NYSDOT Bridge Deck Task Force Evaluation of Bridge Deck Cracking on NYSDOT Bridges

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Abstract: This paper presents a summary of knowledge gained by the NYSDOT Bridge Deck Task Force (BDTF). Although the paper contains recommendations to reduce the prevalence and severity of bridge deck cracking, it does not include a 'silver bullet' solution. There are discussions concerning cracking of the concrete wearing surface on prestressed concrete box girders, transverse cracking of concrete decks on steel girders, factors that influence concrete deck cracking, and methods of treating existing cracks in bridge decks. To limit the number of variables in the data, the study was limited to single span structures. Cracking of multi-span continuous structures was not included.

1. Introduction

Bridge deck cracking is a continuing problem throughout the United States, including New York State. The cracking typically appears within the first few days of the deck being cast and has not been isolated to any particular bridge type. Cracking has been found on bridges with concrete beams and steel beams, and on single and continuous spans. Although the existence of the problem has been widely accepted and identified, no real solutions have been found. New bridges continue to be built with cracks that appear within the first few months of service.

In the spring of 2000, the New York State Department of Transportation (NYSDOT) formed the BDTF to investigate the issue of deck cracking in recently constructed bridge decks. The goals of the BDTF were to minimize deck cracking in new bridges as much as practical, and to develop guidance for treating cracking in existing decks. Members were selected from the NYSDOT Office of Structures, Office of Construction, Transportation Maintenance Division, Materials Bureau and the Federal Highway Authority.

During the past several years, the BDTF has conducted literature reviews, engaged in field surveys of bridge decks in northern New York, and contacted other state DOTs regarding their experience with cracked bridge decks. To further understand the physical properties of High Performance (HP) concrete compared to other concretes commonly used in bridge construction, the BDTF initiated research project SPR C-02-03 – Bridge Deck Material Properties. This research project is on-going.

2.0 Adjacent Prestressed Concrete Bridges

2.1 Historical Designs

The earliest prestressed concrete box girder bridges in Region 7 date to 1960. These structures have a 5 - 8 in. (130-200 mm) asphalt overlay placed directly on the top of the adjacent prestressed units. A continuous grouted keyway extended from the top of the beam to approximately ¼ of the beam depth. Transverse tendons were installed approximately every 20 ft. (6 m) along the beams.

Figure 1 - BIN 1034130 – Adjacent Prestressed Concrete Beam Bridge with Asphalt Wearing Surface, Built in 1960.

In the mid to late 1970s, adjacent box girder bridges were built with a concrete wearing surface. Typically, the concrete wearing surface was 6 in. (150 mm) thick and reinforced with wire mesh fabric. Transverse tendons were typically placed at midspan, regardless of the span length.

In 1992, the standard design of prestressed concrete box girders was modified in an effort to decrease the prevalence of longitudinal concrete deck cracking. The keyway was lengthened to nearly the full depth between adjacent boxes. The number of and locations for transverse tendons were also increased. For spans up to 50 ft. (15 m), a tendon was placed at midspan and one approximately 2 ft. (600 mm) from each end. Longer spans had additional transverse tendons placed at quarter points of the beam.

In 1997, and Engineering Bulletin was issued to require high pressure washing of the beams and 12 hours of continuous wetting of the beams before concrete deck placement.

In 2002, NYSDOT changed the construction specifications for deck placement to eliminate bonding grout, require physical abrading of the concrete surface and soaking of the box beams for 12 hours (min.) prior to placing the concrete deck. Also, three transverse tendons are used per location to provide greater clamping force across the shear key joint between beams. Additionally, a single mat of #4 bars are used instead of wire mesh reinforcement.

2.2 Survey of Existing Decks

It is difficult to determine the effectiveness of two course decks using asphalt, since the asphalt covers any cracks that may exist. The danger of this is that asphalt is not waterproof, and chloride laden water is free to seep into any cracks that exist below.

Performance of bridges constructed in the 1970's is varied, with some bridges performing quite well with others exhibiting severe longitudinal cracking directly above the grouted keyway.

The 2002 change in the wetting procedure has made a significant improvement in the appearance of pattern cracking on newly placed bridge decks. There is not enough information to determine if the additional transverse tendons have had any positive impact. There is no indication that they have been a detriment.

2.3 Factors Influencing Deck Cracking

Cracking of concrete decks on prestressed concrete box beams appears to be of two types, longitudinal cracking which follows the beam joints and random cracking with relatively large spacing of the cracks. The random crack spacing typically divides the deck into 100 ft^2-200 ft^2 (10-20 m^2) segments.

Figure 2 - BIN 334081 Example of Longitudinal Cracking

Longitudinal cracking generally follows the beam keyways, as shown in Figure 2. This type of cracking appears to be caused by the differential movement of the beams at the keyway. One possible cause of the differential movement between beams is rotation of the beams about their longitudinal axis.
This relative movement between adjacent box beams is extremely difficult to eliminate. By nature, all of the differential movement between adjacent boxes must occur in the plane between beams. As a retrofit for existing adjacent beam bridges with the short shear key, NYSDOT Bridge Maintenance forces have installed hard composite wedges at the bottom of the beams to prevent this rotation. For newly constructed bridges, the deeper shear key and the higher transverse tendon force are intended to force the bridge to act closer to a unified slab rather than as individual beams.


Figure 3 - BIN 100069 Example of Pattern Cracking

Instrumentation was installed to measure the stress in the individual beams before and after the installation of the composite shims. The measurements showed that the installation of the wedges reduced relative movements between beams and improved the distribution of the live load, but the rigid overlay placed after the installation of these wedges still cracked.

Random pattern cracking, similar to that shown in Figure 3, is also prevalent on prestressed box beam bridges. This type of cracking has been considered caused largely by placement of wet concrete on dry precast beams. Concrete cores taken at various locations on a distressed concrete deck showed that the cracks were initiating and propagating from the bottom of the concrete deck.

3.0 Concrete Bridge Decks on Spread Steel Girders

3.1 Historical Designs

The NYSDOT has modified the design of the standard bridge deck several times over the past 30 years. During the Interstate construction era (1955-1975), the standard design for a bridge deck on spread steel beams consisted of a 7 in. (175 mm) concrete deck with a separate 2.5-4 in. (65-100 mm) separate asphalt or concrete wearing surface. Many of these structures had a waterproofing membrane between the concrete deck and the wearing surface.

A side benefit of using the asphalt wearing surface is that it hides any concrete deck cracks. Some of these bridge decks are still in service. Some have had the wearing surface replaced in kind and others have been rehabilitated with a bonded concrete overlay.

In the late 1960s, NYSDOT started constructing monolithic bridge decks with a sacrificial wearing surface poured monolithically with the structural deck. Original designs called for 7.5 in. (190 mm) thick deck with 1.5 in. (38 mm) of top bar cover. This basic design was modified at various times during the last 35 years to improve the performance and the service life. Table 1 summarizes these changes:

Year	Deck Design
1967	First monolithic deck. 7.5 in (190 mm) total thickness with 1.5 in (38 mm) cover on the top bars. Uncoated reinforcement in both mats.
1974	Deck thickness increased to 9.25 in (235 mm) with 3.25 in (83 mm) top cover. Uncoated reinforcement in both mats.
1976	Deck thickness reduced to 8.5 in (216 mm) with 2.5 in (64 mm) top cover. Epoxy coated reinforcing for top mat, uncoated reinforcement for bottom mat.
1992	Deck thickness increased to 9.5 in (240 mm) with 3.5 in. (90 mm) top cover. Epoxy coated reinforcing for top mat, uncoated reinforcement for bottom mat.
1996	9.5 in. (240 mm) deck thickness, 3.0 in. (75 mm) cover on longitudinal top bars, isotropic reinforcing design available to designers. Epoxy coated reinforcing for top mat, uncoated reinforcement for bottom mat.
2006	Current Practice. 9.5 in. (240 mm) deck thickness, 3.0 in. (75 mm) cover on longitudinal top bars, isotropic reinforcing design available to designers. Epoxy coated or galvanized reinforcing for both reinforcement mats. Stainless steel and stainless steel clad reinforcing bars are allowed in special cases.

Table 1 – Deck Design Timeline

Current bridge deck standards call for a 9.5 in. (240 mm) deck with 3 in. (75 mm) cover over top steel and 1.5 in. (35 mm) of cover below the bottom steel. Longitudinal bars are placed on top of the transverse bars in the top of the deck. Both mats of reinforcing consist of epoxy coated or galvanized bars. High Performance Concrete designed for a low permeability is used with a 28 day minimum strength of 3,000 psi (21 MPa). The preferred reinforcing design is isotropic bridge deck design with #4 bars on 8.0 in. (200 mm) centers in each direction for both the top and bottom mat. Slab overhang reinforcement is designed based on the level of service demanded by the railing system used on the structure.

3.2 Survey of Existing Decks

Several newly constructed bridges in northern NY were identified as having serious cracking. This prompted inspections and documentation of the extent of the bridge deck cracking. Figures 4 and 5 show the seriousness of the problem.

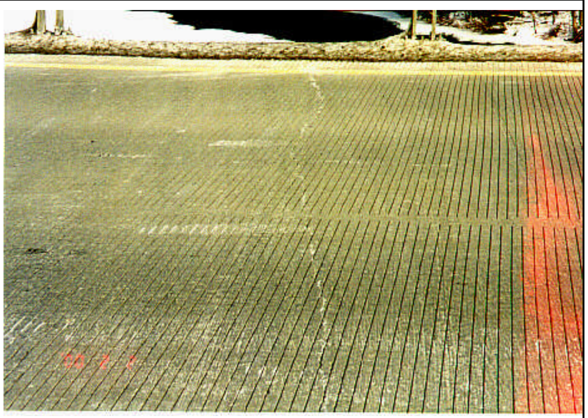


Figure 4 - BIN 1033992 Cracking observed in newly constructed deck.

Figure 5 - Cracking in BIN 3221420, Alcoa Road over Grasse River constructed in 2002.

A total of 63 concrete bridge decks on steel girders have been studied in northern NY. These decks were rated based on the degree of cracking and the frequency of cracking. Rating scales are listed in Tables 2 and 3:

Rating	Description
7	No Cracks
6	Used to shade between 5 and 7
5	Less than 4 cracks per span
4	Used to shade between 3 and 5
3	Average Spacing > 10 ft. (3 m)
2	Average Spacing 5-10 ft.(1.5-3 m)
1	Average Spacing < 5 ft. (1.5 m)

Table 2 – Cracking Frequency

Rating	Description
7	No Cracks
6	Used to shade between 5 and 7
5	All Cracks < 0.007 in. (0.18 mm)
4	Used to shade between 3 and 5
3	All Cracks < 0.016 in. (0.41 mm)
2	Used to shade between 1 and 3
1	Cracks > 0.02 in (0.5 mm)

Table 3 – Cracking Size

The results of this survey showed that 38% of the single span bridges had significant cracking (rated 5 or less) and 67% of the multiple span bridges had significant cracking. Of the 63 bridges studied, only 15 were not cracked.

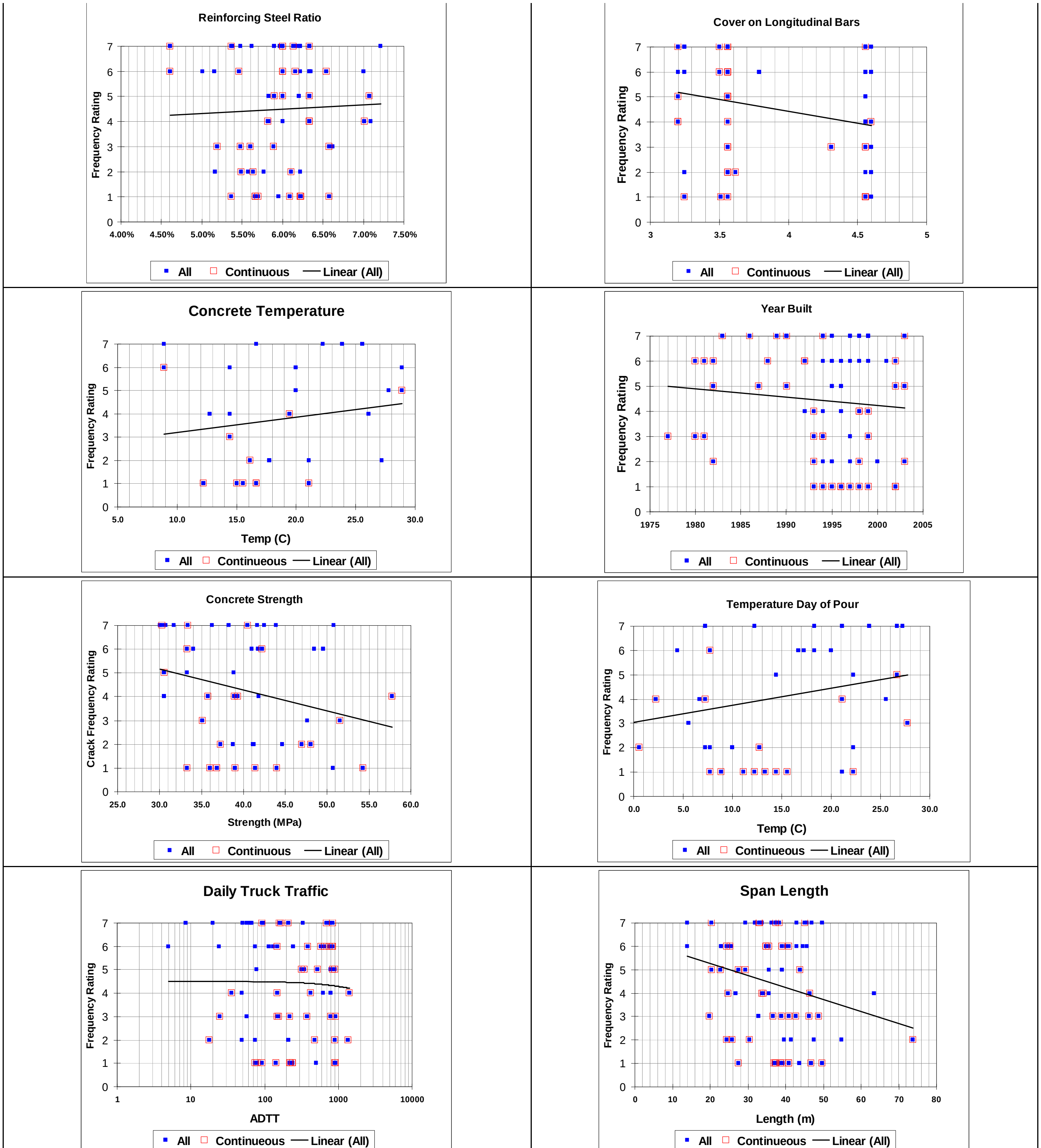


Figure 6 – Correlation of Various Parameters on Cracking Frequency

In addition to field rating the structures, construction records, as-built plans, and materials records were researched to create a database with all known parameters that affect the likelihood of cracking the bridge deck. Plots were created with the crack frequency rating as a function of each of several parameters as shown in Figure 6. A trend line was created using linear fit of the data. It should be noted that there is considerable scatter in the data and it is difficult to show a direct correlation for most of the parameters.

4.0 Factors Influencing Deck Cracking

Based on experience and information obtained from the plots presented in Figure 6, the following parameters have been determined to be the most influential factors that determine whether a concrete bridge deck cracks.

4.1 Concrete Strength

To minimize cracking, concrete should meet minimum strength requirements but not be excessively strong. However, it is costly and inefficient to batch concrete that may not meet the minimum required compressive strength. The NYSDOT HP concrete mix is designed to ensure that it meets the minimum strength of 3,000 psi (21.0 MPa) with 95% confidence (two standard deviations). This requires a target mean strength of approximately 5,000 psi (36 MPa), based on the statewide average compressive strength for an entire calendar year. Concrete curing under cold environmental conditions results in a lower average compressive strength. A higher average strength is obtained when the concrete is cured under warm environmental conditions. There is only one approved mix design for HP concrete in NYSDOT. Therefore, the mix design that ensures bridge decks completed late in the construction season under less than ideal conditions meets the minimum strength requirement also yields very high strengths when completed under more ideal conditions.

4.2 Concrete Cover

Crack control equations are typically a function proportional to $(d_cA)^{0.333}$ where d_c is the concrete cover to the center of the first layer of reinforcement and A is the area of the concrete in tension divided by the number of bars in tension. The cover on the reinforcing steel has increased over the past several years in an effort to delay the time it takes for chlorides applied to the surface in the form of de-icing salt to penetrate the concrete and attack the reinforcing steel. However, increased concrete cover leads to more cracks and larger crack widths.

4.3 Pour Temperature

The temperature the day of the pour and the concrete temperature would appear to have an effect on the degree of cracking. In both cases, warmer temperatures would appear to reduce the degree of cracking. However, it is not easy to control the temperature on the day of the deck pour due to scheduling, available equipment and manpower, and external pressures to complete a project. Significantly cooler overnight temperatures may also have an impact on the curing concrete.

5.0 Causes of Tension Stresses in the Deck

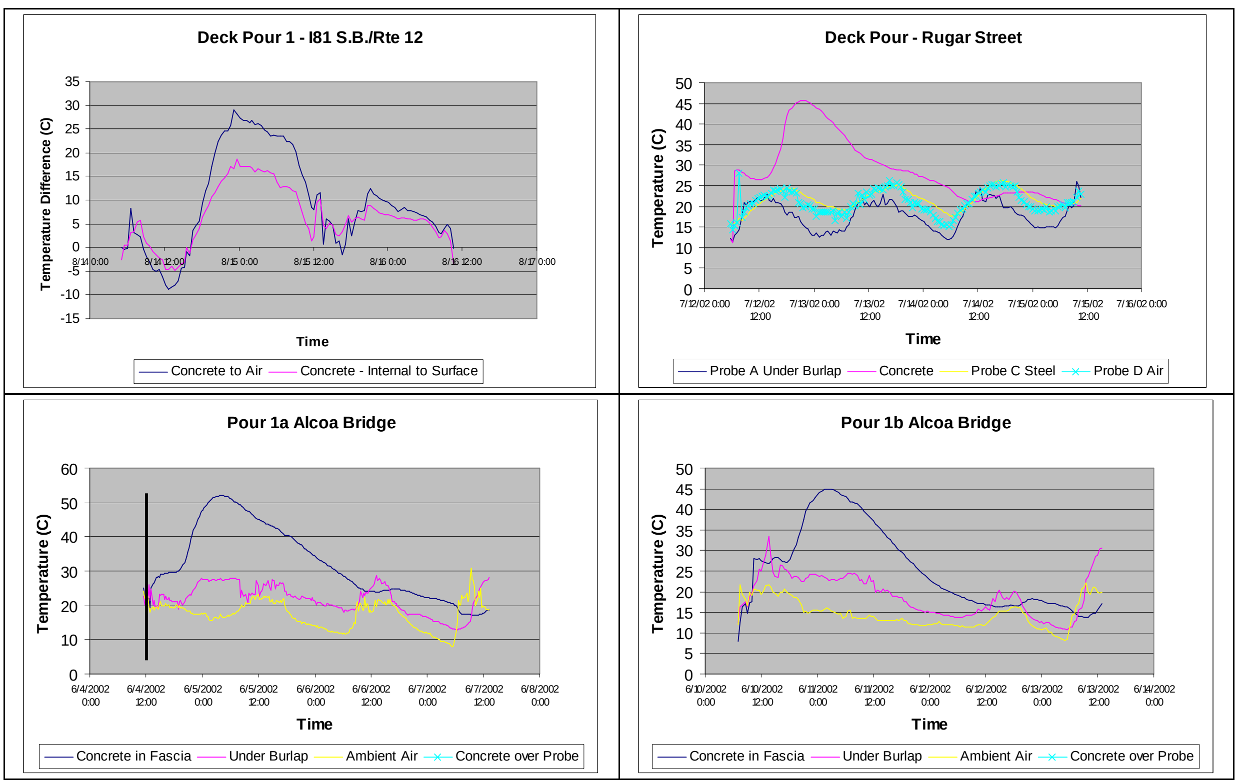
From a mechanics viewpoint, there are three causes of tension stresses in a bridge deck;

1) Thermal stress due to restraint of the deck while it cools.
2) Live load stresses on continuous bridges.
3) Stress due to concrete shrinkage while being restrained by the superstructure.

It will be shown that although any one of these sources of stress may not exceed the deck strength, the sum of the tensile stresses of each of these factors often exceeds the tensile strength of the deck.

5.1 Thermal Effects

The time at which the deck concrete gains its initial strength coincides with an increase in concrete temperature from the chemical reaction of hydration. Thus, the concrete sets at a temperature well above the temperature of the steel. As the concrete cools, the steel, which does not appreciably change in temperature, restrains the deck. Although this effect is difficult to quantify, it is clear that in decks where temperatures were measured in the hours after the deck pour, the temperature rise starts before the initial set of the concrete and the deck returns to ambient temperature occurs well after concrete set and at a point where the strength of the concrete has reached a value of more than half of the 28-day strength. Figure 7 shows the temperature rise relative to time for several bridge decks that were instrumented by NYSDOT.



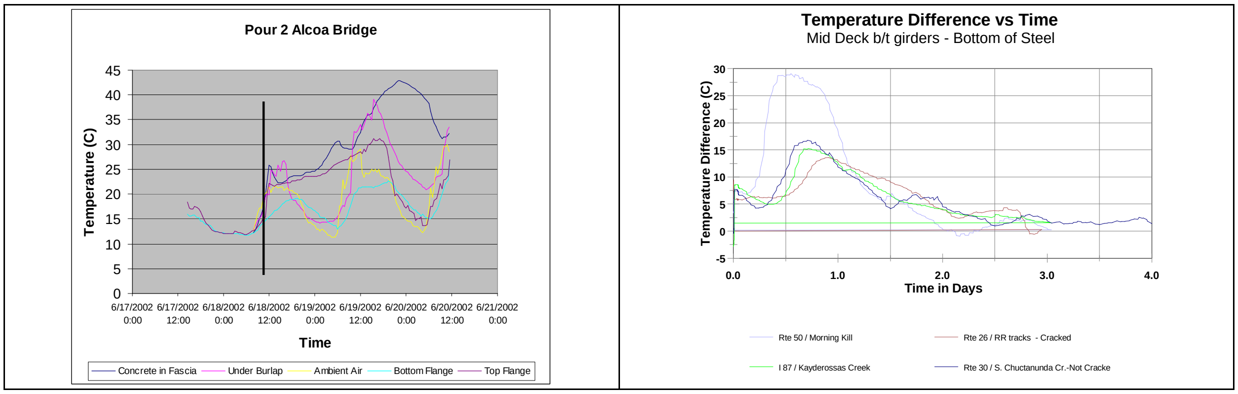


Figure7 - Initial Temperature Rise in Bridge Decks

Peak temperature differences of up to 54° F (30° C) are not uncommon. If the deck sets at a temperature of 120° F (50° C) and the superstructure steel top flange temperature is 85° F (30° C) at the time of set, there will be a built-in thermal stress corresponding to a 35° F (20° C) temperature difference.

If the concrete is free to expand and contract, the temperature difference would result in a strain in the concrete equal to:

$$\varepsilon_{Tu} = \alpha_{concrete} \times \Delta T$$

Where: ε_{Tu} = Thermal strain in the concrete if unrestrained
$\alpha_{concrete}$ = Coefficient of thermal expansion for concrete ($6.0\text{x}10^{-6}$/°F, $10.8\text{x}10^{-6}$/°C)
ΔT = Temperature difference

In reality, the top flange is not capable of fully restraining the concrete deck. As the concrete deck cools, the concrete elastically compresses the top flange until equilibrium is reached. The following expression can be used to determine the strains at equilibrium at the interface between the concrete deck and the top flange:

$$\varepsilon_{Tc} = \varepsilon_{Tu} - \varepsilon_{Ts}$$

Where: ε_{Tc} = Thermal strain in the concrete at equilibrium with steel
ε_{Tu} = Thermal strain in the concrete if unrestrained
ε_{Ts} = Steel strain due to concrete thermal force

It can be seen that elastic shortening of the steel top flange results in a reduction in the thermal stress. For example, if the top flange were infinitely strong, the concrete deck would be fully restrained and the thermal strain in the concrete would be zero. If the top flange were infinitely weak, the concrete deck would be unrestrained and the thermal strain in the concrete is at its maximum.

This equation can be solved to determine the equilibrium stress in the steel and the concrete due to an imposed temperature difference in the concrete. The parameters affecting the thermal stress in the deck will be the cross-sectional area of the deck in relation to the steel girder elastic properties, the coefficient of thermal expansion, the temperature of the concrete during strength gain, the modulus of elasticity of the steel and the concrete, and the temperature of the top flange of steel during concrete strength gain. The stiffer the top flange of the girder, the greater the degree of restraint and, thus, the greater the stress in the concrete. Figure 8 shows the effects that varying the top flange width, bottom flange width, and the web depth have on the resulting thermal stresses in the deck for a deck temperature rise of 54° F (30° C).

There is a linear relationship between the temperature difference at concrete set and the final concrete thermal stress. Likewise, there is a linear relationship between the coefficient of thermal expansion and the final thermal stress as shown in Figure 9.

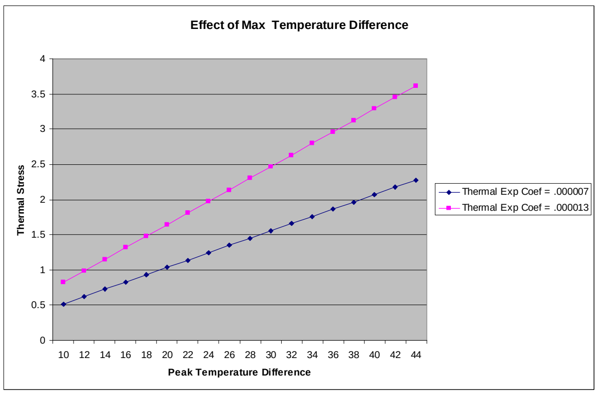


Figure 9 - Relationship of Concrete Thermal Stress and Peak Temperature Differences for range of thermal expansion coefficients reported by Emerson (1979).

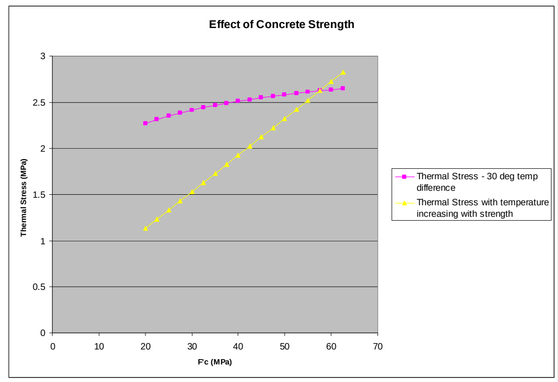


Figure 10 - Effect of Modulus of Elasticity as determined by Concrete Strength (f'_c) on the thermal stress for a 54° F (30° C) temperature rise due to the heat of hydration.

The modulus of elasticity is generally computed as a function of the strength of concrete. Therefore, the thermal stress can be computed as a function of the strength. In addition, higher strength concretes tend to have a higher heat of hydration. Figure 10 shows the relationship of strength to thermal stress for both a constant temperature difference and an increasing temperature difference.

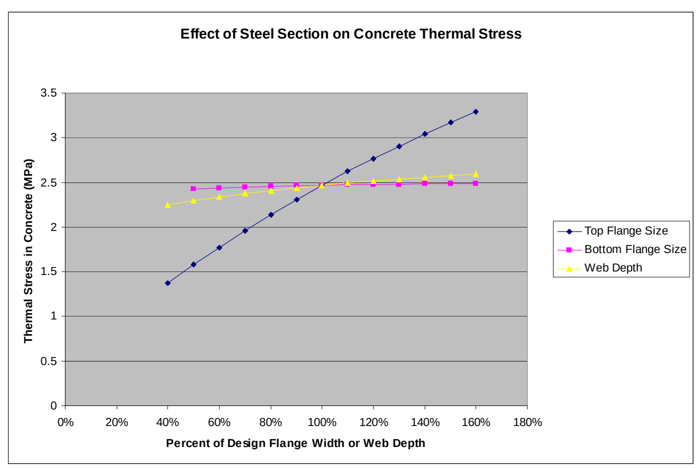


Figure 8 - Effect of Steel Section Properties on Thermal Stress.

In summary, the concrete stress due to the thermal effects of the heat of hydration can range from 75 psi (0.5 MPa) to 500 psi (3.5 MPa) with values of 285 psi (2.0 MPa) reasonable to expect.

A second source of thermal stress in the deck is due to the daily temperature cycling of the bridge superstructure. Temperature gradients develop between the top of the deck and the bottom flange of the composite steel beam, and even within the bridge deck itself. During the day, the exposed top of the concrete deck absorbs heat from the sun while the bottom of the deck and the steel beams remain in the shade. During the night hours, any exposed concrete surface radiates its heat back to the atmosphere while the core of the concrete deck, due to its large thermal mass, remains warm. The British Standard (BS 5400) gives negative thermal gradient design values of 6.25° F (3.5° C) to 9.0° F (5.0° C) for this type of radiation cooling.

5.2 Live Load Effects

Until recently, NYSDOT continuous span bridges were not designed to be composite in the negative moment region. Whether or not shear studs are provided in this region, these bridges will act as composite until deck cracking occurs. After cracking, the cracks will open to match the cumulative strain in the top flange due to live load and impact. When only live load stresses are considered, all of the deck for continuous bridges will be in tension. Based on typical distribution factors, these stresses are between 875 psi (6 MPa) and 1,750 psi (12 MPa) for the continuous span bridges examined in northern NY.

If the actual behavior of the bridge is considered, the maximum negative live load moment will occur at a pier where the live load is in the middle of the adjacent spans. With this loading condition there is a tendency for the entire superstructure cross-section to resist the negative live load moment, and the distribution factor can be approximated by the number of loaded lanes divided by the number of girders. Depending on the type of bridge, the number of lanes that are likely to be loaded simultaneously will also vary. On four (4) lane divided highways, it is not uncommon for one heavy truck to be passing another. It is reasonable that the negative moment corresponding to two trucks would occur relatively frequently. On very low volume local roads it is extremely unlikely that two trucks, each at the design capacity, would ever meet so as to cause maximum moment. With distribution factors based on the likelihood of multiple trucks, the maximum live load tensile stresses in the deck would vary from 150 to 450 psi (1 to 3 MPa). Figure 11 shows the live load stresses at the pier for bridges studied in northern NY.

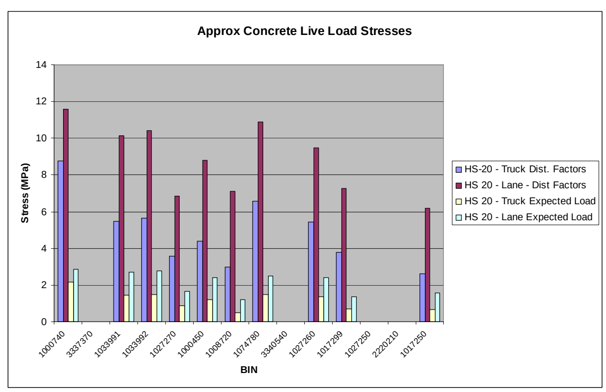


Figure 11 - Approximate Live Load Stress in the deck based on strain in the top flange of the steel at the pier.

5.3 Concrete Shrinkage

Another source of tensile stress is due to the tendency for autogenous shrinkage to occur in the deck. Autogenous shrinkage is due to the withdrawal of pore water in the concrete matrix to feed longer term chemical reaction demands of the cementatious materials. For a low permeability mix design as used for HP concrete, water demands cannot be met by the external environment and similarly, pore water is not replaced from external sources. As the pore water is depleted, capillary forces in the pores cause shrinkage to occur. Based on Section 5.4.2.3.3 of AASHTO LRFD Specifications, this shrinkage strain can be as high as 250 microstrain.

5.4 Combination of Tensile Stresses in Concrete Decks

A bridge deck can have tensile stresses as high as 225 to 300 psi (1.5 to 2 MPa) due to the combination of temperature difference, shrinkage, and live load. Not all of the parameters necessary to compute these stresses are readily available for the bridges in the Northern NY sample. However, reasonable assumptions can be made relative to these parameters and the thermal stresses in the deck can be calculated. The following table gives the assumptions used to calculate tensile stresses:

Parameter	Variables	Source of Data
Temperature difference at initial set of concrete	Steel Temperature and Concrete temperature at initial set.	Assumed to be difference between 122° F (50° C) and the temperature the day of pour with a maximum value of 86° F (30° C) and a minimum value of 59° F (15° C).
Thermal Expansion Coefficient of Concrete	Based on Aggregate type	Emerson Limestone 4.0 $x10^{-6}$/° F ($7.3 x10^{-6}$/° C) Granite 5.3 $x10^{-6}$/° F ($9.6 x10^{-6}$/° C) Dolomite is assumed same as Limestone.
Modulus of Elasticity of Concrete	$57{,}000(f'_c)^{1/2}$ $(4{,}730(f'_c)^{1/2})$	f'_c from DOT records of Pour
Deck Area	Contributory Area for each girder	Calculated from Record Plans
Steel Area	Area and Moment of Inertia	Calculated from Record Plans
Shrinkage	Course Aggregate Adsorption	$2.5 \times 10^{-4/}$ reduced by 1×10^{-5} for every tenth of 0.1% Course aggregate absorption over 0.5%.
Live Load Stress	Actual Truck Loading Bridge Properties	Distribution Factor is based on the number of load/number of girders. Number loaded lanes assumed to be 1.0 for low volume local roads, 1.5 for two-lane highways and 2.0 for interstate bridges.

Table 4 – Parameters that influence Tensile Stresses

Figure 12 shows the deck frequency rating as a function of the computed tensile stresses. These calculated values of the tensile stress show a reasonable correlation with the frequency of cracks. The tensile strength can be related to the 28-day compressive strength by $f_r = 7.5(f'_c)^{0.5}$ ($f_r = 0.62(f'_c)^{0.5}$). Figure 13 shows an even better correlation of data can be made by plotting the crack frequency against the ratio of the Tensile Stress to the Tensile Strength.

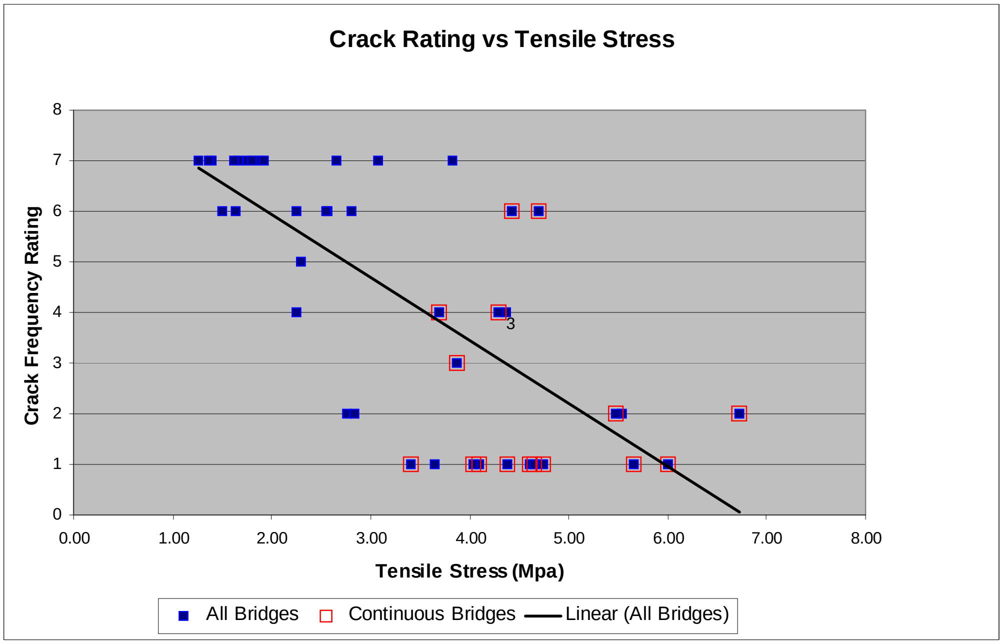


Figure 12 - Crack Frequency plotted as a function of Tensile Stress for northern NY Bridges.

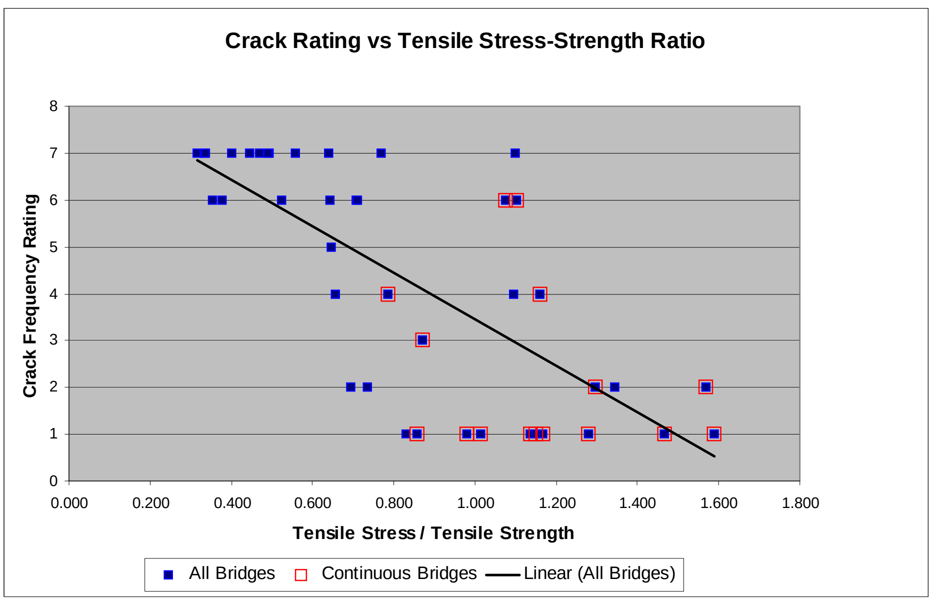


Figure 13 - Crack Frequency as a function of the ration of Tensile Stress to estimated Tensile Strength.

6.0 Effect on Bridge Deck Durability

To date, there has been no noticeable effect on the riding quality or structural condition of the deck due to bridge deck cracking. The rate of deck deterioration can be estimated by examining the average deck rating based on the year built in the bridge inventory. Figure 14 shows the average structural deck rating for all bridges based on the year built.

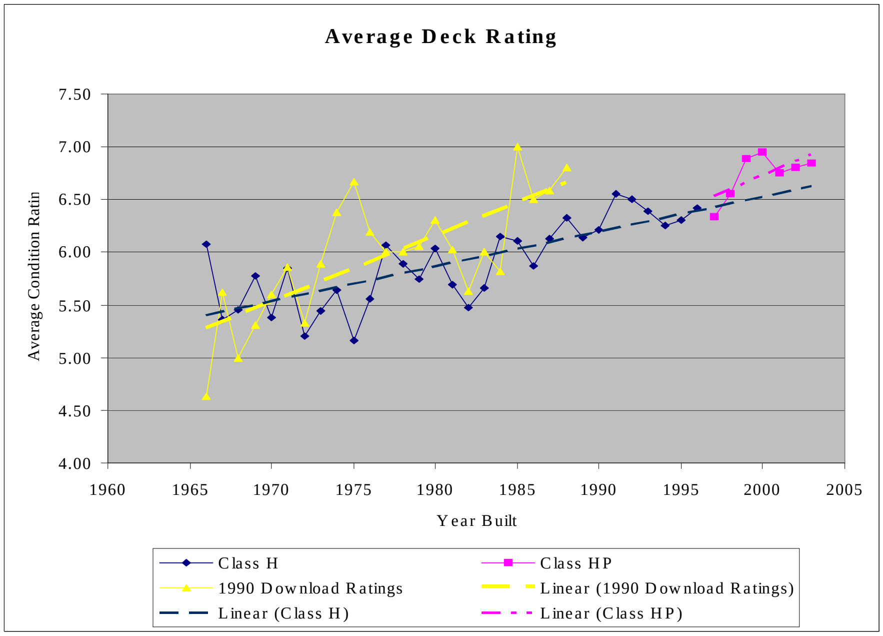


Figure 14 - Average Deck Rating based on Year Built

The average deck rating for decks built in 1970 that are still in service is still above 5. This may be, in part, due to a deck improvement program that was carried out through the 1980s. This program removed damaged concrete from decks originally built with plain reinforcement and 1.5 in. (38 mm) of cover and replaced it with 2.5 in. (65 mm) of new concrete intended for thin applications.

Also shown on the plot are the deck ratings as they appeared in 1990. It appears from these curves that the deterioration is not occurring any faster now than it did in 1990.

Figure 15 - Close up of one of many cracks in BIN 3221420 - Alcoa Road over Grasse River

The maximum crack width for concrete exposure to deicing chemicals recommended by ACI Committee 224[1] is 0.007 in. (18 mm). This width corresponds to a 5 rating on the

[1] ACI Committee 224, "Control of Cracking in Concrete Structures," Journal ACI, Vol. 69 No. 12, December 1972, pp. 717-753.

crack size rating scale. Of the decks that were cracked in the study sample, most had cracks larger than this threshold. Cracks of 0.013 in. (0.33 mm) were not uncommon and cracks as large as 0.030 in. (0.75 mm) were observed. Cracks of this size easily permit chlorides to reach the reinforcing steel (see Figure 15).

7.0 Case Studies

7.1 Galvanized Reinforcing Bars v. Epoxy Coated Reinforcing Bars

In 2002, the I-81 over NYS Route 12 (BIN 1009681) bridge deck was replaced utilizing galvanized reinforcing bars. The parallel bridge (BIN 1009682) was replaced in 2003 under the same contract. Surface spalling of BIN 1009681 appeared a short time after placement of the deck. This was attributed to the fact that the aluminum within the galvanizing reacted with the curing concrete and formed gas bubbles. These bubbles became trapped just below the surface because the finishing operation sealed the surface before the bubbles could escape. After the bridge was opened, the surface spalled at the weakness plane caused by these gas bubbles. Treating the galvanization with a chromate dip after galvanization will eliminate this problem and the specifications have been changed. This was unrelated to transverse cracking. Table 5 shows a comparison between the two decks.

	BIN 1009681	**BIN 1009682**
Reinforcing Bars	Galvanized Top and Bottom	Epoxy Top, Black Bottom
Construction Date	8/14/2002	7/25/2003
Temperature (Air/Concrete)	80/84° F (27/29° C)	64-80/73-82° F (18-27/23-28° C)
Concrete Strength	7,250 psi (50.2 MPa)	5,900 psi (40.5 MPa) (two sets taken, one had 6,500 psi (44.6 MPa) and one set 5,250 psi (36.3 MPa)
Cracking	Span 1 and 2 – 2-3 cracks per span Frequency Rating is 5. Span 3 – Approximately 7 cracks Frequency Rating is 2	None observed by 4/26/2004

Table 5 – Case Study Data

As can be seen from this data the bridge with galvanized reinforcing bars cracked but the bridge with epoxy and plain bars did not crack. However, considering the significant variation in concrete strength and the known problems that occurred due to reaction of the galvanization with the concrete, conclusions relative to the use of the galvanized bars cannot be made.

7.2 Fibrillated Polypropylene Fiber v. Plain Concrete

As part of the same project, another two bridges were used to compare the use of fibrillated polypropylene fibers in the mix to conventional design. As with the galvanized reinforcing experiment, the bridges were identical but they were constructed one year apart. The following shows a comparison between these two decks:

	BIN 1010081	**BIN 1010082**
Fiber Type	Fibrillated Polypropylene Fibers	No Fibers

Construction Date	8/27/2002	7/17/2003
Temperature (Air/Concrete)	72/70° F (22/21° C)	72/64° F (22/18° C)
Concrete Strength	28 day – 8,000 psi (54.9 MPa)	14 day - 5,400 psi (37.2 MPa) 28 day – 4,400 psi (30.4 MPa)
Cracking	Span 1 and 3 – Seven to ten cracks per span. Frequency Rating is 2. Span 3 has three cracks. Frequency rating is 5.	One longitudinal crack except for the closure pour over Pier 1 which has three transverse cracks. Frequency Rating is 6.

Table 6 – Deck Cracking With and Without Fibers

The bridge with the fibers cracked more. Due to the significant variation in concrete strength it is difficult to attribute the additional cracking to the presence of the fibers.

8.0 Recommended Strategies

The following is a listing of recommended strategies to minimize the effects of bridge deck cracking. The suggestions are based on a review of the available literature on this subject, data gathered from existing structures, and the experience of the panel members. The listing is not in any particular order.

8.1 Minimize Size of the Top Flange

As the freshly poured concrete cures, it generates heat. As the deck concrete begins to gain strength, it also begins to cool and shrink. The composite shear studs or composite shear bars prevent the deck from shrinking and causes tensile stresses along the deck. The greater the size of the top flange of the beam, the greater the amount of restraint.

Advantages:
- Smaller top flange reduces beam weight and cost.
- More efficient design for composite structure.

Disadvantages:
- Small top flanges provide limited lateral stability during beam erection and deck pour operations.
- Small top flanges may make the beam non-compact, requiring a more rigorous analysis.

8.2 Reduce Effective Strength of Deck Concrete

High strength concretes have a larger propensity to crack due to early strength gain, rigidity, and indirectly due to the greater heat of hydration of the higher strength concretes.

Advantages:
- Increased flexibility of concrete reduces likelihood of cracks developing.

Disadvantages:
- Difficult to reduce strength of concrete for a typical batch of concrete without increasing the number of batches that will be rejected due to insufficient strength.
- Expected resistance from concrete producers who may have increased production batches of concrete rejected due to insufficient strength.

- Environmental conditions at time of placement are not known. Cannot determine the strength of the concrete until after it has been installed and cured. Removing concrete that fails minimum strength requirements is expensive and time consuming.

8.3 Reduce the Temperature Rise of Concrete during Cure

As the freshly poured concrete cures, it generates heat. As the deck concrete begins to gain strength, it also begins to cool and shrink. Limiting the temperature difference between the concrete deck and the supporting beams will reduce the differential shrinkage and associated stress.

Advantages:
- Reduced tensile stresses in the concrete bridge deck.

Disadvantages:
- Expensive to construct large enclosures for cold weather concreting operations.
- Large amounts of heat required to overcome the thermal mass of the beams.

8.4 Shrinkage Reducing Agents

Shrinkage reducing agents are actually additives that will cause the concrete to expand approximately as much as the shrinkage that would be expected through reduction in free water content of the matrix due to the hydration of the cement.

Advantages:
- May reduce the shrinkage.

Disadvantages:
- Since shrinkage is only one source of the tensile stress in the deck, Shrinkage Reducing Agents may help but will not eliminate cracking of bridge decks.

8.5 Light Weight Aggregate/Internal Curing

High Performance Concrete is designed to use as little water as is necessary for low permeability and optimum strength. There may not be enough water on the inside of the concrete to completely react with the cement. Adding water to the deck surface in the form of continuous wetting will not provide the required moisture to the interior of the concrete pour. Adding small quantities of saturated lightweight aggregate provides free water within the concrete matrix to meet the long term demands of the cement for water. The concept is that the lightweight aggregate would serve as a sponge and store water that would replace capillary water as the hydration continues with time. When light weight aggregates are used for this purpose, the process is referred to as internal curing.

Advantages:
- It is a relatively simple way to add free water to the mix.
- It is not a proprietary product.

Disadvantages:
- Special hoppers are needed at the batch plants for the additional aggregate type.

- To be effective, the aggregate must be 100% saturated. Any amount less than 100% saturation will actually add to the shrinkage as both the aggregate and the cement compete for the capillary water in the concrete.
- Additional cost over traditional concrete.

8.6 Corrosion Inhibitors

Several concrete additives have been proposed by various suppliers that purportedly reduce the potential for corrosion. A study completed by University of Virginia in 2003 did not show any advantage of 5 corrosion inhibitor products[2]. It is difficult to assess the success of such a product because the time frames are so long. By comparison, many of the decks examined in northern NY exhibited cracking and had been in place for more than 15 years, yet there were very few instances where the cracking led to serious corrosion that would have been visible from the surface or underside.

Advantages:

- Easy to implement.

Disadvantages:

- Relatively high cost.
- Additives may have a detrimental effect on the strength or durability of the concrete.

8.7 Fibers in Concrete

Residential and commercial construction use steel and synthetic fibers in concrete to improve durability.

Advantages:

- Reduced plastic shrinkage and subsidence cracking.
- Increased toughness or post-crack integrity.
- In fresh concrete, polypropylene fibers also reduce the settlement of aggregate particles from the pavement surface, resulting in a less permeable and more durable, skid resistant pavement.

Disadvantages:

- Additional cost, although minimal.
- Does not compensate for overstrong or overworked concrete.

8.8 Two Course Decks

Two course decks have a thinner structural deck overlaid with a concrete or, more typically, asphalt wearing surface. Two course bridge decks were typical on NYSDOT bridges through the 1960s.

Advantages:

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[2] Sprinkel, Michael M., "Evaluation of Corrosion Inhibitors for Concrete Bridge Deck Patches and Overlays", Virginia Transportation Research Council (http://www.virginiadot.org/vtrc/main/online_reports/pdf/03-r14.pdf)

- Deck cracks are not visible and waterproof membranes provide a measure of defense against chloride attack of the reinforcing steel.
- The deck can be repeatedly sealed with an impermeable membrane between the concrete deck and the asphalt overlay. Alternatively, polymers can be added to the asphalt to reduce its permeability.

Disadvantages:

- Initial construction is more costly due to additional material and the need for separate placements.
- Generally, the wearing surface and waterproof membrane has a life of 12-15 years, requiring replacement several times during the life of a bridge.
- Joints have been a problem in the past. Rigid headers are usually necessary to support joint systems. Impact points between the asphalt and the concrete header have different wear characteristics. New joint systems and materials may lessen this disadvantage.
- Deck cracks are not visible and accurate evaluation is difficult.

8.9 Saw Cutting

Saw cutting may be a mechanism to force cracks at locations and frequencies where they can be sealed. This is similar to the procedure used for decades on concrete pavement. With this technique, it is imperative that the sawing be completed at a time before cracks have had an opportunity to form, typically within 12-36 hours of placing of the concrete. Subsequently, the cracks are typically re-sawn and an elastomeric strip seal is installed to effectively seal the joint. This technique relieves the tension stresses by allowing shrinkage of individual slabs.

Advantages:

- Cracks can be forced to occur in locations where they can be accommodated.
- Cracks can be sealed to keep out moisture.

Disadvantages:

- This technique effectively creates many bridge joints. For years, bridge joints have been a problem where movement must be accommodated but water must be kept out.
- Sawing and sealing decks would add cost to the deck and add an additional maintenance item.

8.10 Increasing the Amount of Longitudinal Reinforcing

Increasing the amount of longitudinal reinforcing steel does not prevent, but rather reduces the size of cracks. The LRFD specification[3] requires that the area of longitudinal reinforcement equals or exceeds 1% of the deck cross section anywhere the forces in the deck (including live loads as well as shrinkage and thermal loads) exceed the cracking strength of the concrete. A discussion of this is included in Appendix B.

Advantages:

- There would be greater control of the cracking.

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[3] Section 6.10.1.7 of AASHTO LRFD Bridge Design Specifications

- More reinforcing should limit the crack size. In some cases, the crack size would be limited to the size that would prevent intrusion of chlorides.
- If steel area is considered as part of the section in composite design of the girder, the size of the top flange may be reduced thereby reducing the restraint for thermal stresses.

Disadvantages:

- There could be a slight increase in cost. The cost of the additional rebar is estimated to add approximately \$1.00-1.50/ft^2 (10-\$15/m^2) of deck.
- Using 1% of the deck cross-sectional area will not prevent deck cracking nor will it prevent chloride attack. To limit crack size to acceptable levels, the steel ratio must be increased to 2%-3.5% (see Appendix B) which, if physically possible, would add \$4-\$6 per square foot of deck (\$40-60 per square meter).

8.11 Decrease Concrete Cover

Greater concrete cover has been shown to increase the number and size of cracks. It makes little sense to design a dense concrete mix with large cover if the concrete cracks and provides a direct pathway to the reinforcing steel within. Decreasing the concrete cover from 3 in. (75 mm) to 2 in. (50 mm) would result in a 13% decrease in crack width. Reduction to a 1½ in. (40 mm) cover would result in a 19% reduction in crack width.

Advantages:

- Fewer, tighter cracks.
- Direct material cost savings for less deck concrete, and residual savings due to reduced dead load such as smaller superstructure beams, reduced substructure dimension, and fewer piles.

Disadvantages:

- None.

8.12 Use of Galvanized Reinforcing Bars

Galvanized reinforcing bars have been used for several years by many transportation agencies. The galvanization serves as protection against corrosion so cracking will have minimal effect on deterioration of the deck. Some consider that the concrete to rebar bond will be better than epoxy rebar and this will lead to reduced cracking.

Advantages:

- As long as the galvanizing remains intact, the reinforcing bars will not corrode. Without corrosion, there will not be deterioration of the deck associated with chlorides.
- The galvanized bars have greater bond strength and, therefore, less lap length than epoxy bars.
- Cost is comparable to epoxy coated reinforcing bars and only 10% more than plain bars[4].

Disadvantages:

[4] Table 15-2 NYSDOT Bridge Manual

- Without proper post-galvanizing treatment, the galvanizing may chemically react with the cement to cause gas bubbles which have a negative effect on concrete quality.
- Areas where the galvanizing is damaged or where there is incomplete coverage may actually accelerate corrosion.
- The zinc layer of the galvanizing has been shown to disintegrate with time when embedded in concrete as the zinc serves as a sacrificial anode.

8.13 Use Solid Stainless Steel (SSS) Reinforcing Bars

SSS will not prevent cracking of the concrete deck, but its resistance to corrosion negates the negative effects of the cracks. Due to the initial expense, use of SSS has been limited to structures with significant user-costs associated with traffic restrictions due to repair and maintenance.

Advantages:
- Concrete decks will have a longer life span, even if cracked.
- There is no learning curve for designers: Designing with SSS is the same as for black reinforcing bar.
- Required concrete cover is reduced. Direct material cost savings for less deck concrete, and residual savings due to reduced dead load such as smaller superstructure beams, reduced substructure dimension, and fewer piles.
- Life cycle cost may be less than for other types of reinforcing bars.

Disadvantages:
- Stainless steel is initially more costly than plain or epoxy bars. SSS bars are 200% the cost of plain bars[5].
- With increased use, availability could be a problem.
- Cracks will still exist and may present an aesthetic problem.
- The Coefficient of Thermal Expansion for stainless steel is 40% to 50% greater than carbon steel making it less compatible with concrete. In some cases this might be an advantage.

9.0 Methods of Treating Bridge Deck Cracks

Technical representatives from five manufacturers of ultra-low viscosity concrete restoration products were invited to place their material into existing cracks on the I-890 - Rte 5 Connector over the Mohawk River (BIN 4437290). Six products were placed on several of the cracks on October 30, 2001; three High Molecular Weight Methyl Methacrylates (HMWM), two Epoxies and one Methyl Methacrylate (MMA).

Application of the restoration products was mostly done by pouring the material on the crack and retaining it with a paint brush or roller. As the demonstration was conducted with temperatures near the lower limit, most of the products took considerable time to cure. One product had the ability to adjust the cure rate. Acrylic products are sensitive to

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[5] Table 15-2 NYSDOT Bridge Manual

moisture. The epoxy based products claim to be less moisture sensitive than the other products tested. This is a favorable attribute but was not tested in this demonstration.

A characteristic of HMWMs and MWMs is a strong pungent odor. Though not hazardous, the materials can be detected at low levels and the workers question the health effects. Results for methyl methacrylate and mineral spirits monitoring indicate workers were exposed to levels well below the Occupational Safety and Health Administration (OSHA) and Public Employee Safety and Health (PESH) limits.

Four inch diameter cores approximately 4 inches deep were taken on April 30th, 2002. Cores were removed intact and taken to the Materials Bureau lab for preparation. The samples were cut perpendicular to the crack and the inner surfaces polished. The products did display variability in penetrating the crack. The HMWMs and the MMAs reached greater depths than the epoxy products.

Since the formal demonstration, bridge maintenance crews have placed “healer/sealers” on approximately 100,000 sq. ft. of bridge deck using a topical application and numerous linear feet using a “ketchup bottle” to place material into the crack. HMWMs were typically used though MMAs, epoxies, and polyurethanes were also used. HMWMs are reported to be better able to solvate interstitial contaminates than other low-viscosity polymers and should provide a better watertight seal.

The performance of these materials on aged, distressed concrete is currently being questioned. The inability to properly prepare and dry the interstitial surfaces seems to negatively affect the ability of the material to form an adequate bond. Further evaluation is necessary to determine if these materials are suitable in the repair of recently placed concrete decks experiencing excessive transverse cracking.

Appendix A
Bridge Deck Task Force Members

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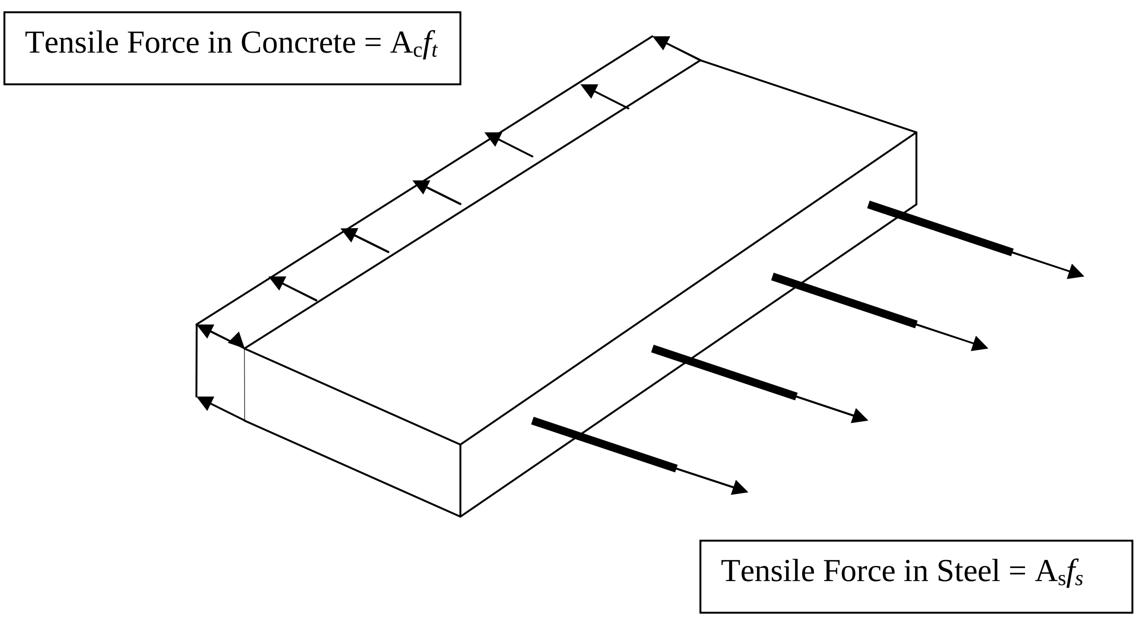
Appendix B
Tensile Cracking and Longitudinal Cracking

Concrete cracks when the tensile forces exceed the capacity of the concrete to carry them. Once cracked, the embedded reinforcing bars carry all the tensile stresses. If the tensile forces are known, adequate reinforcing steel can be provided to resist the applied forces.

Longitudinal reinforcing is designed to distribute applied wheel loads along the deck to the transverse steel spanning between beams (i.e., 2-way slab design). LRFD 6.10.1.7 requires that the area of longitudinal reinforcement equals or exceeds 1% of the deck cross section wherever the tensile stress exceeds Φf_r. Φ is typically taken as the resistance factor for concrete in tension and f_r is taken as the modulus of rupture of the concrete. In calculating the tensile stress, live loads as well as shrinkage and thermal loads shall be considered.

At the instant when the deck cracks, all of the force that was carried by the deck in tension is transferred to the reinforcing steel. This can be shown by a free body diagram as follows:



If the tensile strength of the concrete is assumed to be:

$$f_t = 0.23\sqrt{f'_c} \text{ (ksi)}$$

$$f_t = 0.62\sqrt{f'_c} \text{ (MPa)}$$

then at cracking, the steel stress is:

$$f_s = 0.23\frac{\sqrt{f'_c}}{\rho} \text{ (ksi)}$$

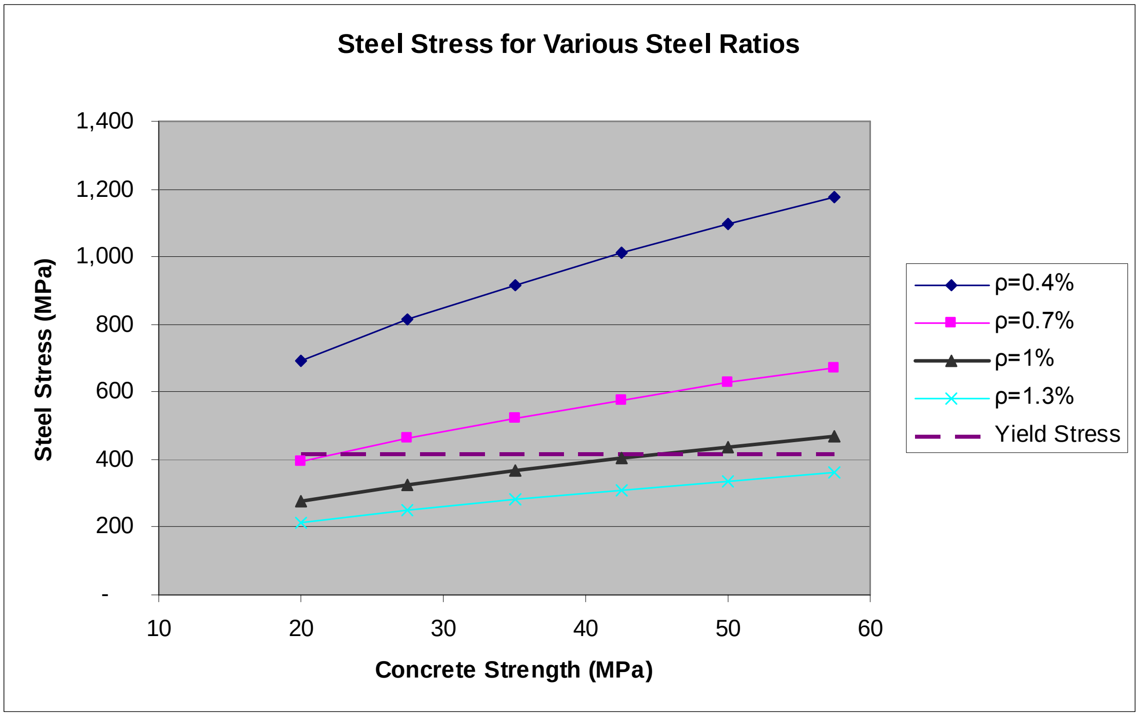
$$f_s = 0.62\frac{\sqrt{f'_c}}{\rho} \text{ (MPa)}$$

Where:

$$\rho = \left(\frac{A_{steel}}{A_{concrete}} \right)$$

Once cracking occurs, the opening of the crack increases the strain in the reinforcing steel as the bars are stretched across the gap, but reduces the built up thermal and shrinkage stresses in the concrete by allowing the concrete sections to shorten.

The following shows the stress in the reinforcing steel at the instant of concrete cracking for various concrete to steel ratios and concrete strengths.



For concrete strengths up to approximately 6.5 ksi (44 MPa), the 1% value of the steel ratio will ensure that the steel does not yield. Obviously, stresses higher than yield cannot occur. However, when yield is reached the crack size could be expected to increase dramatically as plastic deformations occur. Typically, NYSDOT bridge decks have steel reinforcing ratios of 0.35% to 0.45% at midspan.

ACI Committee 224 publishes recommended crack widths for various exposures. These range from 0.016 in. (0.41 mm) for dry air or structures with protective membranes to 0.004 in. (0.10 mm) for water-retaining structures. A maximum crack width of 0.006 in. (0.15 mm) is recommended for seawater or seawater spray, where the concrete is subjected to frequent wetting and drying. Gergely and Lunz have shown a relationship for the crack width as follows:

$$w_{\max} = 0.076 \sqrt[3]{t_b A} \frac{h_2}{h_1} f_s \times 10^{-6} \quad \text{(Formula is in English units)}$$

Where:

$t_b =$ Distance from extreme fiber to center of adjacent bar

$A =$ Effective area of concrete in tension around each bar

$f_s =$ Steel reinforcing bar stress

h_1 *and* $h_2 =$ Distance between the extreme fiber and the neutral axis and the distance between the centroid of the tension steel to the neutral axis, respectively.

Applying this equation together with the assumption that, at the moment of concrete cracking, all the tensile load in the deck is transferred to the reinforcing steel gives the following:

	f_s	18 ksi	48 ksi	60 ksi
	w_{max}	0.006 in.	0.016 in.	0.006 in.
f'_c	f_t	Steel Ratio required (A_s/A_c)		
3 ksi	400 psi	2.28%	0.86%	0.68%
4 ksi	460 psi	2.64%	0.99%	0.79%
5 ksi	515 psi	2.95%	1.11%	0.88%
6 ksi	560 psi	3.23%	1.21%	0.97%
7 ksi	610 psi	3.49%	1.31%	1.05%
8 ksi	650 psi	3.73%	1.40%	1.12%

As the cracks open, some of the thermal and the shrinkage stresses are relieved and, therefore, actual crack size may be somewhat less than predicted. Note that the 1% requirement of the AASHTO LRFD Specifications corresponds to the yield strength of the reinforcing steel for concrete strengths less than 6,000 psi (42 MPa). A steel ratio of 2-3.5% is required to limit the crack size widths to be acceptable for saltwater exposure.