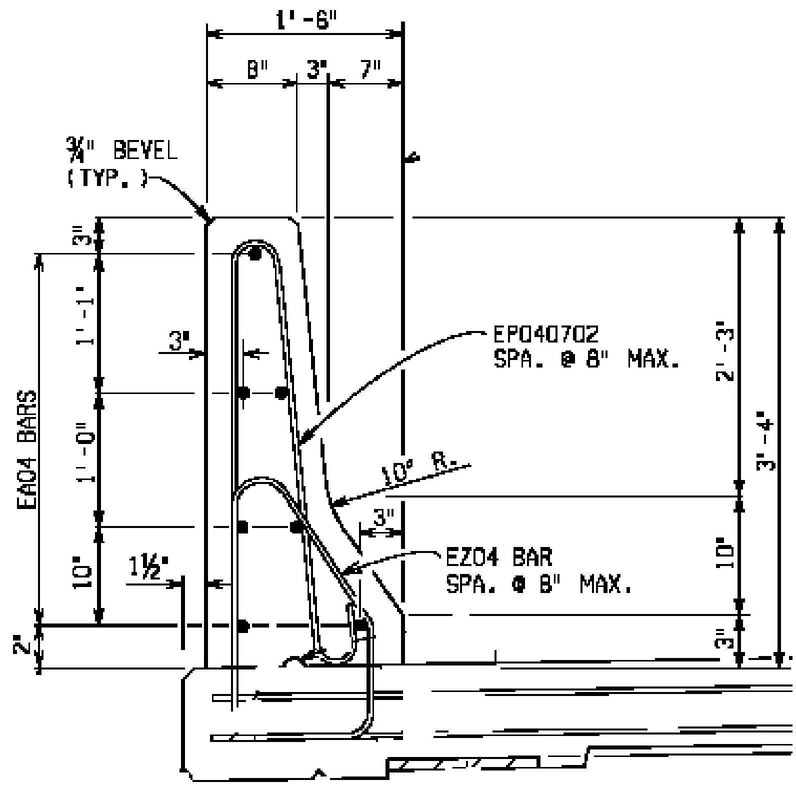


Figure 7-3. Geometry of the barrier

Table 7-1. Estimated Early-Age Mechanical Properties of Barrier Concrete

Period	Compressive Strength (psi) CEB-FIP Model	Elasticity Modulus (ksi) CEB-FIP Model	Direct Tensile Strength (psi) ACI 209 Model
3-day	3315	4071	235
7-day	4094	4524	261
14-day	4602	4797	277
28-day	5000	5000	289

Table 7-2. Shrinkage Strain Predictions by Various Models

Period	ACI 209 (Micro strain)	CEB-FIP 90 (Micro strain)	Bazant B3 (Micro strain)	Gardner & Lockman (Micro strain)
3-day	17.8	13.3	11.5	19.1
7-day	78.0	29.7	25.8	42.7
14-day	159.3	46.0	40.0	66.0
28-day	266.0	67.4	58.5	96.7

7.3 BRIDGE BARRIER CRACKING

7.3.1 Overview

Al-Rawi and Kheder (1990) showed that cracking by volume change is a result of the combined effects of strains and restraint. When strain due to volume change of a concrete component exceeds the concrete strain capacity and is accompanied by sufficient stress from the restraint effects, concrete cracks. Borst and Berg (1986) conducted analytical and experimental studies investigating concrete cracking. The results describing the stress-strain envelope that defines crack formation are shown in Figure 7-4.

As noted in Figure 7-4(a), f_c is the tensile stress, f_t is the tensile strength of concrete, E is the elasticity modulus, ε is the strain normal to the cracking plane measured before cracking, ε_c is the strain of concrete at the tensile strength of concrete, f_{tc} is the cracking strength, ε_{tc} is the cracking strain, ε^{cr} is the strain normal to the cracking plane measured after cracking, and ε_{max} is the strain of cracked concrete when the stress normal to cracking plane is zero.

The approach of simulating the cracking behavior is to assume zero stress normal to cracking plane immediately after crack formation. The stiffness orthogonal to the crack is assumed to be zero. According to Borst and Berg (1986) zero stiffness normal to the crack underestimates the true stiffness of the cracked concrete. Hence, the Figure 7-4(b) is used to define the linear elastic stress-strain relation of cracked concrete.

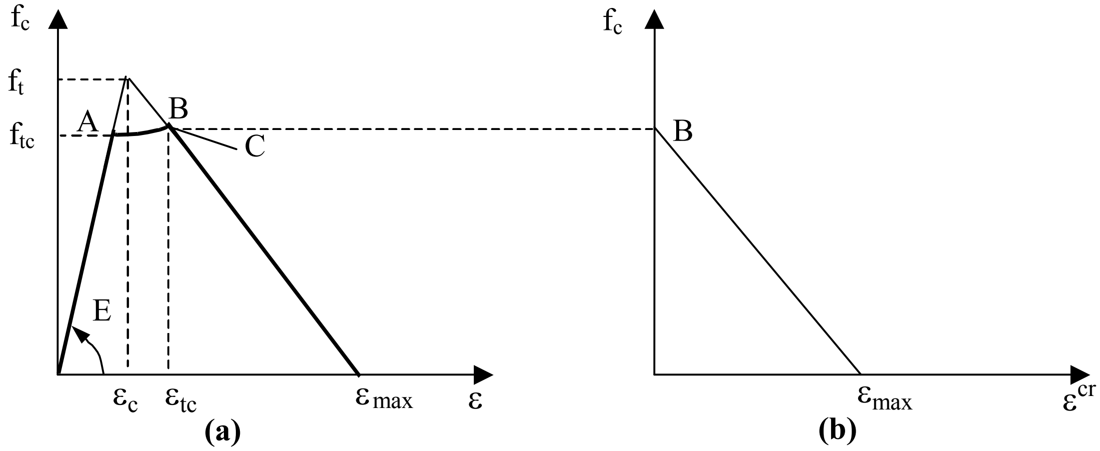


Figure 7-4. (a) Stress-strain envelope for cracking of concrete and (b) stress- strain normal to cracking plane after crack initiation (assuming linear-elastic behavior)

Figure 7-4 (a) is a combination of the linear elastic stress-strain diagram for cracked and uncracked concrete. As the stress develops with the restraint in the major principal direction and stress achieves point A in Figure 7-4 (a), strain softening begins. As a consequence of softening, strain jumps to point B from point A while stress remaining nearly constant. When the combination of stress and strain in the major principal direction reaches the envelope (point B), cracking initiates. Under this stress condition, the tensile strain should be greater than the ultimate tensile strain of concrete (ε_c) for crack initiation. The stress level at point B is assumed to be 80% of the tensile strength of concrete.

The literature describes two approaches for reducing cracking of concrete walls: these are the reduction of length/height (L/H) ratio of the wall and provision of reinforcements in an effort to replace the large cracks in plain concrete walls with an increased number of finer cracks in reinforced walls. The factors governing the cracking of continuously base-restrained walls are given below (Al-Rawi and Kheder 1990):

- L/H ratio
- magnitude of base restraint
- base rigidity and the relative volume change between wall and the base
- rate and magnitude of shrinkage and thermal loads
- percentage and type of horizontal wall reinforcement

Al-Rawi and Kheder (1990) conducted experiments with regard to these factors on 14 reduced–scale base-restrained walls. According to their observations, first crack initiated at the bottom of the central portion of the wall and propagated towards the top of the wall. Further cracks were formed towards the end of the wall. This cracking pattern and sequence confirmed that the cracks initiate in the high restraint zone and propagate towards the lower restraint zones. Comparison of crack spacing between plain concrete and reinforced concrete walls showed that reinforcement reduces the crack spacing as well as the crack width. Further experiments showed the existence of an upper limit for the horizontal reinforcement ratio. At the optimum reinforcement ratio, further reduction in crack spacing and width could be attained only by reducing the L/H ratio. The crack spacing observed with plain concrete walls was 1.24H; this is in agreement with the range given in the ACI Committee 207 report. The Eq 7-8 below is proposed for estimating the minimum crack spacing (S_{min}) in a fully base-restrained reinforced concrete wall (Al-Rawi and Kheder 1990). The thermal load effects were eliminated by the reduced size of specimens and submerged curing.

$$S_{min} = (k_1 dH)/(pH + k_1 d) \tag{7-8}$$

where, k_1 =0.57 for deformed bars, d is the bar diameter, H is the wall height, and p is the reinforcement ratio.

According to ACI 207 (2001), fully base-restrained plain concrete walls ultimately attain full-length cracks spaced at one to two times the height of the wall. Meanwhile, Cusson and Repette (2000) inspected early-age cracking of the Vachon Bridge barriers and the observed crack spacing was 0.8 times the barrier height. The steel reinforcement generates additional restraints and reduces crack spacing (ACI 207 2001). Cusson and Repette (2000) concluded that the amount of reinforcement (p=0.4%) used in Vachon Bridge barriers was the reason for crack spacing of 0.8 times the barrier height.

7.4 FINITE ELEMENT ANALYSIS FOR EVALUATING CRACK SPACING ON BARRIERS

7.4.1 Overview

The bridge barriers on interstate freeway (I-75) over Fort Street (S06 of 82194) are selected as the FE analysis prototype. The barrier consists of 23 and 19 slipformed segments with east and west exposures, respectively. The average length of a segment is 13 feet. Barrier geometry and reinforcement detailing are shown in Figure 7-3. The concrete mix design proportions are given in Table 7-3.

Table 7-3. Concrete Mix Design for Bridge S06 of 82194

Cement ASTM C-150 Type 1 ASTM C-989, GGBS Grade 100	 357 lb/yd^3 197 lb/yd^3
Aggregate MDOT 2NS MDOT 6AA	1141 lb/yd^3, max. size ≤ 1.5 in. 1700 lb/yd^3, max. size ≤ 3/8 in.
Water	250 lb/yd^3
Air	6.5 %
Water/cement	0.45

The barrier slipforming was monitored on September 11, 2002. The barrier condition was inspected on September 13, 2002. Inspection revealed full-length vertical cracks at all the control joints and between the joints of several barrier segments. The average crack spacing on cracked segments was approximately twice the barrier height. In addition to the full-length vertical cracks, there were several shorter vertical cracks near the top portion of the barrier (see Chapter 5 for more details).

The volume change due to shrinkage and thermal loads combined with the base restraint at the deck-barrier interface is described in the literature as the cause of vertical barrier cracking. The objectives of finite element analysis were to study the significance of thermal and shrinkage loads on barrier cracking and to establish the minimum spacing between full-length vertical cracking of barrier. Establishing the predicted minimum crack spacing will help the development of crack management procedures.

The following sections describe the calculation of thermal and shrinkage loads from the concrete mix, barrier geometry, and the environmental conditions. Further, the finite element modeling of barrier, analysis, and the results are discussed.

7.4.2 Thermal Loads

Thermal loads are inherent to concrete construction. Cement hydration is an exothermic process that generates heat and consequently a temperature difference between the concrete mass and the ambient air.

The time period required for a specific concrete mix to attain its highest temperature level due to hydration is needed for determining the thermal load. Thermal load is the temperature difference between the ambient air and the interior of the concrete mass. The peak hydration temperature of the barrier (volume-to-exposed surface ratio of 0.42 ft), which is slipformed using concrete with Type I cement under the ambient temperature of 85^{o}F, is reached at 12 hours after slipforming. The use of GGBS delays the time of peak hydration temperature. According to Siew et al. (2003) the addition of 65% GGBS delays the time of peak hydration temperature by 18 hours. In barrier concrete mix 55% of GGBS is used and the delay is estimated as 12 hours. Hence, the peak hydration temperature of barrier concrete is achieved 24 hours after slipforming.

In order to calculate the peak hydration temperature using ACI 207.2R procedure, the heat of hydration generated by the mix needs to be evaluated. ACI 207.2R procedure is specifically developed for mixtures with OPC. For substitution with GGBS correction needs to be incorporated in calculating the heat of hydration. Figure 7-5 shows that the addition of GGBS reduces the heat of hydration (Ecocem@ 2003). The data given in Figure 7-5 shows that the amount of heat generated during hydration of GGBS is equivalent to 25% of the heat generated by OPC. The amount of GGBS used in barrier concrete mix is 197 lb/yd^3 (Table 7-3). The amount of heat generated by 197 lb/yd^3 of GGBS is equivalent to that of 49.25 lb/yd^3 of OPC. Hence, the equivalent OPC content of barrier concrete mix given in Table 7-3 is calculated as 406.25 lb/yd^3. According to the cement mill report obtained from the concrete supplier, the average Blaine fineness of the cement used in barrier concrete is 1774 ft^2/lb. The peak hydration temperature calculated for barrier concrete, which contains 406.25 lb/yd^3 of cement with the average fineness of 1774 ft^2/lb, is 111^{o}F. The ambient temperature when barrier concrete attain its peak hydration temperature (24 hours after slipforming) is assumed to be 85°F. Hence, the maximum temperature difference between the interior of the concrete barrier and the ambient air is calculated as 26°F.

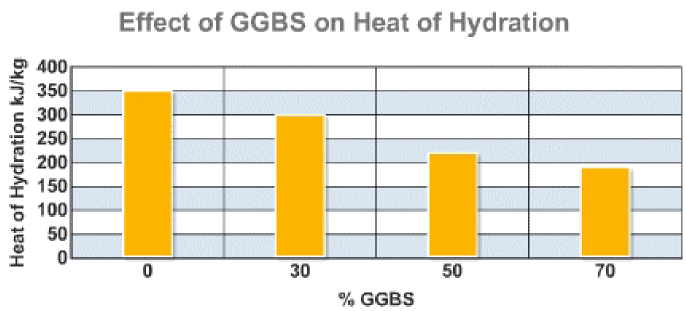


Figure 7-5. Variation of heat of hydration of cement with different percentages of GGBS

7.4.3 Shrinkage Strain

Shrinkage in concrete is due to loss of moisture (drying), chemical reactions of the concrete with water (autogenous shrinkage), and chemical reactions of the cement with atmospheric carbon dioxide (carbonation). Carbonation shrinkage takes place when hardened concrete that contains some moisture reacts with carbon dioxide present in the air. Carbonation shrinkage has little or no contribution to early-age concrete shrinkage. The contribution from autogenous shrinkage to early-age concrete shrinkage is insignificant because it occurs only in the barrier core and only until the drying front reaches the core (Bazant and Murphy 1995). Additionally, autogenous shrinkage is significant if w/c ratio is less than 0.4 (Cusson and Repette 2000). Current interest is on the influence of early-age shrinkage on concrete cracking. Hence, drying shrinkage is the significant shrinkage component of concrete during early ages. Early-age shrinkage strain estimates are based on the prediction models described in Chapter 2. These models assume a specific period of wet curing (Table 7-4). The common practice of slipformed barrier construction is to only apply curing compound without any wet curing. Due to uncertainties of curing effects and differences in the models, total shrinkage is estimated from shrinkage models as maximum and minimum values representing upper and lower bounds of expected shrinkage as shown in columns (6) and (7) of Table 7-5.

Table 7-4. Limitations of Shrinkage Prediction Model

Parameter (1)	ACI 209R-92 (2)	CEB-FIP 90 (3)	Bazant B3 (4)	GL 2000 (5)
Mean 28-day compressive strength $f_c^{'}$ psi	-	2,900 – 13, 000	2,500 – 10,000	2,900 – 10,000
Aggregate/Cement	-	-	2.5 – 13.5	-
Cement lbs/ft^3	-	-	10 – 45	-
Water/cement ratio	-	-	0.3 - 0.85	-
Relative Humidity (%)	40-100	40-100	40-100	40-100
Cement Type	I or III	R, SL, or RS	I, II, or III	I, II, or III
Age of concrete at the time of shrinkage strain calculation (t) or age at which concrete starts drying (t_c) (Moist cured)	≥ 7 days	$t_c \leq$ 14 days	$t_c \leq t$	≥ 2 days
t or tc (Steam cured)	≥ 1-3 days	$t_c \leq$ 14 days	$t_c \leq t$	≥ 2 days

Table 7-5. Total Shrinkage of Concrete at Different Ages

Age (Days) (1)	Shrinkage in./in.x10^{-6}					
	ACI 209 (2)	CEB-FIP 90 (3)	Bazant B3 (4)	Gardner & Lockman (5)	Minimum (6)	Maximum (7)
0.5	4.4	6.2	4.4	9.6	4.4	9.6
0.75	8.8	8.8	6.3	13.6	6.3	13.6
1	13.1	10.8	7.7	16.6	7.7	16.6
2	29.7	16.5	11.7	25.4	11.7	29.7
3	45.5	20.7	14.7	31.8	14.7	45.5
7	100.9	32.3	23.0	49.8	23.0	100.9

7.4.4 Mechanical Properties of Concrete

Early-age mechanical properties of concrete are required to evaluate the cracking potential of barrier concrete. Several models and a model of choice were presented in the section entitled "Mechanical Properties of Concrete" on the estimation of early-age mechanical properties. Early-age mechanical properties of concrete calculated using the model of choice are given in Table 7-6. Additionally, for establishing the crack formation parameters the cracking tensile strain is taken as 1.2 times the ultimate tensile strain of concrete and the cracking strength is 0.8

times the ultimate tensile strength of concrete (Table 7-6). The thermal expansion coefficients used are given in RILEM 42-CEA (1981) as 8.3×10^{-6} /°F and 6.7×10^{-6} /°F for concrete at ages up to one day and 1-7 days, respectively.

Table 7-6. Tensile Stress and Strain Required for Crack Initiation

Age (t) (Days) (1)	Tensile Strength (f_t) psi (2)	Elasticity Modulus (E_c) ksi (3)	Tensile Strain at Tensile Strength (ε_t) Microstrain (4)	Cracking Strength (f_{tc}) psi (5)	Cracking Strain (ε_{tc}) Microstrain (6)
0.5	124	2364	52.5	99	63
0.75	142	2711	52.5	114	63
1	155	2943	52.5	124	63
2	180	3435	52.4	144	63
3	193	3681	52.4	154	63
7	215	4090	52.5	172	63

7.4.5 FE Modeling and Analysis of Barrier Segment

The primary purpose of finite element (FE) analysis of barrier segments is to determine the length/height ratio at which cracking may initiate. Four different analysis models are generated corresponding to several length/height ratios (e.g., L/H =1, 2, 3, and 4). Further, the analysis of longitudinal stress distributions with accompanying strain distributions establishes the vertical crack spacing. Reinforced and plain concrete barrier models are developed to study the effects of reinforcement. In FE modeling of the barrier, C3D8 continuum elements and T3D2 truss elements are used to represent concrete and reinforcements, respectively. Using symmetry, half of the barrier segment length is modeled.

7.4.6 Analysis Results

Concrete bridge barriers are subjected to shrinkage and thermal loads simultaneously. However, shrinkage and thermal loads are considered independently for the analysis. The main objective of the analysis is to determine the L/H ratios of barrier at which cracking may initiate. In order to determine the L/H ratios, the longitudinal stress distributions needs to be studied. After establishing the critical L/H ratio for cracking, significance of very early-age shrinkage and thermal loads on barrier cracking is investigated.

7.4.6.1 Shrinkage and Reinforcement Effects

Shrinkage effects in concrete are simulated using an equivalent temperature load corresponding to the estimated upper and lower shrinkage limits. Maximum tensile stress occurs at the area with the highest restraint (Figure 7-6). This further confirms that cracking of barriers initiates at a location close to the highly restrained base. For barriers with different L/H ratios, longitudinal stress along the barrier length close to the base increases towards the center section of the barrier. Changes to maximum stress are very limited for increased L/H ratio beyond two (Figure 7-7). Figure 7-8(a) and (b) show the longitudinal stress distribution along the barrier height at mid section. In these figures, the longitudinal stresses are normalized with respect to the maximum longitudinal stress at the base of the barrier. For L/H=1, compressive stress develops at the upper portion of the barrier preventing the formation of full-length vertical cracks (Figure 7-8(a)). However, for L/H=2, the barrier cross-section is only subjected to tensile stresses (Figure 7-8(b)). This illustrates the potential of full-length vertical crack formation at a spacing equal to the barrier height.

Though perfect bond between concrete and reinforcement is assumed, the influence of reinforcement located close to the base of the barrier is insignificant, irrespective of L/H ratios. There is an incremental effect on longitudinal stress for low L/H ratios from the reinforcement located at sections near the upper portion of the barrier. With increasing L/H ratio, the base restraint negates the contribution of the reinforcement regardless of its location along the barrier section (Figure 7-8).

The analysis results verify that the potential full-length vertical crack spacing in bridge barriers is approximately equal to the barrier height if the barrier is fully base restrained. In reality, the barrier is not fully base restrained due to the fact that there is a cold joint between the deck and the barrier. Effect of base restraint on longitudinal stress distribution along the barrier height was investigated for a barrier with L/H = 2. Figure 7-9 shows the longitudinal stress distribution along the barrier height at the mid section. With the reducing base restraint, compressive stress region at the barrier upper portion increases thus eliminating the vertical cracking. Therefore, reduction in base restraint will increase crack spacing.

In concrete of 2-day age, the maximum shrinkage strain calculated from the prediction models is less than the cracking strain. However, this strain level still causes significant levels of tensile stress (near the cracking strength of concrete) at the base. Additionally, without moist curing early-age shrinkage is substantially increased at a time when tensile strength is very low. In that case cracks may form even before two days after placement, which is the time at which shrinkage strains are calculated.

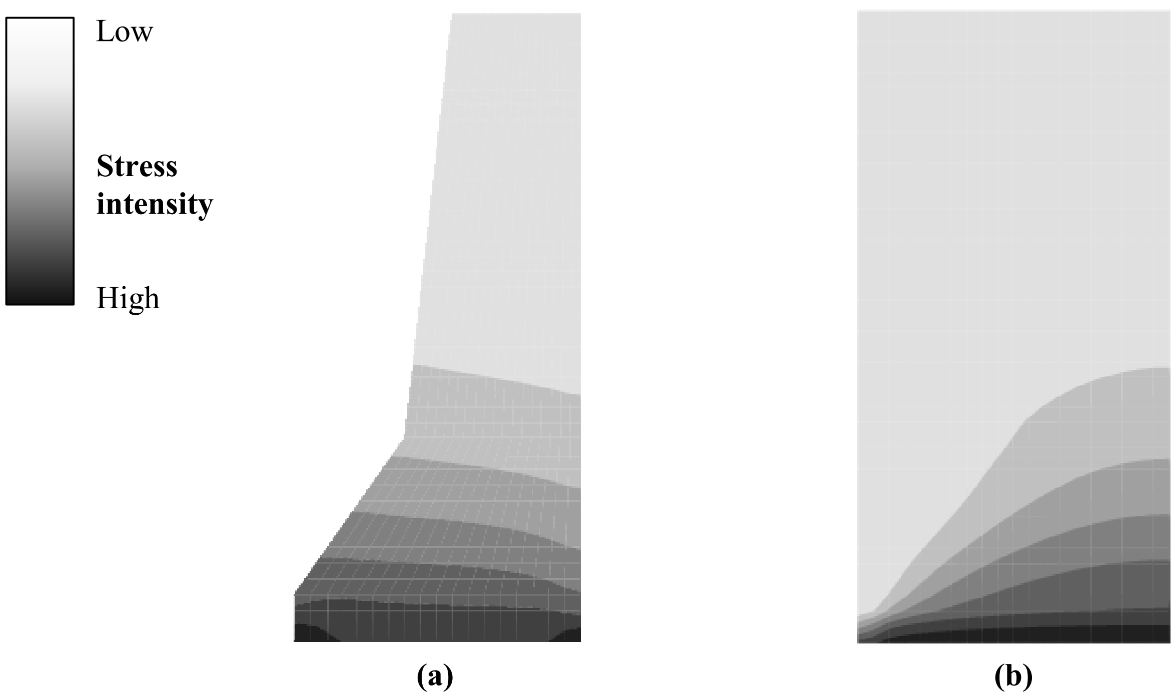
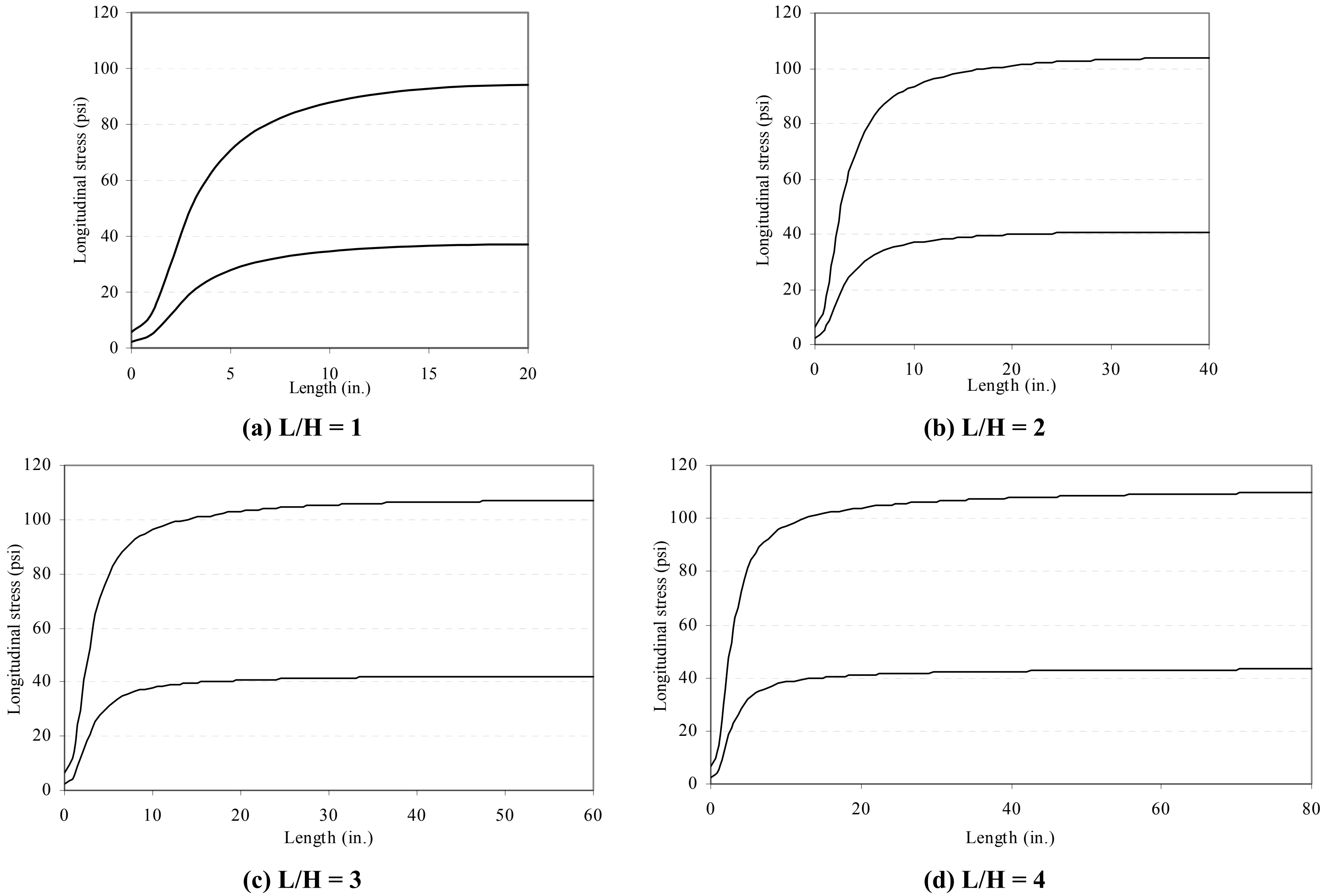


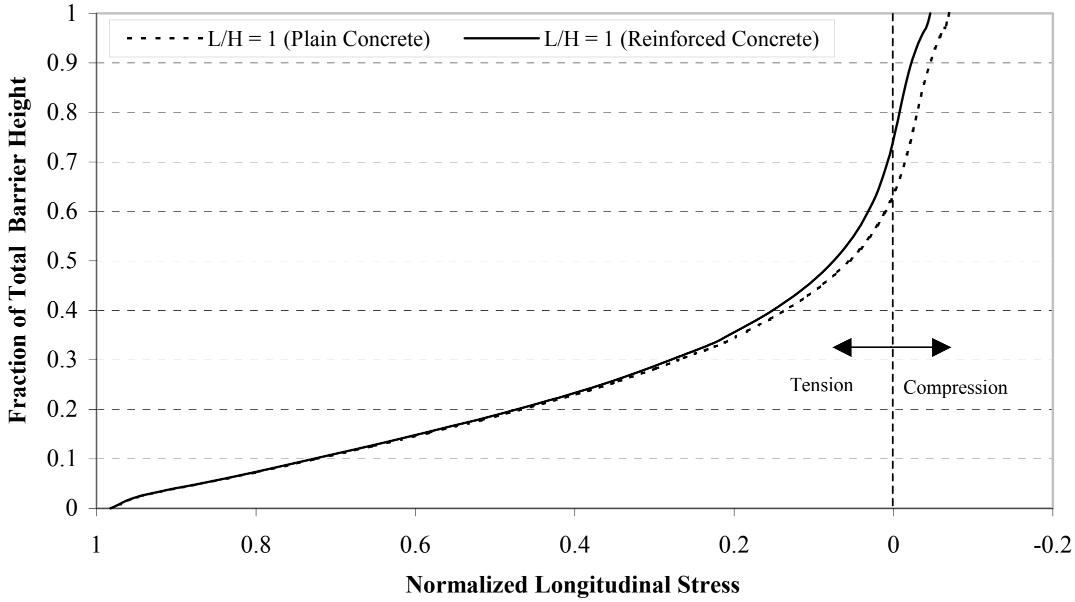
Figure 7-6. Longitudinal stress distribution along the barrier (a) height and (b) length for L/H=1



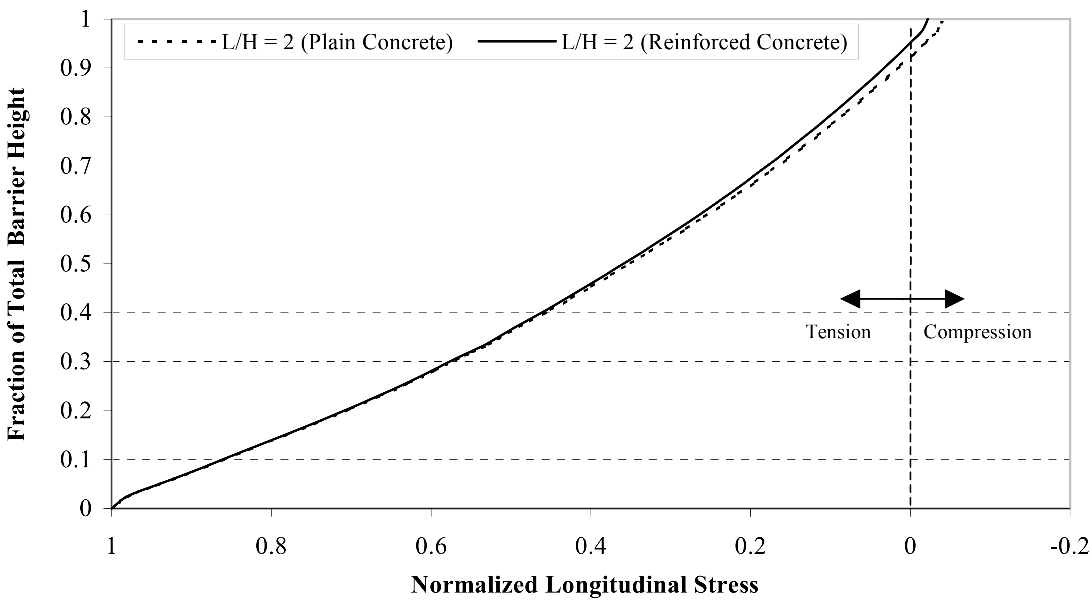
(a) L/H = 1 **(b) L/H = 2**

(c) L/H = 3 **(d) L/H = 4**

Figure 7-7. Longitudinal stress distribution along the barrier length under upper and lower limits of shrinkage



(a)



(b)

Figure 7-8. Longitudinal stress distribution along barrier height at mid section (a) L/H=1 and (b) L/H=2

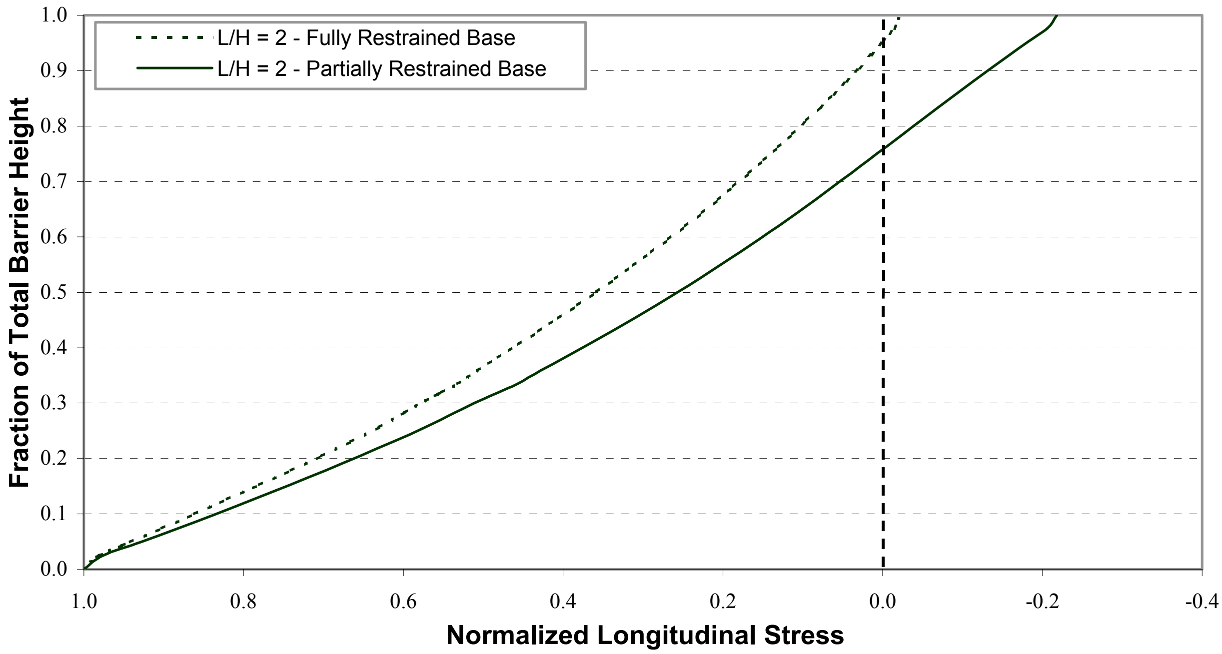


Figure 7-9. Longitudinal stress distribution along barrier height at mid section for L/H = 2

7.4.6.2 Thermal Load Effects in Reinforced Barriers

In Section 7.4.6.1, it is established that the minimum potential crack spacing in bridge barrier with full base restraint is equal to the barrier height. To investigate the stress magnitudes under thermal loading, barrier models with L/H = 2 are analyzed. For the analysis, perfect bond between concrete and reinforcement and full base restraint conditions are assumed. The thermal loading was calculated following the procedure given in ACI 207.2R (2001) and using the other relevant factors discussed in Section 7.4.2. The maximum temperature difference between the interior and exterior of the barrier was estimated at 24 hours after concrete placement. At this time, the maximum temperature difference between the ambient air (85 ^{o}F) and the interior of the concrete barrier is estimated as 26 ^{0}F. The analysis results showed that the longitudinal stress at barrier base for L/H=2 is 627 psi. This stress and associated strains are significantly higher than the cracking strength of concrete at respective age.

Regardless of bond quality between concrete and reinforcement (model assumes perfect bond), under thermal loading the influence of reinforcement to stresses and strains within the barrier is insignificant, irrespective of L/H ratios. This is because the temperature of both concrete and steel are equal and cool down at about the same rate.

In summary, the analysis results showed that the thermal loading alone can cause early-age barrier cracking. Reinforcements are ineffective in controlling cracking under thermal loads. Additionally, volume change due to shrinkage and thermal loading occurs simultaneously. Though the shrinkage contribution alone may not be sufficient to induce cracking, the combined effects of shrinkage and thermal loading generate strains well above the cracking strains.

7.5 CONCLUSIONS

Full-length vertical cracking that is observed at early ages is often the predominant type of distress in a barrier segment. Other documented distresses are due to the progression of distress initiated by vertical cracking. Map cracking is also a common defect observed on barrier surfaces. Map cracking generally remains near the barrier surface and its impact on long term barrier distress is minimal.

Field inspection data obtained from barriers of 21 bridges show that the full-length vertical crack spacing to barrier height ratio is equal to two. Additional data indicated that barriers exposed to north and south directions show an increased amount of map cracking.

Major causes of cracking are identified as volume change of concrete barrier and restraint. Early-age volume change is mainly due to drying shrinkage and thermal loads. Cracking due to volume change is a result of the combined effects of volume change strains and restraint. When strain due to volume change of a concrete component exceeds the concrete strain capacity accompanied by sufficient stress due to restraint effects, concrete cracks.

The analysis presented in this research demonstrated that the tensile stresses due to early-age thermal load alone can cause barrier cracking. Volume change of concrete due to temperature and shrinkage occurs simultaneously. An increase in drying shrinkage due to construction errors and delays in wet cure can increase the tensile stresses. Drying shrinkage, beyond very early ages, will increase crack widths that have formed due to thermal loads.

The time concrete reaches peak temperature during hydration depends, among other parameters, on the volume to exposed-surface ratio and the concrete placement temperature. Volume to exposed-surface ratio of New Jersey Type 4 concrete barrier is 0.42 ft. For small volume to exposed-surface ratio, the percent absorbed or dissipated heat between placement and ambient temperatures is very high. In that case, the effective placement temperature approaches the ambient temperature. Consequently, the concrete temperature measured before placement does not play a major role in predicting early-age concrete properties. The ambient temperature at the time of placement governs the early-age concrete thermal properties.

The concrete parameters controlling the thermal load are the cement type, content, and fineness, ambient temperature at the time of concrete placement, and the time of inception of curing. These parameters govern the temperature rise in barrier concrete during hydration process. The temperature difference between peak temperature reached within the concrete mass and the ambient temperature establishes the thermal load on the barrier. The thermal load controls the magnitude of the tensile stress developed at the barrier. Reducing thermal load will reduce the cracking potential of barriers. Today's weather prediction technology can be utilized as a decision tool in using admixtures that can refine the hydration process in order to minimize the thermal load. Use of mineral additives that generates low heat of hydration is helpful in reducing early-age thermal load.

The shrinkage prediction models used in calculating the early-age shrinkage of barrier concrete are based on a certain wet curing duration. In the case of slipformed barriers, higher values of shrinkage strains should be expected due to lack of wet curing.

From the literature review, field inspection data, and finite element analysis results, the following conclusions are made:

1. If volume change of barrier is not controlled until the tensile strength of concrete develops, vertical cracks will develop on the barrier with the minimum spacing equals to barrier height.
2. Cracking can be delayed in barriers with extended wet curing but cannot be prevented.
3. Contribution of reinforcement is not significant in controlling cracking under thermal loads. After the crack formation, reinforcement will establish the crack width provided there is adequate bond with concrete.

8 SUMMARY & CONCLUSIONS

8.1 SUMMARY

The research need was established in a report by J.F. Staton and J. Knauff (1999), titled, "*Evaluation of Michigan's Concrete Barriers"*. The observations made in the report indicated that many of the New Jersey type concrete barriers used on Michigan bridges are deteriorating at a rate faster than expected. The current study was designed in order to further evaluate the observations described in this report and to develop a comprehensive understanding of the barrier life span, from construction to repair or replacement.

The project was organized into eight tasks. The first task covering the literature review which included the review of relevant MDOT design, construction, and maintenance practices concerning bridge barriers and a historical review of bridge barrier design, material specifications and construction practices. Also included in the literature review are reports dealing with cracking distress and its impact to bridge barrier service life. The literature identified the primary distress types on barriers as vertical cracking (termed as transverse cracking), map cracking, horizontal cracking, popouts, spalling or disintegration, efflorescence, corrosion, and delamination. Vertical cracking that is observed at early ages is often reported as the predominant type of distress observed in any given barrier segment. Map cracking is also reported as a common defect observed on barrier surfaces. Other types of distress originate primarily from the vertical cracking.

As the second task, bridge engineers at state highway agencies were surveyed. The results of this survey were used to assess the types of problems other states might have with their bridge barriers, as well as their material specifications, design, construction practices and acceptance parameters, and tests.

Under the third task, selected bridge barriers were inspected for documentation of distress types, their extent, and their progression. Representative categories of barrier distress were established from pre-inspection field observations and the literature review. These categories were used as a platform to select candidate barriers for obtaining core specimens. The inspected barriers included both good and poor performing barriers, representing a range of variables including age, material types, and construction methods.

Task four included sampling eight of the inspected barriers for laboratory investigations. Sampling included collection of two specimens for petrographic examination. Additional specimens were sought depending on the visual assessment and UPV, RCPT, absorption, and permeability tests were performed.

The four bridge barrier construction projects were monitored under task five. The purpose of construction monitoring was to observe and document the construction procedures, materials, and test procedures. The construction of two slipformed barrier and two form-cast barrier projects were monitored.

Under task six, laboratory testing of the specimens obtained by coring existing barriers and the standard specimens prepared during barrier construction was performed. Mechanical property testing, UPV, RCPT, absorption, and air-permeability tests were performed on the standard specimens obtained during construction monitoring to assess the material properties that related to concrete cracking and durability. Barrier performance parameters were established using the findings of the literature review, inspection data, construction monitoring observations, laboratory analysis, and finite element analysis. The recommendations of this study are grouped under the processes of design, construction, and maintenance. The recommendations are for improvement of barrier service life.

8.2 CONCLUSIONS

The study findings established that early barrier deterioration is initiated by vertical (transverse) cracking and accelerated by the presence of voids and cavities, reinforcement cover, and the overall soundness and quality of the concrete barrier. The barrier concrete quality is an aggregate of quality of material, construction procedures, and workmanship. The cracking of concrete is the result of stresses that form due to volume change under thermal and shrinkage loads by the restraint developed between the deck and the barrier base. Concrete cracking is more likely if volume change takes place prior to concrete achieving sufficient tensile strength. Cracking of form-cast and slipformed barriers cannot be prevented but can be controlled.

Vertical cracking can be minimized by eliminating the base restraints (e.g., use of precast barrier segments).

In this research a solution was sought without making a dramatic change to the current practice; the means of reducing volume change strains at early ages is related to the environmental conditions during early ages. How concrete is handled, placed, and cured is defined in construction specifications. Unfortunately, specifications do not specifically address barriers, although, significant detail in the specification is provided for decks. This omission is indicative and representative of the attention given to barrier construction. Thus, in order to emphasize the intent of improving barrier performance, specifications should include specific requirements for barrier construction (although a section for general structural concrete is described in the MDOT- Standard Specifications for Construction in addition to decks). The specifications do not describe slip forming but construction practices imply that slipforming is allowed. As implemented today, slip forming needs significant improvements in order to produce a barrier segment with acceptable compaction and surface quality. In form-cast concrete, Standard Specifications allow form removal after 18 hours without any expectation of wet curing. The only requirement is the application of curing compound upon form removal. After placement, the barrier top surface needs to be protected by an application of curing compound. Limitations to ambient temperature and evaporation are discussed in the specifications during the time of concrete placement. Expectations of concrete protection upon placement are not at an equal level of detail with those of during placement.

Emphasizing the fact that early-age crack control, or in more general terms crack management, is the key to durable barriers, further recommendations are as follows:

1. It is established that when using the current cast-in-place practices, the minimum expected vertical crack spacing of the barrier is about 3 feet. A prudent approach may be to implement the use of crack arrestors by placing a trim inside the forms at about 3 feet intervals. The crack arrestors, some with cracks formed at full length, can be sealed with a durable silicone-based flexible material during the first scheduled maintenance cycle.

2. Concrete mix design needs to be modified to tolerate substandard curing and to generate lower heat of hydration. Substitution of mineral admixtures results in generation of concrete mixes that generate lower heat of hydration and curing tolerable concrete.

3. The curing process needs to be improved by delaying form removal to five or even seven days after placement. The barrier casting process needs to be shifted to evening or nighttime for the purpose of limiting evaporation during placement and temperature difference between barrier concrete and ambient air. The top surface of the form-cast barrier needs to be protected with curing compound or a wet burlap.

9 SUGGESTIONS FOR FUTURE RESEARCH

There are two topics of significant importance with regards to improvement of the performance of concrete barriers. The first one deals with developing new concrete mix designs with mineral admixtures that can reduce shrinkage and tolerate substandard curing. There are numerous studies in the literature describing potential designs. It will be very beneficial to test these mix designs and evaluate their resistance to exposure conditions in Michigan.

The second topic is a more pragmatic approach to dealing with barrier durability. The use and applicability of precast barriers should be investigated. Some states already allow precast barriers. The research can evaluate both the effective length of precast barriers as well as a means of fastening the precast barrier components to the bridge structure. The strength and ductility expectations from the fasteners need to be defined. Testing should be performed on prospective fastening mechanisms.

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APPENDIX A

Cover Letter and the Proposed Survey

Causes and Cures for Cracking of Concrete Barriers Michigan Department of Transportation

This survey will take you approximately 10- 15 minutes

☐ Please check this box if you like to receive a copy of survey results.

Project Summary

The objective of this research project, funded by Michigan Department of Transportation (MDOT), is to recognize causes and to employ cures for premature deterioration of New Jersey (NJ) type concrete barriers. The causes will be investigated and recommendations will be made that are intended to increase the service life and to reduce premature deterioration of concrete barriers. As a result of this research, those methods that are immediately implementable in Michigan will be recommended for action. The following information would be greatly appreciated.

Your Name: ______________________________ Phone: ___________________________

E-mail: __________________________________

If you cannot complete this entire survey, please provide us the brief information requested only on this page. This will lead us to the personnel in your organization who may have the information and/or experience on concrete barriers. Thank you very much for your time and effort.

Name: __________________________________ Phone: ___________________________

E-mail: __________________________________

Name: __________________________________ Phone: ___________________________

E-mail: __________________________________

Should you have questions in this regard, please contact the MDOT project manager.

John F. Staton, P.E.

Supervising Engineer,
Materials Research Group
Construction & Technology,
Michigan Department of Transportation
P.O. Box 30049
Lansing, MI 48909
Phone: 517-322-5701
Fax: 517-322-5664
E-mail: statonj@michigan.gov

1. How frequently do you observe any of the following premature distress conditions in your bridge barriers? If yes, please indicate.

Map cracked areas

Low	Medium	High
☐	☐	☐

Horizontal cracking near joints

Low	Medium	High
☐	☐	☐

Continuous horizontal cracks near the top and along the length of barrier

Low	Medium	High
☐	☐	☐

Vertical cracking within the proximity of the joint

Low	Medium	High
☐	☐	☐

Multiple vertical cracks between joints

Low	Medium	High
☐	☐	☐

Multiple vertical cracking near the barrier toe

Low	Medium	High
☐	☐	☐

Severe section loss at or near the top of the barrier

Low ☐ Medium ☐ High ☐

Local popout

Low ☐ Medium ☐ High ☐

Signs of corrosion due to lack of concrete cover

Low ☐ Medium ☐ High ☐

2. Do you observe an overall durability problem with the NJ-type concrete bridge barriers?

Yes ☐ / No ☐

3. What construction procedures are being used for casting NJ-type barriers in your Agency?

☐ Cast-in-place
☐ Slipfom
☐ Other (Please specify): ________________________________

4. What curing procedures are being used/specified for barriers?

5. Is your Agency specifying epoxy-coated reinforcement for the barriers?
Yes ☐ / No ☐
If yes, please specify since when: ________________________________

6. Are coatings/sealants currently being specified for barriers?

Yes ☐ / No ☐

7. Does your Agency specify a concrete mix design for the barriers different than the deck? (Please describe)

8. Has your Agency used ground granulated blast furnace slag, fly ash or other pozzolans in the mix for the reduction of the permeability of barrier concrete?

Yes ☐ / No ☐
If yes, please specify: ________________________(Percentages of total cementitous materials)

9. What is the surface finish method that is used for barrier concrete?

Sacked ☐ / Rubbed ☐ / Other ☐, please specify: ________________________

10. Do you observe differences in performance between barriers on rural roads versus trunkline/interstate routes?

Yes ☐ / No ☐

If yes, any explanation why?

11. Does your Agency apply deicers to the bridge deck?

Yes ☐ / No ☐ If yes, what kind of deicer (CMA, Salt, etc.): _________________

12. Has your Agency initiated any changes to improve the durability of NJ-type barriers? (Check all that apply)

☐ Changing the mix design
(Cement content, cement type, mineral admixture, water-cement ratio, aggregate, and admixture)

☐ Changing curing and/or construction procedure

Other changes, please explain:

13. If your Agency performed or participated in research related to this topic, can you please provide a contact name:

Name: _________________________________ Phone: __________________________
E-mail: _________________________________

You can also print and fax a hard copy to 313-577 3881

Compiled Survey Response Data

Table A-1. Coding System

Codes Used	Meaning of the Codes
1	Yes
DN	Don’t Know
NR	Not responded
None	Don't have the problem

Table A-2. Response to Question No. 1(a)

	States	How frequently do you observe any of the following premature distress conditions in your bridge barriers? (a) Map cracked areas			
		High	Medium	Low	Comments
1	Alabama (1)	1			
	Alabama (2)	1			
2	Connecticut		1		
3	D.C.	1			
4	Florida	1			
5	Hawaii	1			
6	Idaho (1)		1		
	Idaho (2)	1			
7	Illinois (1)	1			
	Illinois (2)		1		
8	Indiana		1		
9	Maryland	1			
10	Massachusetts	1			
11	Michigan (1)		1		
	Michigan (2)			1	
	Michigan (3)		1		
	Michigan (4)	1			
12	Minnesota		1		
13	Missouri	1			
14	Montana	1			
15	Nebraska		1		
16	Nevada	1			
17	New Hampshire	1			
18	New Jersey	1			
19	New Mexico (1)			1	
	New Mexico (2)		1		
	New Mexico (3)	1			
20	New York (1)	1			
	New York (2)			1	
21	North Dakota	1			
22	Tennessee (1)	1			
	Tennessee (2)		1		
	Tennessee (3)	1			
23	Texas	1			
24	Vermont			1	
25	Virginia (1)		1		
	Virginia (1)		1		
26	Washington	1			

Table A-3. Response to Question 1(b)

	States	How frequently do you observe any of the following premature distress conditions in your bridge barriers? (b) Horizontal cracking near joints			
		High	Medium	Low	Comments
1	Alabama (1)	1			
	Alabama (2)	1			
2	Connecticut	1			
3	D.C.	1			
4	Florida	1			
5	Hawaii	1			
6	Idaho (1)		1		
	Idaho (2)		1		
7	Illinois (1)	1			
	Illinois (2)	1			
8	Indiana		1		
9	Maryland	1			
10	Massachusetts	1			
11	Michigan (1)		1		
	Michigan (2)			1	
	Michigan (3)		1		
	Michigan (4)		1		
12	Minnesota		1		
13	Missouri	1			
14	Montana	1			
15	Nebraska	1			
16	Nevada	1			
17	New Hampshire	1			
18	New Jersey	1			
19	New Mexico (1)		1		
	New Mexico (2)	1			
	New Mexico (3)	1			
20	New York (1)				NR
	New York (2)	1			
21	North Dakota	1			
22	Tennessee (1)	1			
	Tennessee (2)		1		
	Tennessee (3)	1			
23	Texas	1			
24	Vermont		1		
25	Virginia (1)			1	
	Virginia (2)		1		
26	Washington	1			

Table A-4. Response to Question 1(c)

	States	How frequently do you observe any of the following premature distress conditions in your bridge barriers? (c) Continuous horizontal cracking near the top &along the length of barrier			
		High	Medium	Low	Comments
1	Alabama (1)	1			
	Alabama (2)	1			
2	Connecticut	1			
3	D.C.		1		
4	Florida	1			
5	Hawaii	1			
6	Idaho (1)	1			
	Idaho (2)	1			
7	Illinois (1)	1			
	Illinois (2)		1		
8	Indiana	1			
9	Maryland	1			
10	Massachusetts				NR
11	Michigan (1)	1			
	Michigan (2)			1	
	Michigan (3)	1			
	Michigan (4)			1	
12	Minnesota		1		
13	Missouri	1			
14	Montana	1			
15	Nebraska				
16	Nevada	1			
17	New Hampshire	1			
18	New Jersey				None
19	New Mexico (1)		1		
	New Mexico (2)		1		
	New Mexico (3)	1			
20	New York (1)				NR
	New York (2)	1			
21	North Dakota				NR
22	Tennessee (1)	1			
	Tennessee (2)	1			
	Tennessee (3)	1			
23	Texas	1			
24	Vermont		1		
25	Virginia (1)			1	
	Virginia (2)	1			
26	Washington	1			

Table A-5. Response to Question 1(d)

	States	How frequently do you observe any of the following premature distress conditions in your bridge barriers? (d) Vertical cracking within the proximity of the joint			
		High	Medium	Low	Comments
1	Alabama (1)	1			
	Alabama (2)	1			
2	Connecticut		1		
3	D.C.	1			
4	Florida	1			
5	Hawaii	1			
6	Idaho (1)		1		
	Idaho (2)		1		
7	Illinois (1)	1			
	Illinois (2)			1	
8	Indiana	1			
9	Maryland		1		
10	Massachusetts				NR
11	Michigan (1)		1		
	Michigan (2)			1	
	Michigan (3)		1		
	Michigan (4)		1		
12	Minnesota	1			
13	Missouri	1			
14	Montana		1		
15	Nebraska	1			
16	Nevada	1			
17	New Hampshire				NR
18	New Jersey				None
19	New Mexico (1)		1		
	New Mexico (2)		1		
	New Mexico (3)		1		
20	New York (1)			1	
	New York (2)	1			
21	North Dakota		1		
22	Tennessee (1)		1		
	Tennessee (2)		1		
	Tennessee (3)			1	
23	Texas				NR
24	Vermont			1	
25	Virginia (1)		1		
	Virginia (2)	1			
26	Washington	1			

Table A-6. Response to Question 1 (e)

	States	How frequently do you observe any of the following premature distress conditions in your bridge barriers? (e) Multiple vertical cracking between joints			
		High	Medium	Low	Comments
1	Alabama (1)	1			
	Alabama (2)		1		
2	Connecticut			1	
3	D.C.		1		
4	Florida	1			
5	Hawaii		1		
6	Idaho (1)		1		
	Idaho (2)	1			
7	Illinois (1)	1			
	Illinois (2)			1	
8	Indiana	1			
9	Maryland	1			
10	Massachusetts	1			
11	Michigan (1)			1	
	Michigan (2)			1	
	Michigan (3)			1	
	Michigan (4)	1			
12	Minnesota			1	
13	Missouri		1		
14	Montana		1		
15	Nebraska			1	
16	Nevada	1			
17	New Hampshire				NR
18	New Jersey	1			
19	New Mexico (1)				
	New Mexico (2)	1			
	New Mexico (3)	1			
20	New York (1)		1		
	New York (2)			1	
21	North Dakota				NR
22	Tennessee (1)			1	
	Tennessee (2)			1	
	Tennessee (3)			1	
23	Texas	1			
24	Vermont			1	
25	Virginia (1)		1		
	Virginia (2)			1	
26	Washington		1		

Table A-7. Response to Question 1 (f)

	States	How frequently do you observe any of the following premature distress conditions in your bridge barriers? (f) Multiple vertical cracking near the barrier toe			
		High	Medium	Low	Comments
1	Alabama (1)	1			
	Alabama (2)		1		
2	Connecticut	1			
3	D.C.	1			
4	Florida	1			
5	Hawaii		1		
6	Idaho (1)		1		
	Idaho (2)		1		
7	Illinois (1)	1			
	Illinois (2)	1			
8	Indiana		1		
9	Maryland	1			
10	Massachusetts				NR
11	Michigan (1)		1		
	Michigan (2)		1		
	Michigan (3)		1		
	Michigan (4)		1		
12	Minnesota		1		
13	Missouri	1			
14	Montana	1			NR
15	Nebraska	1			
16	Nevada	1			
17	New Hampshire				NR
18	New Jersey	1			
19	New Mexico (1)			1	
	New Mexico (2)	1			
	New Mexico (3)		1		
20	New York (1)		1		
	New York (2)			1	
21	North Dakota	1			
22	Tennessee (1)		1		
	Tennessee (2)	1			
	Tennessee (3)	1			
23	Texas				
24	Vermont			1	
25	Virginia (1)		1		
	Virginia (2)	1			
26	Washington		1		

Table A-8. Response to Question 1 (g)

	States	How frequently do you observe any of the following premature distress conditions in your bridge barriers? (g) Severe section loss at or near the top of the barrier			
		High	Medium	Low	Comments
1	Alabama (1)	1			
	Alabama (2)	1			
2	Connecticut	1			
3	D.C.	1			
4	Florida	1			
5	Hawaii	1			
6	Idaho (1)	1			
	Idaho (2)	1			
7	Illinois (1)	1			
	Illinois (2)	1			
8	Indiana	1			
9	Maryland	1			
10	Massachusetts	1			
11	Michigan (1)	1			
	Michigan (2)		1		
	Michigan (3)	1			
	Michigan (4)			1	
12	Minnesota	1			
13	Missouri	1			
14	Montana				NR
15	Nebraska				NR
16	Nevada	1			
17	New Hampshire				NR
18	New Jersey				None
19	New Mexico (1)	1			
	New Mexico (2)	1			
	New Mexico (3)	1			
20	New York (1)				NR
	New York (2)	1			
21	North Dakota				NR
22	Tennessee (1)	1			
	Tennessee (2)	1			
	Tennessee (3)	1			
23	Texas				
24	Vermont		1	1	
25	Virginia (1)		1		
	Virginia (2)	1			
26	Washington	1			

Table A-9. Response to Question 1 (h)

	States	How frequently do you observe any of the following premature distress conditions in your bridge barriers? (h) Local pop out			
		High	Medium	Low	Comments
1	Alabama (1)	1			
	Alabama (2)	1			
2	Connecticut	1			
3	D.C.	1			
4	Florida	1			
5	Hawaii	1			
6	Idaho (1)	1			
	Idaho (2)	1			
7	Illinois (1)	1			
	Illinois (2)	1			
8	Indiana	1			
9	Maryland	1			
10	Massachusetts				NR
11	Michigan (1)	1			
	Michigan (2)		1		
	Michigan (3)		1		
	Michigan (4)		1		
12	Minnesota		1		
13	Missouri	1			
14	Montana	1			NR
15	Nebraska				
16	Nevada	1			
17	New Hampshire				NR
18	New Jersey				None
19	New Mexico (1)	1			
	New Mexico (2)	1			
	New Mexico (3)	1			
20	New York (1)	1			
	New York (2)	1			
21	North Dakota				NR
22	Tennessee (1)	1			
	Tennessee (1)	1			
	Tennessee (1)	1			
23	Texas				
24	Vermont	1			
25	Virginia (1)		1		
	Virginia (2)	1			
26	Washington	1			

Table A-10. Response to Question 1 (i)

	States	How frequently do you observe any of the following premature distress conditions in your bridge barriers? (i) Signs of corrosion due to lack of concrete cover			
		High	Medium	Low	Comments
1	Alabama (1)	1			
	Alabama (2)	1			
2	Connecticut		1		
3	D.C.	1			
4	Florida	1			
5	Hawaii	1			
6	Idaho (1)	1			
	Idaho (2)	1			
7	Illinois (1)	1			
	Illinois (2)		1		
8	Indiana		1		
9	Maryland	1			
10	Massachusetts	1			NR
11	Michigan (1)	1			
	Michigan (2)			1	
	Michigan (3)		1		
	Michigan (4)	1			
12	Minnesota	1			
13	Missouri	1			
14	Montana	1			
15	Nebraska				NR
16	Nevada	1			
17	New Hampshire				NR
18	New Jersey				None
19	New Mexico (1)	1			
	New Mexico (2)	1			
	New Mexico (3)	1			
20	New York (1)				NR
	New York (2)		1		
21	North Dakota				NR
22	Tennessee (1)		1		
	Tennessee (2)	1			
	Tennessee (3)	1			
23	Texas	1			
24	Vermont		1		
25	Virginia (1)		1		
	Virginia (2)		1		
26	Washington	1			

Table A-11. Response to Question 2

	States	Do you observe an overall durability problem with the NJ-type concrete bridge barriers?		
		Yes	No	Comments
1	Alabama (1)		1	
	Alabama (2)		1	
2	Connecticut		1	
3	D.C.		1	
4	Florida		1	
5	Hawaii		1	
6	Idaho (1)		1	
	Idaho (2)		1	
7	Illinois (1)	1		
	Illinois (2)	1		
8	Indiana		1	
9	Maryland			
10	Massachusetts		1	
11	Michigan (1)	1		
	Michigan (2)	1		
	Michigan (3)	1		
	Michigan (4)	1		
12	Minnesota		1	
13	Missouri		1	
14	Montana		1	
15	Nebraska		1	
16	Nevada		1	
17	New Hampshire		1	
18	New Jersey		1	
19	New Mexico (1)	1		
	New Mexico (2)		1	
	New Mexico (3)		1	
20	New York (1)		1	
	New York (2)		1	
21	North Dakota		1	
22	Tennessee (1)		1	
	Tennessee (2)		1	
	Tennessee (3)		1	
23	Texas		1	
24	Vermont	1		
25	Virginia (1)	1		
	Virginia (2)		1	
26	Washington		1	

Table A-12. Response to Question 3

	States	What construction procedures are being used for casting NJ-type barriers in your agency?		
		Cast in place	Slip form	Others
1	Alabama (1)		1	
	Alabama (2)		1	
2	Connecticut	1		
3	D.C.	1		
4	Florida	1		Precast
5	Hawaii	1		
6	Idaho (1)	1		
	Idaho (2)	1		
7	Illinois (1)	1	1	
	Illinois (2)	1	1	
8	Indiana	1	1	
9	Maryland	1	1	Precast
10	Massachusetts	1		
11	Michigan (1)	1	1	
	Michigan (2)		1	
	Michigan (3)		1	
	Michigan (4)	1	1	
12	Minnesota	1	1	80% slip form, 20% cast in place
13	Missouri	1	1	
14	Montana	1	1	
15	Nebraska	1	1	
16	Nevada	1	1	
17	New Hampshire	1	1	Precast
18	New Jersey	1	1	
19	New Mexico (1)	1	1	
	New Mexico (2)	1	1	
	New Mexico (3)	1	1	
20	New York (1)	1	1	Precast
	New York (2)	1	1	Precast
21	North Dakota	1	1	
22	Tennessee (1)	1	1	
	Tennessee (2)	1	1	
	Tennessee (3)		1	
23	Texas	1	1	Precast
24	Vermont			Precast
25	Virginia (1)		1	Precast
	Virginia (2)	1	1	
26	Washington	1	1	

Table A-13. Response to Question 4

	States	What curing procedures are being used/ specified for barriers?	
		Yes	Comments
1	Alabama (1)	Same as for cast in place concrete	
	Alabama (2)	Spray on curing compound.	
2	Connecticut	Moist curing for seven days	
3	D.C.	Standard wet burlap	
4	Florida	Moisture, steam curing & membrane curing compound	
5	Hawaii	Liquid membrane forming compound	
6	Idaho (1)	Spray on curing compound	
	Idaho (2)	Curing compound	
7	Illinois (1)	Wet burlap or curing compound	
	Illinois (2)	Wet burlap or curing compound	
8	Indiana	Burlap or membrane forming curing compound	
9	Maryland	Wet burlap or curing compound	
10	Massachusetts	2-3 days curing in the form	
11	Michigan (1)	Curing compound	
	Michigan (2)		NR
	Michigan (3)	Curing compound 1	
	Michigan (4)	Pigmented curing compound	
12	Minnesota	Wet burlene sheets	
13	Missouri		DN
14	Montana	Spray on curing	
15	Nebraska	Wet burlap cover	
16	Nevada	Wax based curing compound	
17	New Hampshire	Seven days of curing	
18	New Jersey	Curing Compound	
19	New Mexico (1)	Curing compound 1	
	New Mexico (2)	Curing Compound for seven days	
	New Mexico (3)	Wet burlap cover	
20	New York (1)	Curing compound for seven days	
	New York (2)	Curing compound 1	
21	North Dakota	Wet curing	
22	Tennessee (1)	Curing compound	
	Tennessee (2)	Curing compound 1	
	Tennessee (3)	Curing compound (membrane forming)	
23	Texas	Non resin based pigmented curing compound	
24	Vermont	Moist curing for seven days	
25	Virginia (1)	High grade curing compound	
	Virginia (2)	Curing compound	
26	Washington	Blanket-Cast in place; Slip form: Curing compound	

Table A-14. Response to Question 5

	States	Is your Agency specifying epoxy coated reinforcement for the barriers?		
		Yes	No	If yes, please specify since when
1	Alabama (1)		1	
	Alabama (2)		1	
2	Connecticut	1		NR
3	D.C.	1		NR
4	Florida		1	
5	Hawaii		1	
6	Idaho (1)	1		1975
	Idaho (2)	1		NR
7	Illinois (1)	1		NR
	Illinois (2)	1		Many years
8	Indiana	1		1980
9	Maryland	1		Late 1970's
10	Massachusetts	1		About 1980's
11	Michigan (1)	1		NR
	Michigan (2)	1		NR
	Michigan (3)	1		Early 1980's
	Michigan (4)	1		1984
12	Minnesota	1		Circa 1985
13	Missouri	1		Date unknown
14	Montana	1		Last 5-10 years
15	Nebraska	1		About 1994
16	Nevada	1	1	Epoxy is not used in South Nevada
17	New Hampshire		1	NR
18	New Jersey	1		Over 20 years
19	New Mexico (1)		1	NR
	New Mexico (2)	1		At least 14 years
	New Mexico (3)	1		Approximately Late 80's
20	New York (1)	1		Always
	New York (2)	1		Mid 1980's
21	North Dakota	1		Only started a couple of years ago
22	Tennessee (1)	1		Since early 80's
	Tennessee (2)	1		Unknown
	Tennessee (3)	1		Bridges only +10 years
23	Texas	1		Vary rarely
24	Vermont	1		Unsure but possibly early mid 1990's
25	Virginia (1)	1		NR
	Virginia (2)	1		Early 1980's
26	Washington	1		Inside face of barrier to slab only

Table A-15. Response to Question 6

	States	Are coatings/sealants currently being specified for barriers?		
		Yes	No	Comments
1	Alabama (1)	1		
	Alabama (2)		1	
2	Connecticut		1	
3	D.C.	1		
4	Florida	1		
5	Hawaii		1	
6	Idaho (1)		1	
	Idaho (2)		1	
7	Illinois (1)	1		
	Illinois (2)		1	
8	Indiana	1		
9	Maryland		1	
10	Massachusetts		1	
11	Michigan (1)	1		
	Michigan (2)		1	
	Michigan (3)		1	
	Michigan (4)	1		
12	Minnesota		1	
13	Missouri		1	
14	Montana		1	
15	Nebraska	1		
16	Nevada	1		
17	New Hampshire		1	
18	New Jersey		1	
19	New Mexico (1)	1		
	New Mexico (2)	1		
	New Mexico (3)	1		
20	New York (1)	1		
	New York (2)	1		
21	North Dakota	1		
22	Tennessee (1)	1		
	Tennessee (2)	1		
	Tennessee (3)	1		
23	Texas	1		
24	Vermont		1	
25	Virginia (1)	1		
	Virginia (2)		1	
26	Washington	1		

Table A-16. Response to Question 7

	States	Does your Agency specify a concrete mix design for the barriers different than the deck?	
		Please describe	Comments
1	Alabama (1)	No	
	Alabama (2)	No	
2	Connecticut	No	
3	D.C.		NR
4	Florida	Different from deck	
5	Hawaii	Generally no	
6	Idaho (1)	No	
	Idaho (2)	Same as deck	
7	Illinois (1)	Different from deck	
	Illinois (2)	40 lbs less cement used for central mixed concrete	
8	Indiana	No	
9	Maryland	Same as deck (4500 psi)	
10	Massachusetts	Silica fume modified concrete with 6% silica fume	
11	Michigan (1)	For slip form barriers, minimum air content is 4.5%	
	Michigan (2)	Standard Specification	NR
	Michigan (3)		NR
	Michigan (4)	No	
12	Minnesota	Different from deck	
13	Missouri		NR
14	Montana	Same as deck	
15	Nebraska	No	
16	Nevada	Deck concrete is generally the same as rail concrete	
17	New Hampshire	Different than deck	
18	New Jersey	No	
19	New Mexico (1)	No	
	New Mexico (2)	Different than deck	
	New Mexico (3)	Depends on slip formed or cast in place	
20	New York (1)	Different than deck (Class H or Class HP)	
	New York (2)	Class A for CIP, Class J for Slip formed	
21	North Dakota		NR
22	Tennessee (1)		DN
	Tennessee (2)	3000 psi mix	
	Tennessee (3)	Lower slump etc for slip forming	
23	Texas	No	
24	Vermont		DN
25	Virginia (1)	Use low slump mixes with lower air content	
	Virginia (2)	Same design strength but smaller aggregate	
26	Washington	Deck uses concrete class 400 D with min FA content of 75 lb/yd^3 &minimum cement content of 660 lb/yd^3; barrier uses a minimum cement content of 565 lb/yd^3	

Table A-17. Response to Question 8

	States	Has your Agency used ground GBFS, FA or other pozzolans in the mix for the reduction of the permeability of barrier concrete?		
		Yes	No	If yes, please specify
1	Alabama (1)	1		Per job basis
	Alabama (2)	1		Up to 25% of substitution rate
2	Connecticut		1	
3	D.C.		1	
4	Florida	1		Varies depending on application
5	Hawaii		1	
6	Idaho (1)	1		FA occasionally min 20%
	Idaho (2)	1		Dependent upon mix design
7	Illinois (1)		1	
	Illinois (2)	1		GBFS- max 25% & Class C FA-max 20%& Class F FA- max 15%
8	Indiana	1		30%
9	Maryland	1		GBFS 50% max slip form
10	Massachusetts	1		6% silica fume, 15% FA, 25-40% GBFS
11	Michigan (1)	NR		DN
	Michigan (2)		1	
	Michigan (3)	1		Optional
	Michigan (4)	1		FA
12	Minnesota	1		15%
13	Missouri	NR		NR
14	Montana	1		5+ %
15	Nebraska		1	
16	Nevada		1	
17	New Hampshire	1		Varies
18	New Jersey		1	
19	New Mexico (1)	1		FA-25% as of 1999
	New Mexico (2)	1		FA-20%
	New Mexico (3)	NR		NR
20	New York (1)	1		Class HP requires 20% FA &6% silica fume
	New York (2)		1	
21	North Dakota	1		Up to 30% FA is optional
22	Tennessee (1)		1	
	Tennessee (2)	1		Maximum 25% replacement
	Tennessee (3)		1	
23	Texas	1		Numerous FA mixers exist
24	Vermont	DN		
25	Virginia (1)	1		Max 50% slag, 35% FA, 10% SF
	Virginia (2)		1	
26	Washington	1		

Table A-18. Response to Question 9

	States	What is the surface finish method that is used for barrier concrete?			
		Sacked	Rubbed	Other	Comments
1	Alabama (1)		1		
	Alabama (2)	1			
2	Connecticut		1		
3	D.C.		1		
4	Florida			Class 3 surface finish	
5	Hawaii		1		
6	Idaho (1)	1			
	Idaho (2)	1			
7	Illinois (1)			Light brush finish	
	Illinois (2)		1		
8	Indiana		1	Trowel smooth-top of the barrier	
9	Maryland		1		
10	Massachusetts			As - cast	
11	Michigan (1)		1		
	Michigan (2)	1		1	
	Michigan (3)		1		
	Michigan (4)		1		
12	Minnesota		1(CIP)	Slip form-lightly broom brushed,	
13	Missouri				NR
14	Montana			Broomed	
15	Nebraska		1		
16	Nevada			Grind fins & sprayed coating	
17	New Hampshire			As cast	
18	New Jersey		1		
19	New Mexico (1)		1		
	New Mexico (2)				NR
	New Mexico (3)				NR
20	New York (1)			Minor repair as necessary	
	New York (2)			Hand finish if necessary	
21	North Dakota	1			
22	Tennessee (1)		1		
	Tennessee (2)	1			
	Tennessee (3)		1		
23	Texas		1	Off the form finish	
24	Vermont			Form finished	
25	Virginia (1)			Form finished	
	Virginia (2)			Fill in voids	
26	Washington	1			

Table A-19. Response to Question 10

	States	Do you observe differences in performance between barriers on rural areas versus trunk line/interstate routes?		
		Yes	No	If yes, please explain why?
1	Alabama (1)		1	
	Alabama (2)	NR		
2	Connecticut	NR		
3	D.C.	NR		
4	Florida	NR		
5	Hawaii	NR		
6	Idaho (1)	NR		
	Idaho (2)	NR		
7	Illinois (1)	NR		
	Illinois (2)	DN		
8	Indiana	NR		
9	Maryland	NR		
10	Massachusetts	1		NR
11	Michigan (1)	1		Salt spray
	Michigan (2)	NR		
	Michigan (3)	NR		
	Michigan (4)	NR		FA
12	Minnesota	1		Trunk line receives more salt than local routes
13	Missouri	1		Because of traffic &deicing materials
14	Montana	1		More vertical cracking on barriers on interstate
15	Nebraska	None		
16	Nevada	NR		
17	New Hampshire	NR		
18	New Jersey		1	
19	New Mexico (1)	1		More problems in urban area where barriers get splashed with water
	New Mexico (2)	NR		
	New Mexico (3)	NR		
20	New York (1)	NR		
	New York (2)	1		Lower salt, better overall performance
21	North Dakota	NR		
22	Tennessee (1)	NR		
	Tennessee (2)	NR		
	Tennessee (3)	NR		
23	Texas	1		Length of spans affect cracking, typically longer span & truck impacts more severe on curved structures in urban areas,
24	Vermont	DN		
25	Virginia (1)	1		Increased salt content on snow pack
	Virginia (2)	NR		
26	Washington	NR		

Table A-20. Response to Question 11

	States	Does your Agency apply deicers to the bridge deck?		
		Yes	No	If yes, what kind of deicers?
1	Alabama (1)	1		CMA
	Alabama (2)		1	
2	Connecticut	1		Salt
3	D.C.		1	
4	Florida		1	
5	Hawaii		1	
6	Idaho (1)	1		Salt-Magnesium Chloride
	Idaho (2)	1		Magnesium Chloride
7	Illinois (1)	1		Very heavy salt/brine
	Illinois (2)	1		Typically salt or salt brine
8	Indiana	1		Salt
9	Maryland	1		Salt & Magnesium Chloride
10	Massachusetts	1		NR
11	Michigan (1)	1		Salt &CMA
	Michigan (2)	1		Salt
	Michigan (3)	1		Salt (CMA)
	Michigan (4)	1		Salt
12	Minnesota	1		Salt
13	Missouri	1		Salt along with salt brine as pretreatment
14	Montana	1		Freeze guard (Magnesium Chloride)
15	Nebraska	1		NR
16	Nevada	1	1	Northern part only
17	New Hampshire	1		Salt
18	New Jersey		1	
19	New Mexico (1)	1		Salt
	New Mexico (2)	1		CMA & salt
	New Mexico (3)	1		Calcium or Magnesium Chloride
20	New York (1)	1		Salts mostly, some liquids also
	New York (2)	1		Salt
21	North Dakota	1		Salt
22	Tennessee (1)	1		Salt
	Tennessee (2)	1		NR
	Tennessee (3)	1		Salt
23	Texas	1		CMA, salt sand,
24	Vermont	1		Salt & Chloride
25	Virginia (1)	1		CMA, salt, Calcium Chloride
	Virginia (2)	1		Salts
26	Washington	1		Urea

Table A-21. Response to Question 12

	States	Has your Agency initiated any changes to improve the durability of NJ-type barriers?			
		Changing the mix design	Changing curing procedure	Other Changes	Comment
1	Alabama (1)				NR
	Alabama (2)				NR
2	Connecticut				NR
3	D.C.				NR
4	Florida			Discontinued	
5	Hawaii				None
6	Idaho (1)				NR
	Idaho (2)				None
7	Illinois (1)			In depth review	
	Illinois (2)				None
8	Indiana			Epoxy coated rebar	
9	Maryland				NR
10	Massachusetts	1			
11	Michigan (1)		1	Sealers are used	
	Michigan (2)				
	Michigan (3)		1		
	Michigan (4)			Improved consolidation	
12	Minnesota	1	1	Added FA, epoxy coated rebar	
13	Missouri				DN
14	Montana		1		
15	Nebraska				None
16	Nevada				NR
17	New Hampshire		1		
18	New Jersey				NR
19	New Mexico (1)	1		Added FA to counter act ASR problem	
	New Mexico (2)			Constant slope barrier design	
	New Mexico (3)				NR
20	New York (1)	1		HP mixture	
	New York (2)	1			
21	North Dakota				NR
22	Tennessee (1)				DN
	Tennessee (2)				NR
	Tennessee (3)		1	Placing grooved joints every 10 ft	
23	Texas				None
24	Vermont				DN
25	Virginia (1)	1	1	Use of admixture to control alkali silica action	
	Virginia (2)				NR
26	Washington				None

APPENDIX B

Laboratory Data Sheets

Project Name : CAUSES & CURES FOR CRACKING OF CONCRETE BARRIERS

Test Name :Determination of Air Content of Freshly Mixed Concrete by the Pressure Method

Related Code : (ASTM C231)

Conducted By : ____

Date : ____

- Dampen the interior of measuring bowl and place it on a flat, level, firm surface. Place fresh concrete in three equal layers. Consolidate each layer by rodding (if slump>3 in), by either rodding or vibrating (if 1<slump<3 in), by vibrating (if slump<1 in).
- Strike-off the top surface of concrete and finish it smoothly with a flat strike-off plate using great care to leave the measuring bowl just level full.
- After strike-off clean all excess concrete from the exterior of the measuring bowl (base rim) and cover. Then clamp base rim and cover.
- Open petcocks. Inject water through one petcock until water is expelled from opposite petcock.
- Close air bleeder valve on air chamber. Close petcocks. Pump up air to the marked point on gauge. Wait a few seconds and tap gauge lightly. If necessary add or subtract air to attain reading at the marked point.
- Press needle valve lever to release air into base. Continue pressing lever and lightly tap gauge. Read direct percentage of air.
- Thoroughly clean base, cover and petcock openings with running water.

A (Air Content, %) = ______

Project Name: Causes & Cures for Cracking of Concrete Barriers

Test Name : RCPT Test

Related Code : (ASTM C 1202-91)

Conducted By : ____

Date: ____

Specimen No :
Cell No :
Diameter (in) :
Length (in) :
Vacuum Saturation Started :
Saturation Ended :
Initial Specimen Weight before Saturation (g) :
In-water Specimen Weight After Saturation (g) :
Surface Dry Specimen Weight After Saturation (g) :

Time	Voltage (mV)	Current (Amp)	Temp in NaCl (F)	Temp in NaOH (F)

Project Name: CAUSES & CURES FOR CRACKING OF CONCRETE BARRIERS

Test Name : Determination of Slump of Concrete

Related Code : (ASTM C143)

Conducted By : ______________________________

Date: ______________________________

- Fill the mold with concrete in three layers of equal volume, (one third of the volume of the mold fills it approximately to a depth of 6.7 cm; two thirds of the volume approximately fills it to a depth of 15.5 cm) each time rodding the layers with 25 strokes of tamping rod evenly distributed on the surface.
- Strike-off the top surface of concrete and finish it smoothly by rolling the damping rod.
- Remove the mold by raising it vertically and put the mold near the concrete.
- Measure the decrease in height to the nearest 6 mm.

Slump = _____

	Specimen Number	Length 1	Length 2	Length 3	Diameter 1	Diameter 2	Diameter 3
		(inch)					
1							
2							
3							
4							
5							
6							
7							
8							
9							
10							
11							
12							
13							
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27							
28							
29							

Project Name: CAUSES & CURES FOR CRACKING OF CONCRETE BARRIERS

Test Name: Determination of Unit Weight of Concrete

Related Code: (ASTM C138)

Conducted By : ____

Date: ____

- Determine the volume of the measure. (V)
- Determine the mass of the measure. (T)
- Fill the measure with concrete in three layers each time rodding the layers with 25 strokes of tamping rod evenly distributed on the surface. After rodding each layer tap the sides of measure.
- Strike-off the top surface of concrete and finish it smoothly with a flat strike-off plate using great care to leave the measure just level full.
- After strike-off clean all excess concrete from the exterior of the measure.
- Determine the mass of the measure plus its contents. (M)

V (Volume of measure) = _____

G (Mass of conc. and measure) = _____

T (Mass of measure) = _____

M (Unit weight of concrete) = (G-T)/V = ____

Project Name: CAUSES & CURES FOR CRACKING OF CONCRETE BARRIERS

Test Name : Absorption Test

Related Code :

Conducted By : ____

Specimen No					
Weight 1 (gr)		**Date**		**Time**	
Weight 2 (gr)		**Date**		**Time**	
Weight 3 (gr)		**Date**		**Time**	
Weight 4 (gr)		**Date**		**Time**	
Dry Diameter (in)	**1**	**2**	**3**		
Dry Length (in)	**1**	**2**	**3**		

Saturation Starts

Weight 5 (gr)		**Date**		**Time**	
Weight 6 (gr)		**Date**		**Time**	
Weight 7 (gr)		**Date**		**Time**	
Weight 8 (gr)		**Date**		**Time**	

Start boiling for 5 hours

Weight after boiling (gr)		**Date**		**Time**	
Weight in water (gr)		**Date**		**Time**	
Saturated Diameter (in)	**1**	**2**	**3**		
Saturated Length (in)	**1**	**2**	**3**		