To the Graduate Council:

I am submitting herewith a thesis written by Gordon James McClellan entitled "Deterioration of Concrete Bridge Decks." I recommend that it be accepted in partial fulfillment of the requirements for the degree of Master of Science, with a major in Civil Engineering.

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Vice Chancellor
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DETERIORATION OF CONCRETE BRIDGE DECKS

A Thesis

Presented for the

Master of Science

Degree

The University of Tennessee

Gordon J. McClellan

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INTRODUCTION

1. The Problem

During the past several years bridge deck deterioration has become a severe problem that is encompasing much time and money. New bridges are being closed because of deck failures and for repairs, and many bridges are now receiving overlays on their decks. For these reasons a more thorough examination of bridge deck deterioration is needed.

The deterioration of concrete bridge decks involves many variables. Instead of looking at the problem from a laboratory view with controlled conditions, this study was performed on actual highway bridges in use. Because of this approach many variables were not considered. This study is primarily concerned with transverse cracking with an evaluation of the deterioration related to the type of traffic during the study period. Aspects of other forms of deterioration were also studied. While much has been written on the effects of deicing agents and the freeze-thaw process, little has been written of the effects of the type of traffic during the early life of a bridge. It is hoped that this study will help to fill this gap.

2. Objectives

It was the purpose of this study to measure the amount of deterioration of the bridge decks under study. This was done primarily by measuring the severity of the transverse cracking. Other forms of deterioration were observed such as the occurrence of popouts. Measurements of the natural frequency of vibration of the bridges were made during the study.

It was thought that the amount of clear cover could influence the amount of transverse cracking. To investigate this theory several bridges were measured for clear cover in each pour, and this depth was compared with the intensity of cracking for each pour.

This study also attempted to determine if there was a relationship between transverse cracking and the type of traffic that used the bridge in its early life.

An attempt was made to correlate the transverse cracking with one of the dynamic characteristics of the bridges. The natural frequency of vibration was chosen as the dynamic characteristic to be investigated. The natural frequencies were compared to the highest cracking intensity for each bridge.

3. Scope

Because so many variables are involved in bridge deck deterioration it was decided to limit this study mainly to

transverse cracking and longitudinal cracking. Other forms of deterioration such as popouts, spalling, and scaling were noted where found but major emphasis was placed on the cracking. It was hoped to evaluate the effects of normal traffic and construction traffic on the cracking of the concrete bridge decks.

The bridges selected for this study were divided into two categories: bridges subjected to construction traffic and bridges subjected to normal traffic. All the bridges studied were constructed recently. This was necessary in order to study the bridges under construction traffic. The bridges subjected to construction traffic were inspected early in the construction period and again before being opened to normal traffic. The bridges subjected to normal traffic were inspected before or right after opening to traffic and approximately nine months later. All but two of the bridges studied were steel bridges with continuous spans.

Another area given specific attention was the amount of clear cover over the reinforcing steel. In order to determine if the amount of clear cover would influence the amount of transverse cracking, clear cover was measured on several bridges in each pour and this was compared with the cracking intensity in each pour.

An attempt was made to correlate the transverse cracking with one of the dynamic characteristics of the

bridges. The natural frequency of vibration was chosen as the dynamic characteristic to be investigated. The natural frequency of vibration was compared to the highest cracking intensity for each bridge. No attempt was made to measure dynamic stresses in the slabs or the deflections of the bridges investigated.

I. RELATED RESEARCH

A survey[1*] was conducted in 1955 by the Highway Research Board to determine the principal problems confronting bridge maintenance engineers. In this survey, concrete deterioration was rated fourth in the frequency of appearance. A similar survey was made in 1967 and the following question was asked: "What type of structural maintenance requires the greatest effort?"[1] Concrete bridge decks was the answer most often given. In 12 years, the deterioration of concrete bridge decks had gone from fourth to first in the area of structural maintenance. Why has this happened and why was the problem not apparent before?

During the 1950's many states instituted a bare pavement policy in times of bad weather. To keep the pavement free of snow and ice, deicing chemicals, notably sodium chloride, was used to melt the snow and ice. The resulting water and salt solution then penetrated the transverse cracks on the bridge decks and reached the transverse reinforcement. The brine solution accelerated the corrosion of the steel.[2] It has been found that spalls occur in areas in which the transverse cracks are directly over the top transverse steel.[2] Concrete bridge decks have

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*Superscripts refer to the entries in the List of References.

always cracked but the increased use of salt solutions has increased the deterioration.

Transverse cracking is thought to be the forerunner of many problems associated with bridge decks.[3] It has been found that transverse cracks are associated with the top transverse reinforcement.[4] On bridges where the transverse reinforcement is skewed, the transverse cracks are skewed at the same angle.[4,5] In the case of bent bars, the transverse cracks occurred over the bars when the bars were at the top of the slab and wandered away as random cracks where the bars were bent down.[4] A study in Missouri[6] revealed that transverse cracks occurred directly over the transverse reinforcement or were slightly offset from the reinforcement. This was also found to be true in Michigan[5] and California.[4] It was also found by Stark[2] that crack patterns consisted of cracks over the top reinforcing steel for clear cover of less than two inches. In areas of two inches or more of clear cover, the predominate form of cracking was random or pattern cracking.

Callahan, Siess, and Kesler[7] considered that transverse cracking was caused by one or more of the following:

1. Plastic and/or drying shrinkage of the concrete
2. Poor consolidation of concrete
3. Improper placement of steel
4. Excessive drying by heaters during winter concreting

5. Long time volume changes, including temperature effects
6. Resistance to subsidence of concrete during the bleeding period by the rigidly held top reinforcement
7. Vehicle loading
8. Stresses resulting from the removal of falsework from continuous reinforced concrete bridges.

The direction of critical stress induced by traffic is in the transverse direction and would tend to produce cracks in the longitudinal direction. Stresses in the transverse direction would be secondary and transverse cracks are probably caused by volume changes in the concrete, structural displacements, or consolidation of the plastic concrete.[7] In a test in which galvanized iron pipe with a slit in the top side covered with a rubber membrane replaced the steel reinforcement, it was found that vertical cracks over the reinforcement could be easily produced for covers of less than 1.5 inches by the application of a hydraulic pressure to the pipes.[7] Once these vertical cracks are formed, they are opened and closed by the action of the traffic.

A study in Kansas[8] indicated that older bridges had a greater amount of cracking than newer bridges. Several factors were thought to influence the transverse cracking:

1. Number of load repetitions

2. Greater probability of larger maximum live load with increase in age
3. Longtime volume changes
4. Number of cycles of freeze-thaw
5. Corrosion of steel.

The transverse cracks in the positive moment regions were thought to be caused by live load tensile stresses as a result of vibration and also the loading of an adjacent span may increase the width of the cracks since there is very little steel resisting the tensile stresses.[8]

Cracks in the concrete deck are not bad in themselves except that they offer an easy passage for the water-salt solution to follow in reaching the reinforcing steel. Once the water is at the level of the steel, it may corrode the steel and force the concrete away from the steel, or it may freeze and cause internal pressures which can cause a pop-out over the reinforcing steel.[3] Severe corrosion results in the formation of additional cracks parrallel to the reinforcement. These cracks are caused by internal pressure caused by the rust which occupies a greater volume than the original steel.[9]

Core samples have shown that some transverse cracks form before the concrete has hardened. This is shown by the crack going around the aggregate instead of through it.[4,5,6,8] Other cores have shown that cracks passed through the aggregate.[4,5,6,8] This suggests that the crack formed after the concrete hardened.

Spalling is generally recognized as the most troublesome defect because the deck is weakened locally, the reinforcing steel is exposed, the riding quality is impaired, and repair work is difficult.[1] There are indications that spalls may be caused by resistance to subsidence during the bleeding period and/or by early drying shrinkage.[8] In many instances, spalling appears to be associated with vertical cracks.[6,8] It is also thought that the crack pattern of a bridge deck influences the development of spalls. Measurements of clear cover indicate that spalls were associated with reinforcing steel that had less than 2.0 inches of clear cover.[2,10] Usually the cover was less than 1.5 inches. Numerous unspalled areas with 2.0 or more inches of cover were found next to spalled areas with cover less than 1.5 inches.[2] It has been found that there is a greater tendency for severe spalling on the outside lanes of four lane bridges.[5] It is thought that vehicular traffic has a great effect in propagating these spalls.[5]

Scaling is the least destructive type of deterioration. It is formed by the flaking or peeling away of the surface mortar in the concrete.[1] It has been concluded that scaling can be attributed to one or more of the following items:[7]

1. Excessive and/or late finishing
2. General hand-finishing practices
3. Batch-to-batch differences in air content and/or slump of concrete

4. "Washing out" of air due to excessive bleeding and/or additions of water during finishing and placement
5. Insufficient dispersion of entrained air during mixing
6. Poor drainage of bridge decks
7. Excessive use of deicers.

Callahan states that stress will not cause scaling nor will it prevent scaling.[11] Surfaces under static tensile stress scaled at a greater rate than non-stressed surfaces, and surfaces under static compressive stress scaled at a lesser rate, however cyclic tensile stresses had a negligible effect on the rate of scaling while cyclic compressive stresses scaled faster.[11]

Some investigators feel that construction practices have a great effect on the concrete bridge decks. Cady and Theisen state that sprinkling and reworking or overworking deck surfaces, high winds and high ambient temperatures, high slumps, low air contents, shallow placement or inadequate support of reinforcing steel, and improper or late application of curing procedures are detrimental to durability.[12] They also feel that high winds and temperatures and delays in applying curing procedures are associated with cracking.[12]

The masses of reinforcing steel invite the use of high slump concrete for workability. The high water-cement

ratio increases the permeability and even if there are no cracks, water as well as salt can reach the reinforcing steel in this porous concrete.[1] Figure 1 shows the relationship between permeability and the water-cement ratio. Two inches of permeable concrete may give less protection to the reinforcing steel than 1/4 inch of impermeable concrete.[9] Kinneman observed that 1/2 inch of high quality concrete will protect reinforcing steel from salt water corrosion.[9] Mozer, Bianchini, and Kesler state "The permeability of concrete is probably the most important single factor affecting the corrosion of steel."[9]

The rapid evaporation of water from the surface of fresh concrete is considered to be the most important factor influencing plastic shrinkage and plastic shrinkage cracks.[13] Concrete has a tendency to dry rapidly when placed under the following conditions:[13]

1. Low humidity
2. Low dew point
3. In windy weather
4. In hot weather
5. When concrete is heated
6. When concrete is placed in heated enclosures.

If the evaporation rate of the water is much greater than 0.1 pound of water per square foot per hour, cracking will almost certainly occur unless precautions are taken.[1] Receding moisture in the capillaries of the fresh concrete develop capillary forces which compact the surface.[13] This

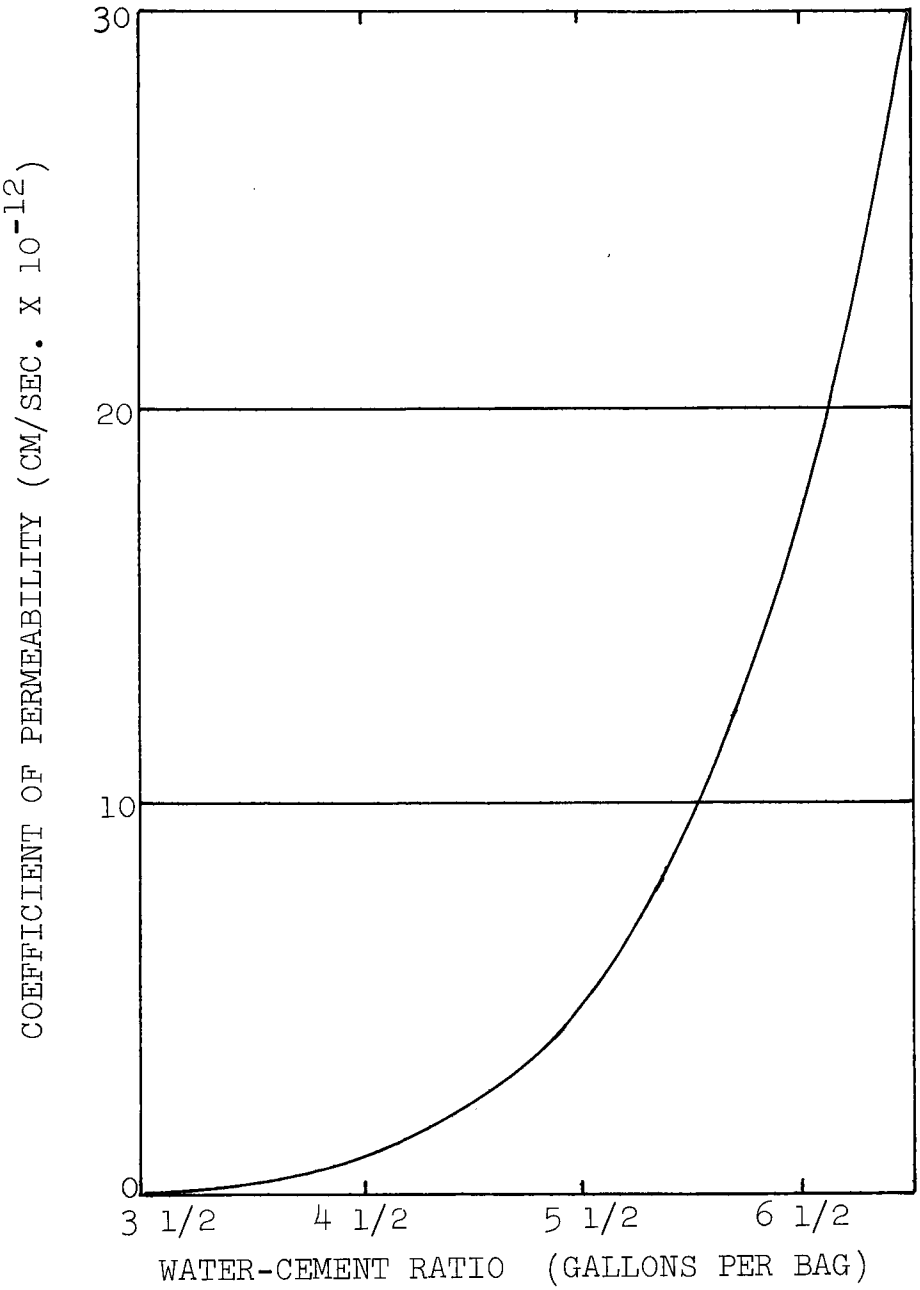


Figure 1. Relationship of Permeability of Cement Paste to Water-Cement Ratio. (Source: Portland Cement Association)

produces a layered system with a greater density at the surface than immediately below the surface, and this may contribute to scaling tendencies under freeze-thaw action.[13]

When air-entrainment agents are added to the concrete mix, the resistance of the concrete to freezing and thawing is increased.[4,5,6,8,14] This is true even in the presence of deicers. Linseed oil has been found to protect concrete but not as well as air entrainment.[14] Air entrainment provides voids for the redistribution of moisture when the concrete is undergoing freeze-thaw action. When a saturated cavity is frozen, either the volume must expand by 9 per cent or the excess water is forced out.[1] Entrained air pockets provide space for the water to flow into. For this reason, air-entrained concrete is protected from scaling and non-entrained concrete is susceptible to scaling.[1] To achieve maximum benefit from air-entrainment, the air voids should be uniformily distributed.

It is becoming evident that the amount of concrete cover has an influence on the deterioration of concrete bridge decks. Cores taken in Missouri[6] revealed that the average cover over reinforcing steel in non-spalled areas was 1 3/4 inches and in spalled areas, the cover was 1 1/4 inches. This study also stated that inadequate cover increases the possibility of shrinkage cracks forming in the thin layer of concrete directly over the reinforcing

steel.[6] In California, cover measurements were noted as low as 1/2 inch over reinforcing bars in spalled areas.[4] In Michigan, instances of cover over a given bar varying more than the cover specified were found.[5] Cover over bars related with transverse cracks varied from 7/16 inch to 1 1/2 inches.[5] As stated earlier by Stark, for cover of less than 2.0 inches, the cracking is primarily transverse, and for cover of over 2.0 inches, the cracking is primarily random or pattern. It is thought by some that the uniformity of cover over the reinforcement plays an important role in protection against corrosion.[9] It has proven to be more decisive than the quality of concrete in many cases.

Most organizations now recommend a minimum of two inches of clear cover over the topmost steel.[1,2,4,5,6,8] The Portland Cement Association also recommends placing lighter (No. 3 bars) and more widely spaced (9 inches) longitudinal steel on top of the transverse reinforcement.[1] This would provide reinforcement to help close the transverse cracks. In order to assure a minimum clear cover of 2.0 inches, the clear cover may have to be specified as somewhat greater than 2.0 inches. One researcher found that it was necessary to specify a clear cover of 2 3/8 inches.[1]

II. METHODS USED IN STUDY

1. Division of Bridges

The bridges examined in this study were divided into two categories: bridges subjected to construction traffic and bridges subjected to normal traffic during the study period. This was done to try to determine if the type of traffic on the bridges had any effect on the cracking of the concrete bridge decks. A brief description of the bridges in each category follows.

2. Bridges Subjected to Construction Traffic

H-E and H-W

These bridges are part of Interstate 40 and carry Interstate 40 over Tennessee State Route 61 in Roane County near Harriman. Each bridge consists of seven continuous spans (55 ft., 72 ft., 110 ft., 115 ft., 110 ft., 72 ft., 60 ft.). The bridges are skewed 44° 40'. The width of the roadway on H-E is 42 ft. and 3 in. and the width of the roadway of H-W is 31 ft. and 3 in. Both bridges consist of plate girders with a composite deck having a thickness of 7 1/2 in. The bridges are located on a portion of the interstate that was under construction during this study.

M-E and M-W

These bridges are located in Memphis at the interchange

of I-40 and I-255. Both bridges are ramps over Bellevue Boulevard. M-E channels traffic from I-255 to I-40 eastbound. M-W routes traffic from I-40 westbound to I-255 northbound. Both bridges are curved simple spans, and M-E has a span of 79 ft. while M-W has a span of 102 ft. M-E has a roadway width of 31 ft. and 2 in. and a deck thickness of 7 in. M-W has a roadway width of 22 ft. and 2 in. and a deck thickness of 6 1/2 in. This section of interstate was under construction during this study.

N2-N and N2-S

These bridges are located on Ellington Parkway in Nashville, Tennessee. The bridges are over Douglas Avenue. Both bridges have two continuous spans of 63 ft. and 6 in. and 69 ft. and 6 in. The skew is 90°. N2-N has a roadway width of 44 ft. and N2-S has a roadway width of 38 ft. Both bridges have a slab thickness of 7 in. The slab of N2-N is supported by six girders and N2-S is supported by five girders. These bridges were inspected only after being subjected to construction traffic. This road system is now open to traffic.

3. Bridges Subjected to Normal Traffic

CO-E and CO-W

These bridges are located on Interstate 40 at Crab Orchard, Tennessee and carry Interstate 40 traffic over

US 70 and the Tennessee Central Railroad. Each bridge consists of five continuous spans (82 ft. and 6 3/4 in., 107 ft. and 6 in., 79 ft. and 6 in., 95 ft. and 8 7/8 in., 74 ft. and 10 1/2 in.). The bridges have a roadway width of 30 ft. and a slab thickness of seven inches. The bridges are on a 30° 33' skew. The slab is supported by three girders on each bridge. The average daily traffic for these bridges was 9016 vehicles in 1971.

C-N and C-S

C-N and C-S are located in Cookeville, Tennessee and carry State Route 111 traffic over Interstate 40. Both bridges consist of three continuous spans of 110 ft. and 6 in. each and each bridge has a skew of 63°. The width of the roadway is 40 ft. for each bridge. The slabs are 7 1/2 in. thick and are supported by five girders. These bridges have a combined average daily traffic of 1070 vehicles.

N1-N and N1-S

These bridges are located on Interstate 65 north of Nashville, Tennessee and carry the Interstate traffic over the Louisville and Nashville Railroad. Both bridges are three continuous spans and consist of spans of 78 ft. and 2 1/2 in., 88 ft., 80 ft. and 2 1/4 in. Both bridges have a skew of 28° 36'. N1-N has a roadway that varies from 44 ft. and 5 in. to 63 ft. and 6 3/4 in. and for this study a section was used that had a width of 30 ft. N1-S has a roadway

width of 41 ft. and 2 in. Both bridges have a deck thickness of seven inches. The combined average daily traffic is 29,408 vehicles.

4. Location of Cracks

In order to report the deterioration due to cracking and also that the deterioration could be compared after the second inspection, some method of recording the location of the cracks was needed. From this information other methods of presenting the deterioration was possible.

A relatively simple method was employed to mark the cracks on a bridge deck. The bridge deck was swept clean and then wet with water. As the water evaporated from the surface, the water in the cracks evaporated slower than the surface water. For this reason the cracks were seen as dark lines on the bridge deck. The cracks were then marked. After all the cracks were marked, a three foot grid was marked on the surface of the deck. Using the grid for reference, the cracks were recorded. These records were used to make drawings of the bridge deck and also to figure the total length of cracks per ten foot section and the cracking intensity. Although this method is very simple, it was found to work quite well.

5. Determination of Total Length of Cracks over Six Feet in Length Per Ten Foot Section

It was decided to present the data collected in mapping the cracks in some manner. Some thought was given to the form used by Chu[16] and Young[3]. Chu and Young both plotted the number of cracks per ten foot section. Chu used cracks over six feet in length and Young used cracks that were 1/2 the width of a lane and if the crack was thought to be part of another crack, it was counted as part of the other crack. Chu presented data in the form before open to traffic and after nine months of service while Young made only one presentation of data.

It was felt that the method used by Young was subject to some error. A crack over a reinforcing bar has been shown to wander randomly when the bar it is over is bent down.[4] For this reason a crack may be over two or three bars and be connected while there are single cracks over the bars. Using Chu's method, the number of cracks could remain the same while the total length of cracks increased. Chu included tables containing the toal length of cracks, but it was felt that a better comparison could be made if the toal length of cracks over six feet in length was plotted instead of the number of cracks. An arbitrary length of six feet was chosen because Chu indicated that shrinkage cracks occurring on bridges during curing rarely exceeded six feet in length.[16]

The length of the bridge was measured at the center of the studied width, and the total length of cracks over six feet in length was calculated for each ten foot section and these values were plotted along the length of the bridge. This is presented for the bridges study in Figures 2 through 7.

6. Determination of Cracking Intensity

Cracking intensity is a measure of the cracking in a given moment region. Young defined cracking intensity as the number of cracks divided by the length of the moment region.[3] Chu also used this definition.[16] Since it was felt that total length of cracks over six feet in length was superior to total number of cracks, the total length of cracks over six feet in length per moment region was divided by the length of the moment region. Since bridges of different widths could have the same cracking intensity, it was decided to further define the cracking intensity as the total length of cracks for all cracks over six feet in length divided by the length of the moment region divided by the studied width of the bridge. In this way, the cracking intensities of bridges with different widths could be compared directly.

The cracking intensities were calculated in the following manner. The moment regions were determined using the placement of shear connectors as indicators of the ends

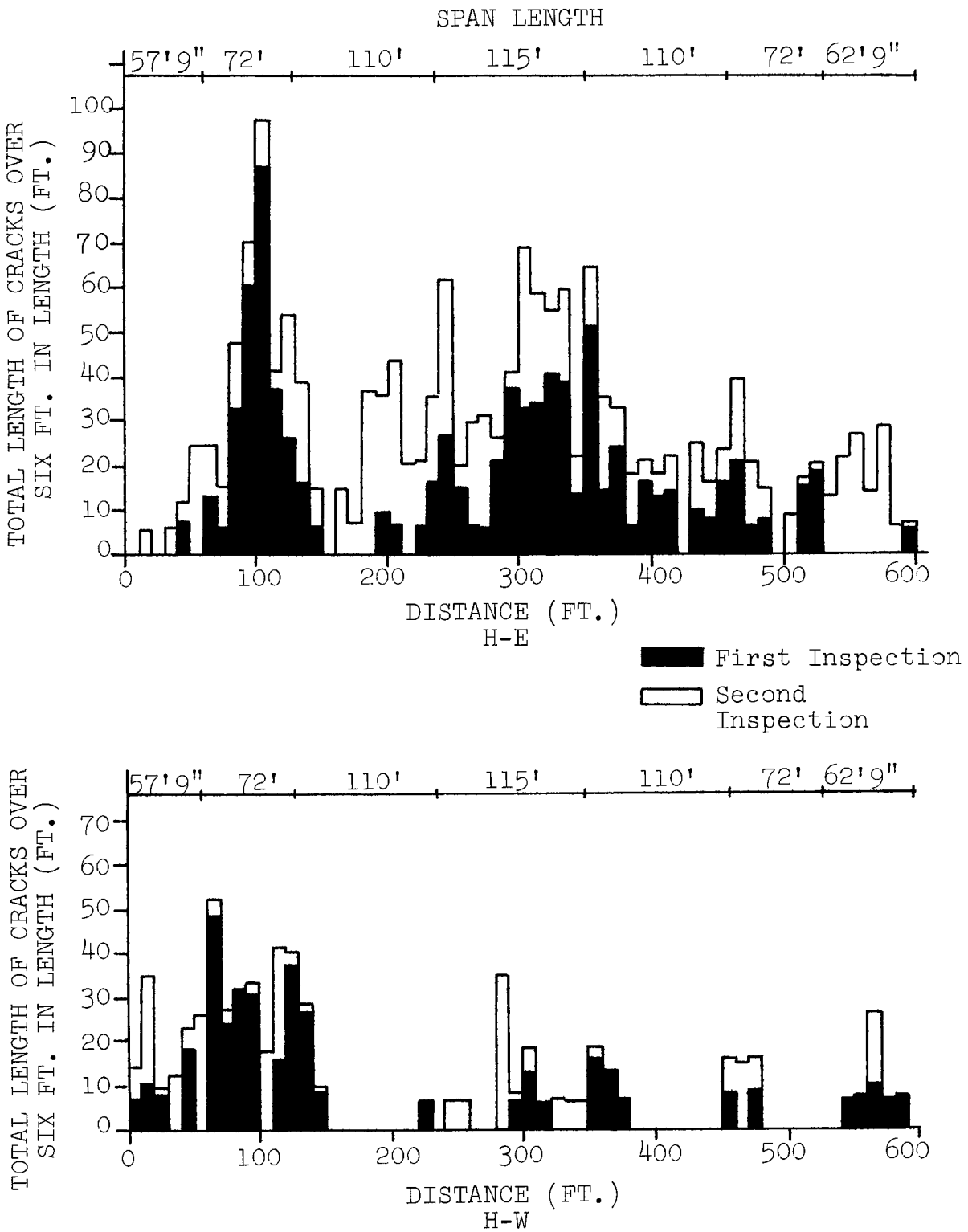


Figure 2. Transverse Crack Length Per Ten Foot Section, Bridges H-E and H-W.

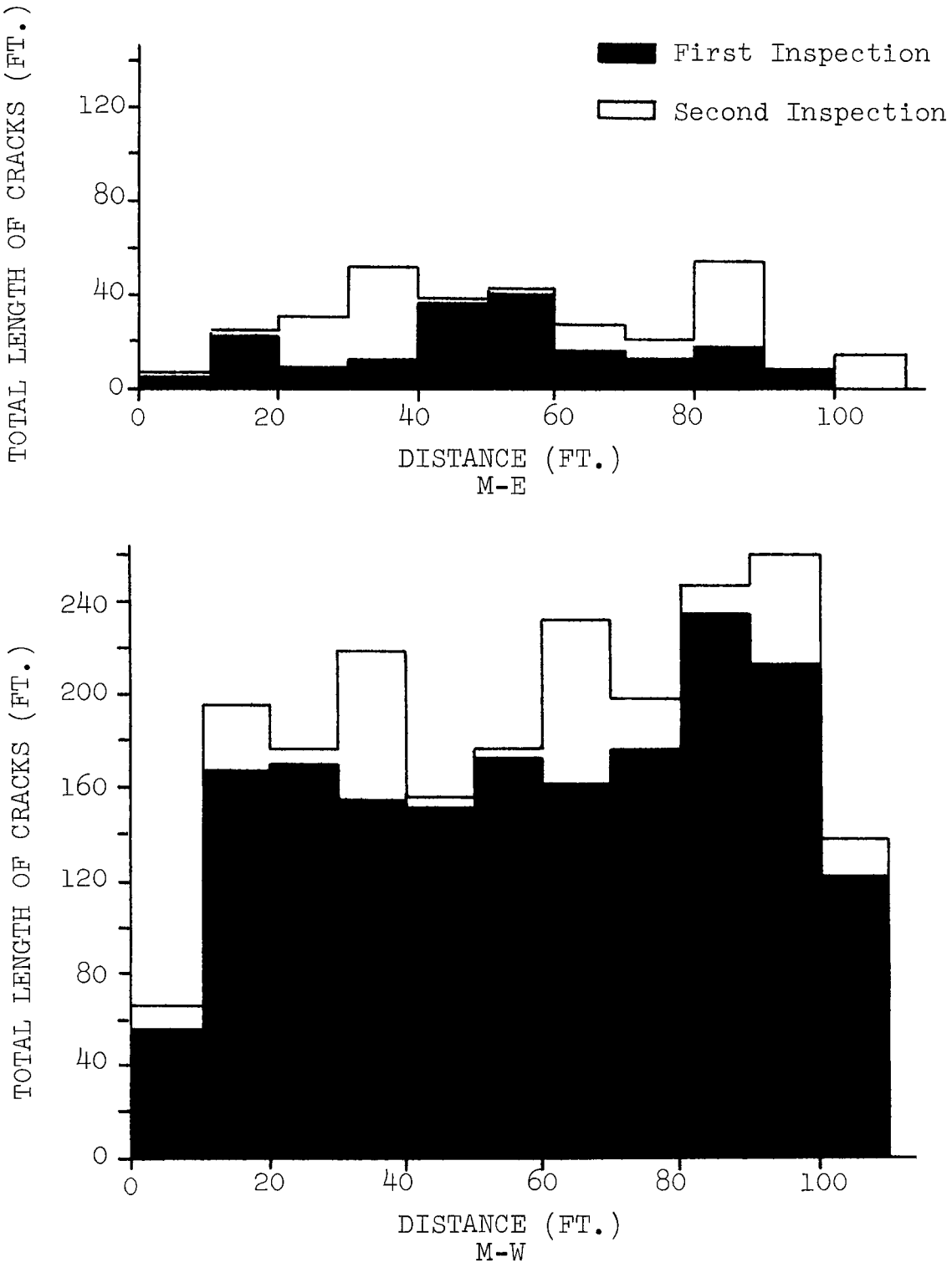


Figure 3. Transverse Crack Length Per Ten Foot Section, Bridges M-E and M-W.

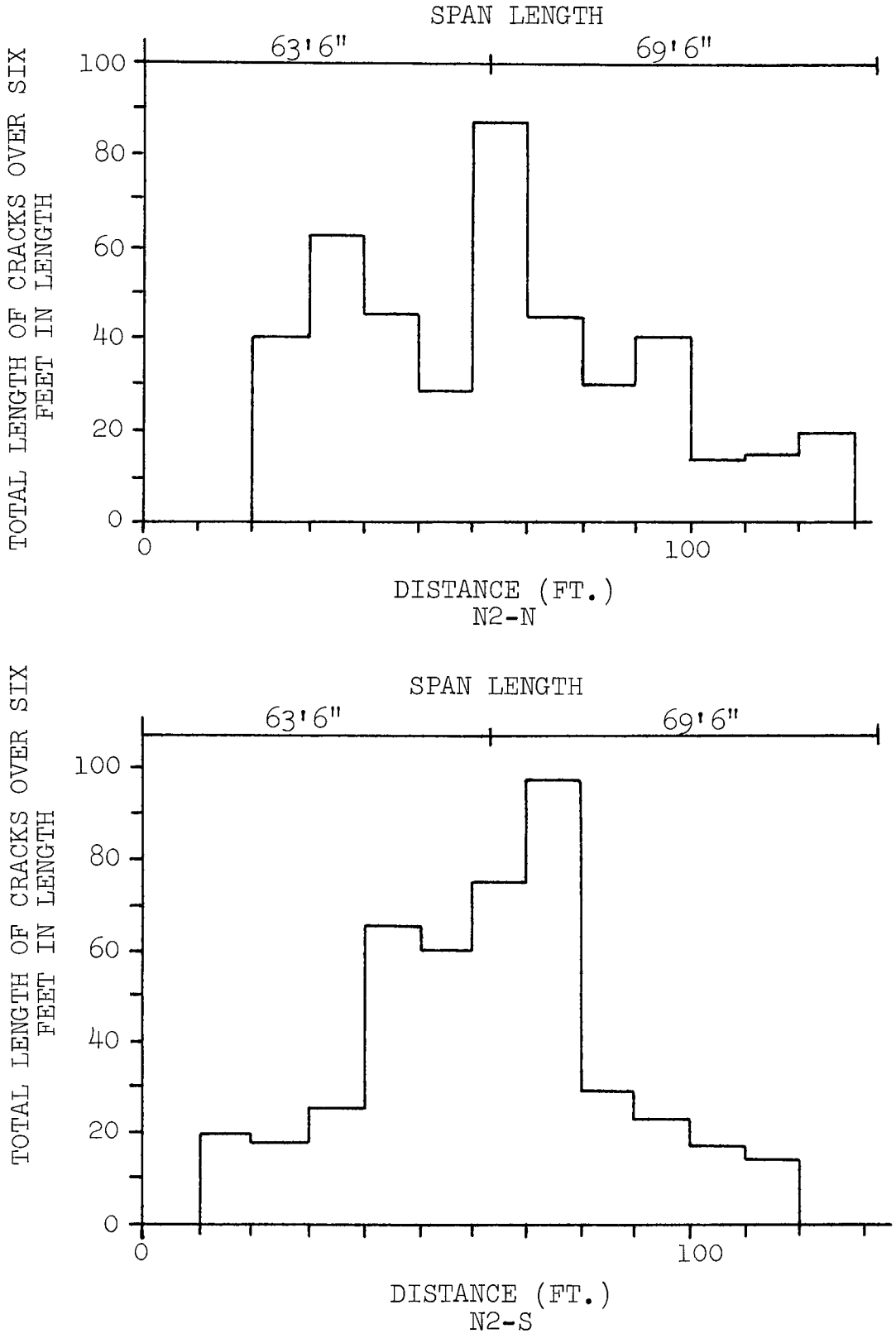


Figure 4. Transverse Crack Length Per Ten Foot Section, Bridges N2-N and N2-S

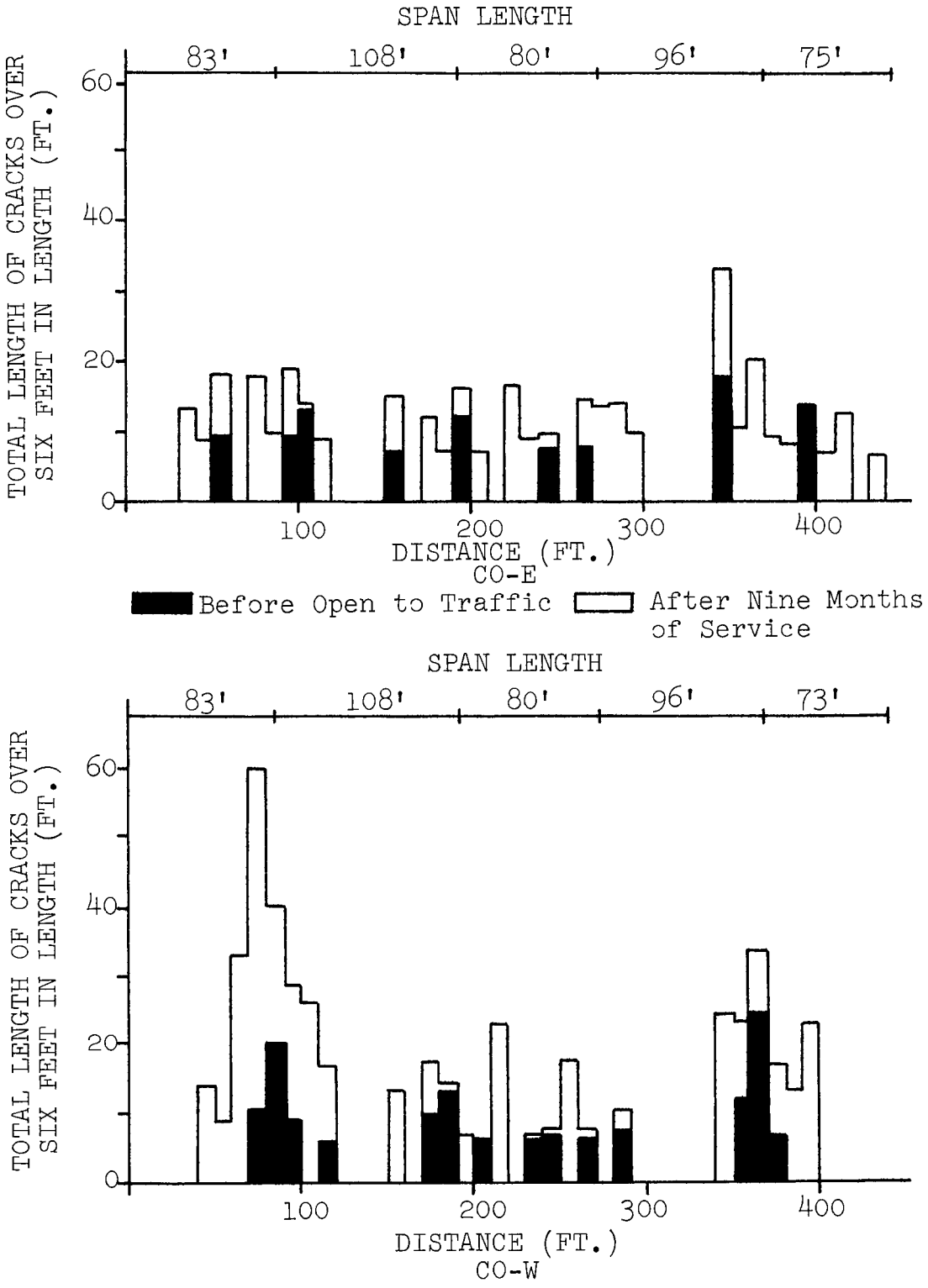


Figure 5. Transverse Crack Length Per Ten Foot Section, Bridges CO-E and CO-W.

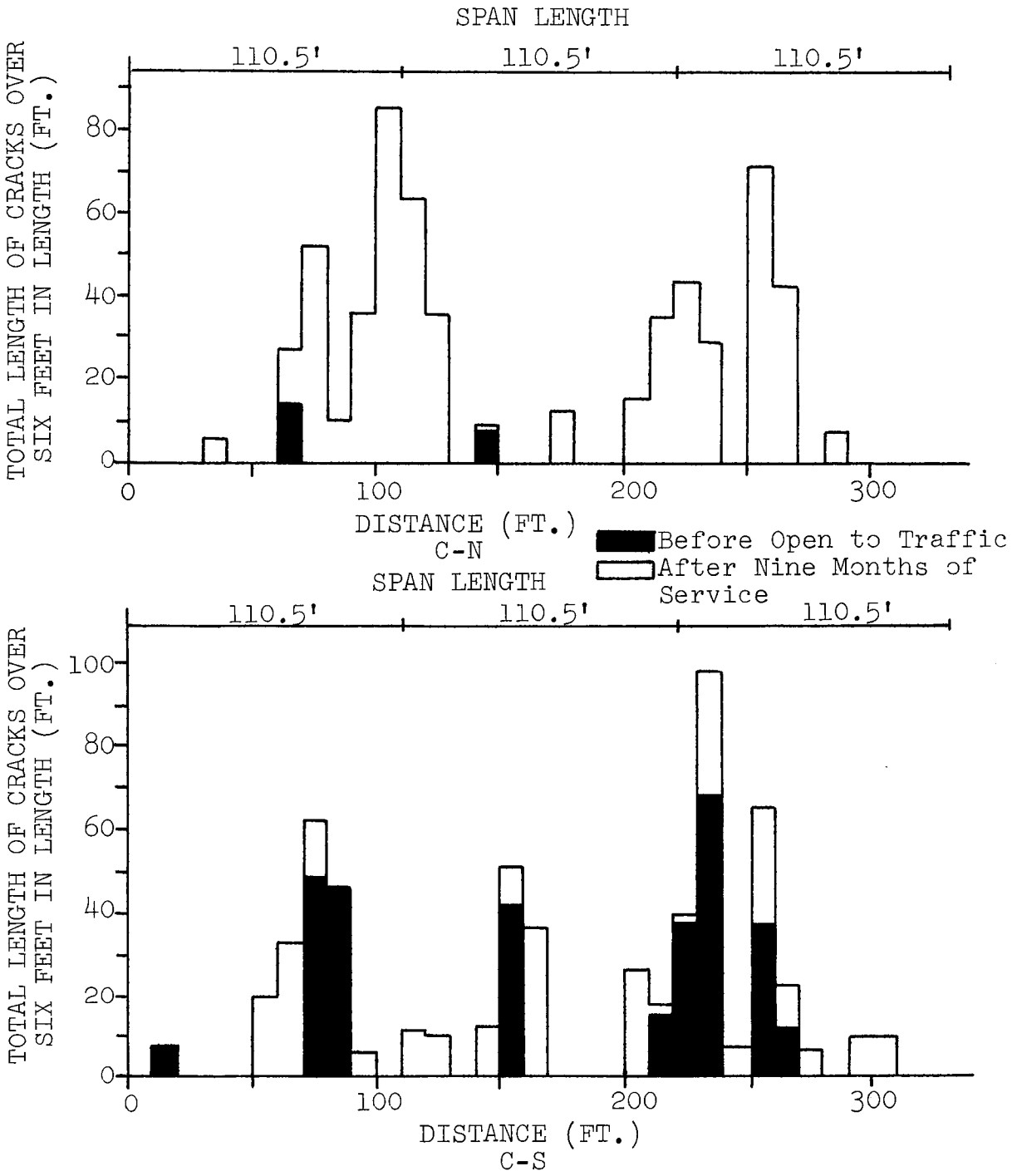


Figure 6. Transverse Crack Length Per Ten Foot Section, Bridges C-N and C-S.

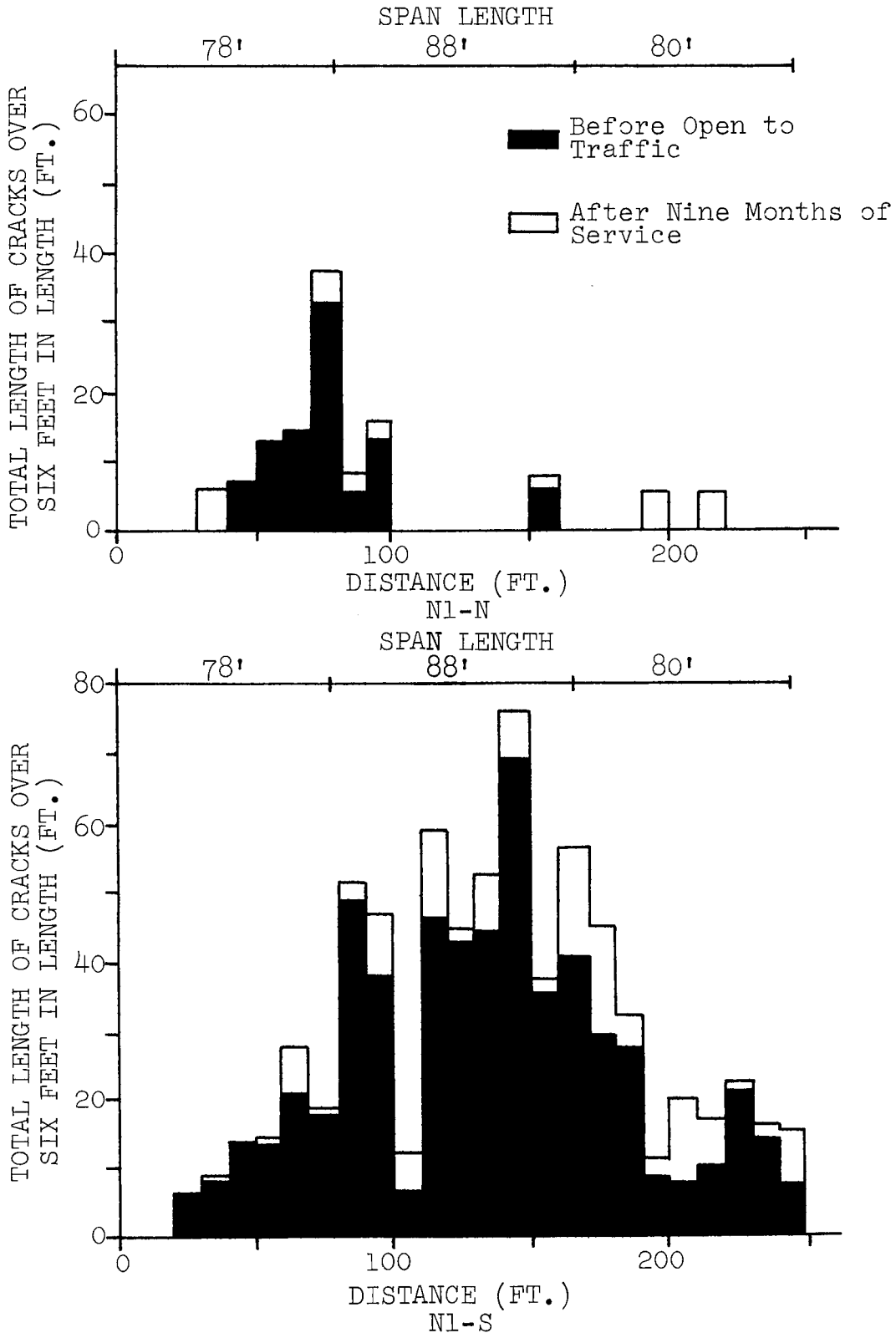


Figure 7. Transverse Crack Length Per Ten Foot Section, Bridges N1-N and N1-S.

of the moment regions. The total length of all cracks over six feet in length was divided by the length of the moment region and by the width of the section. This gave the cracking intensity units of ft./ft.^{2} and the cracking intensities for the bridges under study are given in Figures 8 through 12.

7. Determination of Natural Frequencies and Damping

The natural frequencies and damping were determined for each bridge using a deflection gage and a Sanborn recorder. The deflection gage was attached to a girder at the centerline of the span. When a truck came on the span, the recorder was turned on and recorded the deflection of the bridge. The part of the recording after the truck left the bridge was used to calculate the frequency of the bridge. The damping was calculated using the magnitude of the deflection at a selected starting point and at a point several cycles later. The natural frequencies and the damping for each bridge are shown in Table 1.

8. Measurement of Clear Cover

The clear cover on several bridges was measured using a James C-4949 deep pachometer. Goodpasture and Goodwin[17] discuss the workings of the pachometer in greater detail than will be attempted in this study. Calibration curves were generated for a No. 4 and a No. 6 deformed reinforcing bar. Measurements in the field consisted of recording the

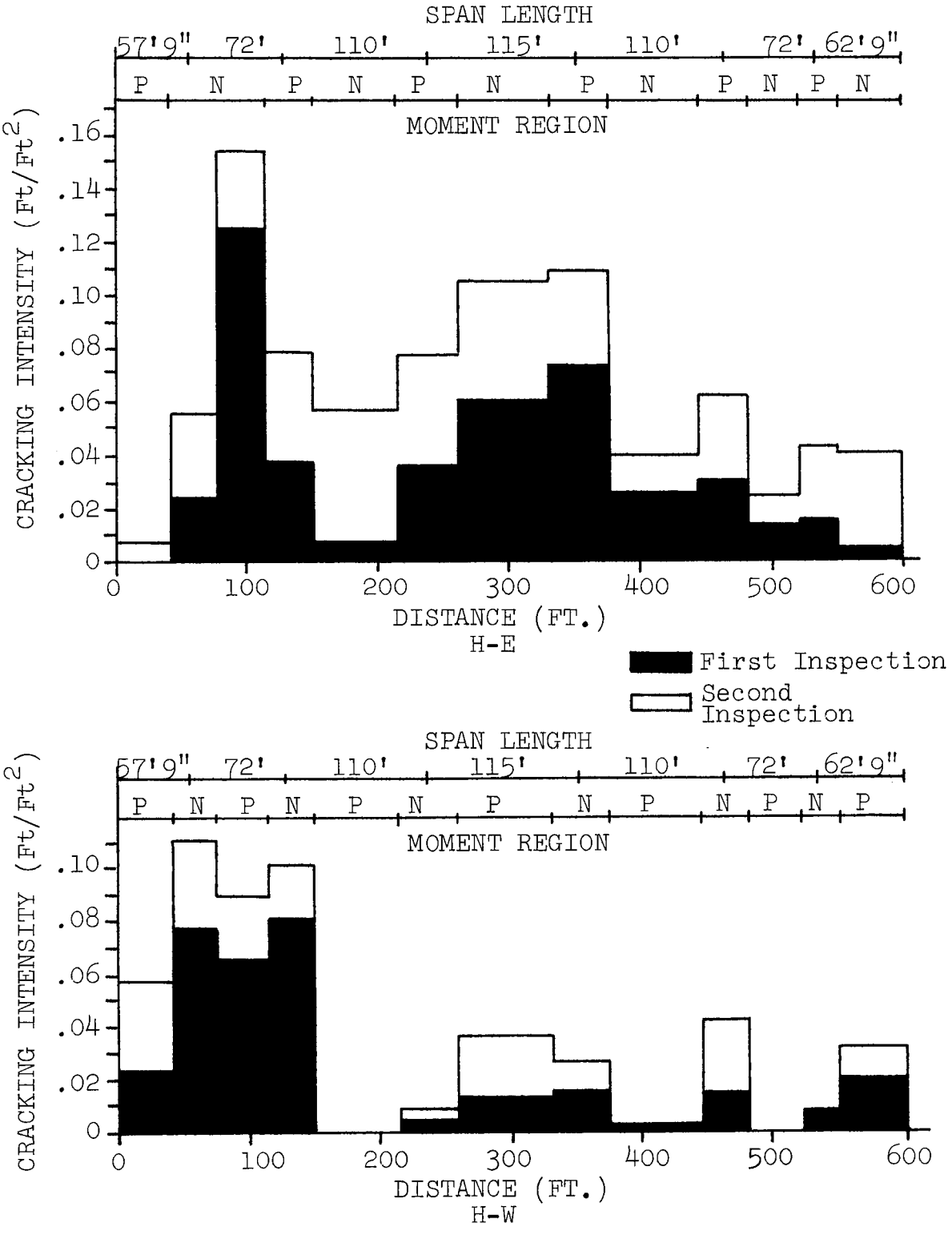


Figure 8. Cracking Intensity (Transverse) Bridges H-E and H-W.

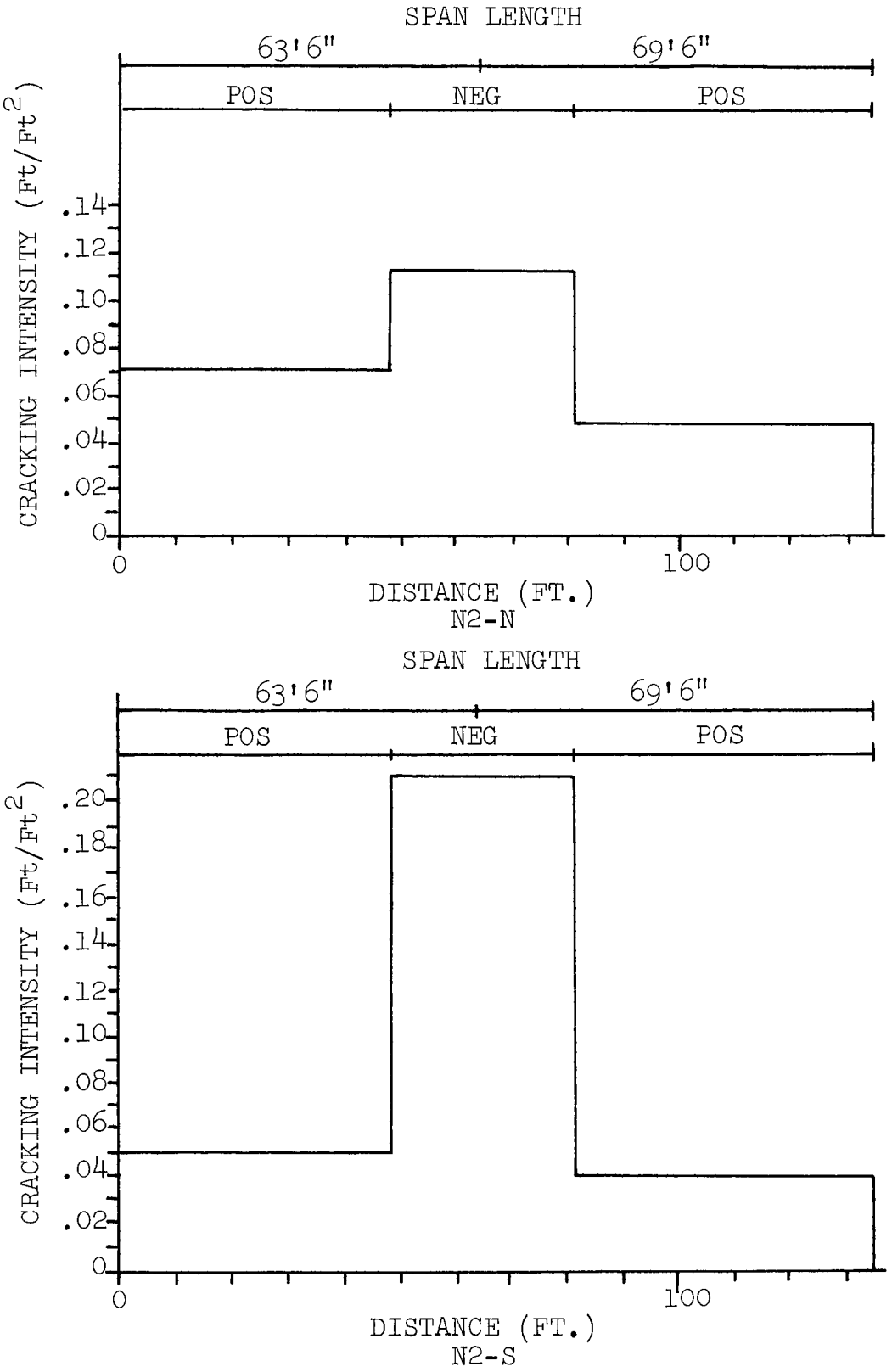


Figure 9. Cracking Intensity (Transverse) Bridges N2-N and N2-S.

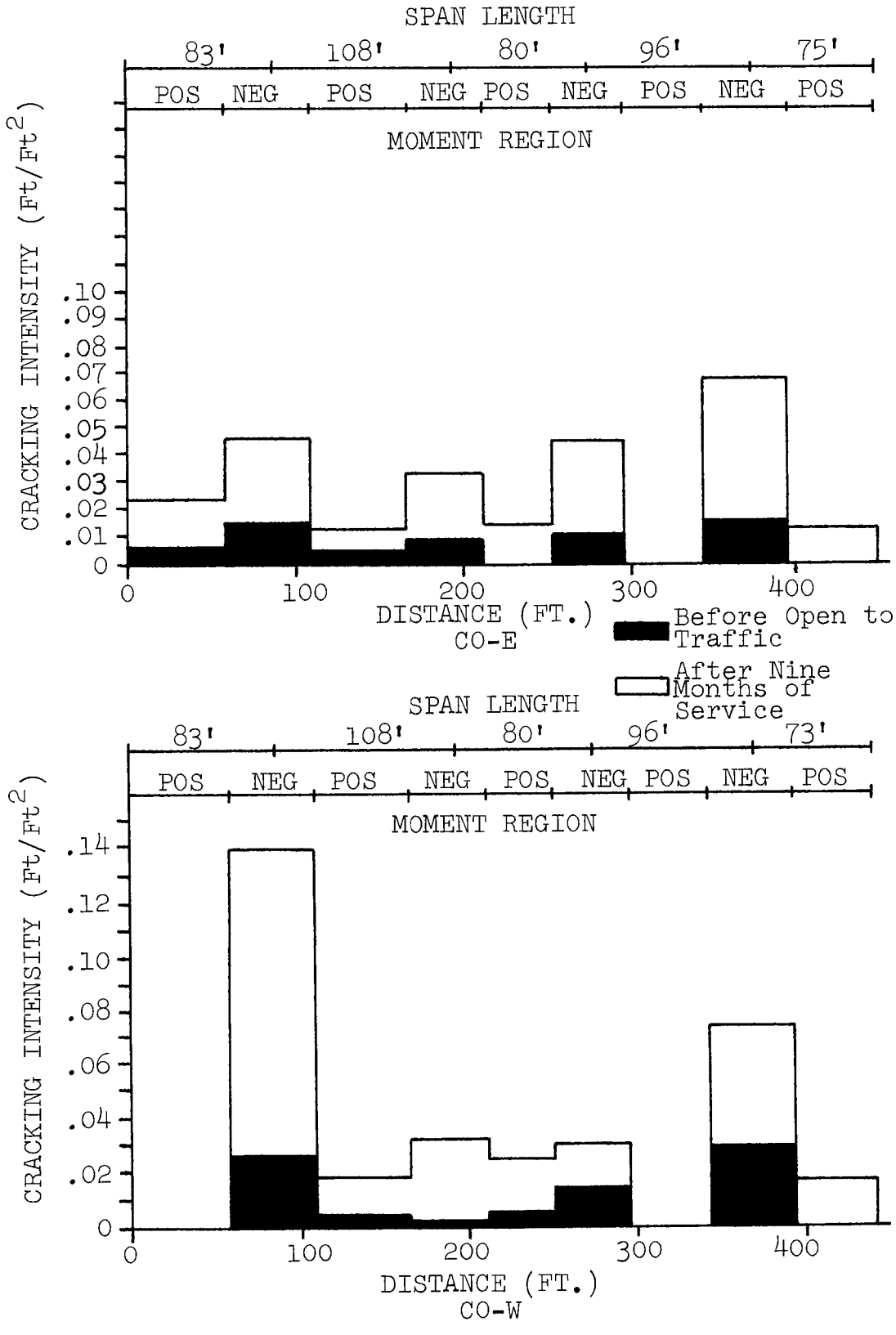


Figure 10. Cracking Intensity (Transverse) Bridges CO-E and CO-W

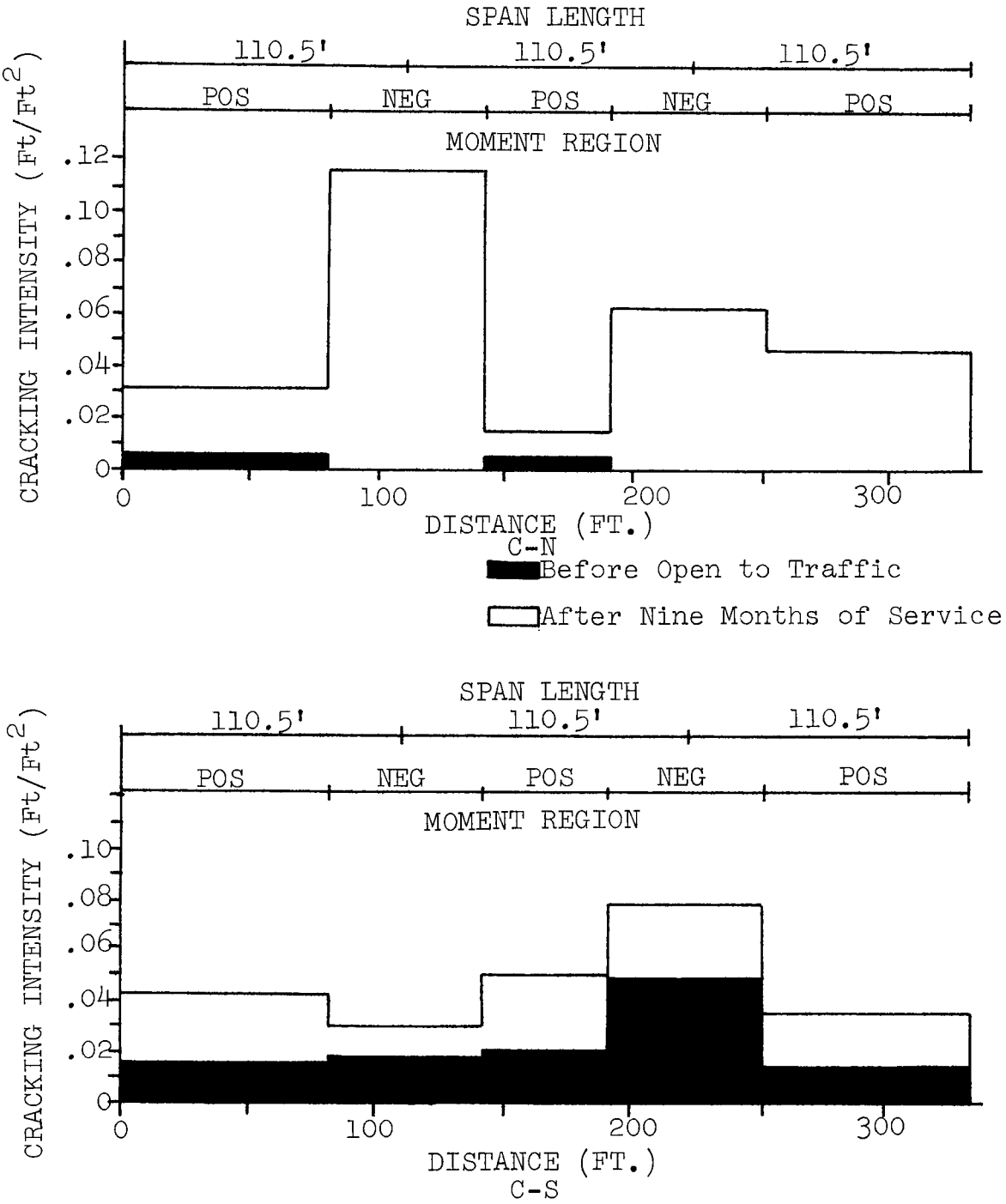


Figure 11. Cracking Intensity (Transverse) Bridges C-N and C-S.

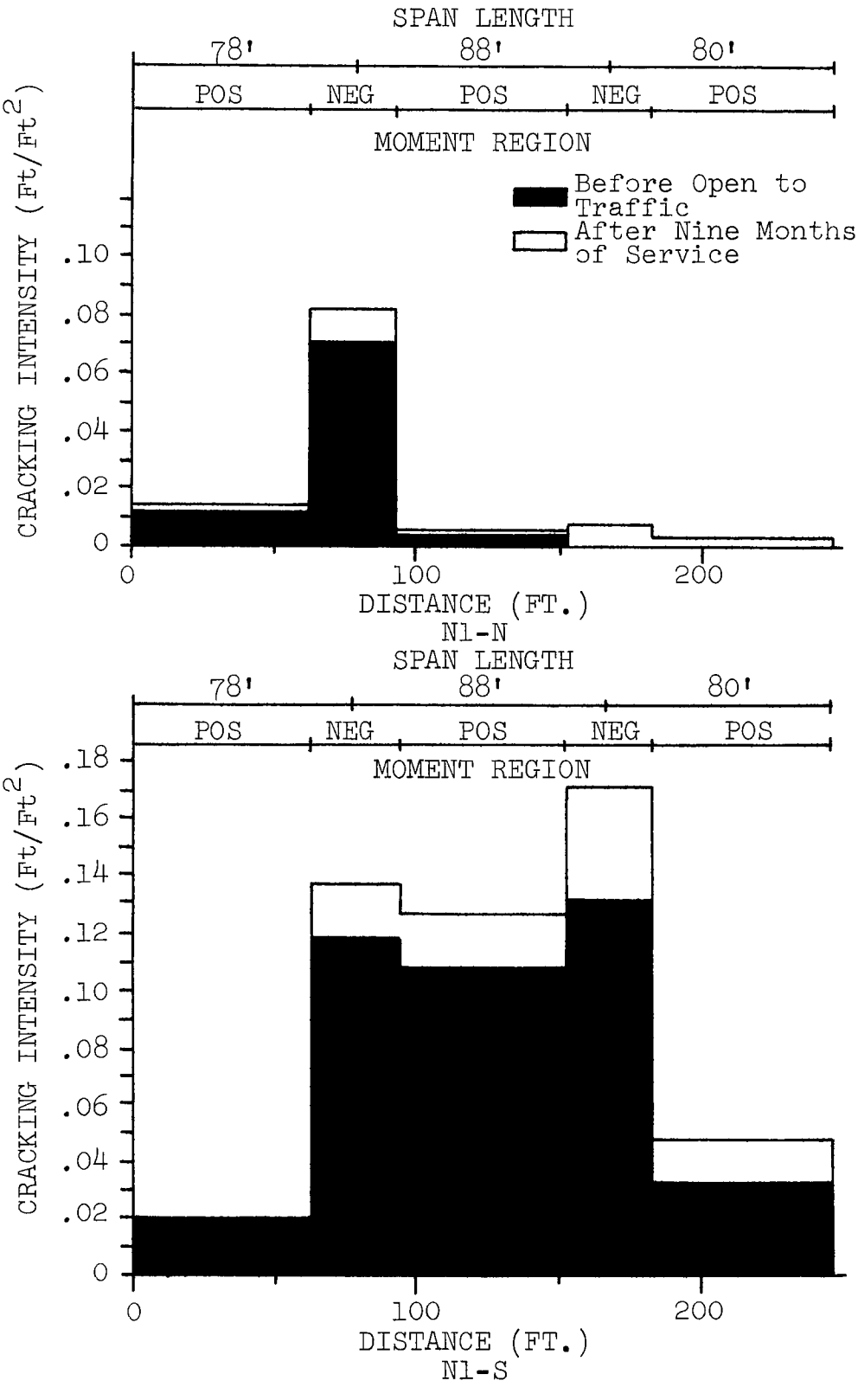


Figure 12. Cracking Intensity (Transverse) Bridges N1-N and N1-S

TABLE I

BRIDGES SUBJECTED TO NORMAL AND CONSTRUCTION TRAFFIC

Bridge	Date Mapped	Frequency Hertz	Damping Percent Critical	Date Remapped	Frequency Hertz	Damping Percent Critical
H-E	6/26/69	2.42	.49	8/12/71	--	--
H-W	7/1/69	2.51	.31	8/10/71	--	--
CO-E	7/7/69	3.84	.92	7/14/70	3.79	1.28
CO-W	7/9/69	3.79	.42	7/15/70	3.81	1.50
C-N	7/14/69	2.54	.31	5/20/71	2.61	.51
C-S	6/10/70	2.60	.46	5/21/71	2.59	.33
N1-N	7/24/69	4.54	.63	7/8/70	4.51	1.39
N1-S	7/23/69	4.54	.67	7/7/70	4.20	.94
N2-N	5/20/70	5.58	1.03	--	5.50	1.56
N2-S	5/21/70	5.74	1.18	--	5.17	1.28
M-E	7/29/69	(7.65) (4.05)	--	10/30/70	7.44	.64
M-W	7/29/69	3.17	.94	10/30/70	2.97	1.35

deflection of the galvanometer in dial units. The calibration curves were then used to determine the amount of clear cover in inches for each reading of the galvanometer. To determine the average clear cover, the sum of all measurements was divided by the number of measurements. This was used since the cover in inches is not linearly proportional to the deflection in dial units.

III. RESULTS

1. Bridges Subjected to Construction Traffic Only

H-E and H-W

At the time of the first inspection, transverse cracking was the major form of cracking on H-E. There were many long transverse cracks and there was also some pattern cracking. Most of the long cracks were located in a section from 47 feet from the east end of the bridge to 133 feet from the east end of the bridge. At the time of the second inspection, it was noted that cracking had increased not only by the lengthening of the original cracks but also by the formation of new cracks. Many of the new cracks were located in the end pours of the bridge. Cracking had increased to such an extent that the cracking intensity had more than doubled for some moment regions as shown in Figure 8, page 28.

The natural frequency of H-E was measured one time and found to be 2.42 cycles per second. The damping was found to be 0.49 per cent of critical.

The highest cracking intensity on H-E occurred in a positive moment region and this was part of the final pour made on this bridge. The final pour covered a positive moment region and two negative moment regions. The high temperature on the day of casting was 79°F and the low

temperature was 50°F. For the next five days, the high temperature ranged between 62°F and 81°F while the low temperature ranged between 28°F and 46°F.

The depth of clear cover was measured for each pour on H-E. The average depth of cover was found to be 1.88 inches for the whole bridge. The average depth of cover for each pour is shown in Figure 13. The depth of cover was found to vary from less than 0.75 inch to 2.88 inches.

The transverse cracking of H-W was not as bad as on H-E. It was also noted that adjacent pours differed in cracking. One pour would have predominately transverse cracks and the adjacent pour would have few transverse cracks and be badly pattern cracked. The increase in cracking was noted to be caused by the lengthening of original cracks and the formation of new cracks. The cracking intensity did not increase between inspections as greatly as on H.E.

The natural frequency of H-W was measured and found to be 2.51 cycles per second. The damping was found to be 0.31 per cent of critical.

The depth of cover for H-W was measured and the average depth of cover was found to be 1.91 inches. The depth of clear cover was found to vary from a low of 0.92 inch to a high of 2.88 inches. The average depth of cover was found for each pour and this is shown in Figure 13.

Traffic on both H-E and H-W consists of construction

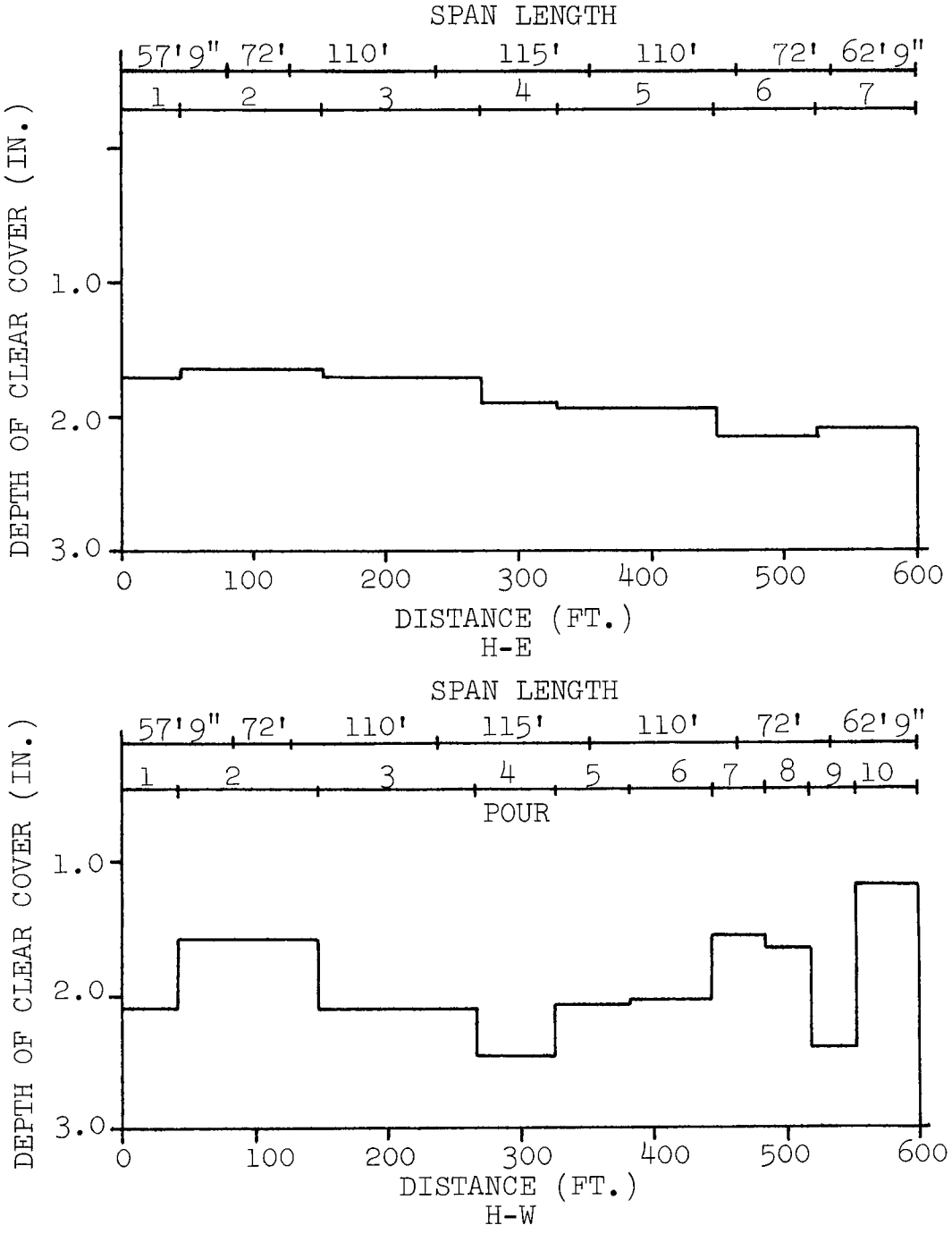


Figure 13. Depth of Clear Cover, Bridges H-E and H-W.

traffic with some local traffic. H-E is used by the traffic to a much greater extent than H-W. The main construction road is in direct line with H-E which is the wider bridge. At the time of the second inspection, three axle and four axle dump trucks loaded with rock were running over H-E and were diverted to H-W while H-E was being inspected. It was noted at this time that there was a larger dropoff at the ends of H-W than H-E.

M-E and M-W

At the time of the second inspection, M-E was not observed to be cracked significantly greater than at the time of the first inspection. There was some increase in the cracking, mostly lengthening of the cracks noted on the original inspection. To show the increase in cracking, the total length of all cracks was plotted instead of the total length of cracks over six feet in length. This is shown in Figure 3, page 22. Since this bridge is a simple span, the cracking intensity was not determined.

The major increase in deterioration of M-E was a large increase in the number of popouts. At the time of the original inspection there were 14 popouts and the number of popouts had increased to 75 at the time of the second inspection. Several sections which showed signs of an imminent popout were inspected. The section was pried from the concrete deck and it was found that a piece of chert aggregate was located in the bottom of each popout. The

chert was found to be fractured and loose and it was evident that the aggregate was not holding the popout to the deck and that the cement paste was supplying the only holding force.

The natural frequency of M-E was measured and was found to have decreased from 7.65 cycles per second to 7.44 cycles per second. The damping of the bridge was not measured at the original inspection and was found to be 0.64 per cent of critical at the time of the second inspection.

M-E was cast on August 17, 1967. On this date the high temperature was 83°F and the low temperature was 70°F. During a period of 15 days after and including the day of casting, the high temperatures were below 87°F, and the low temperatures were above 55°F. On the day of casting, the lowest relative humidity was 61 per cent, and the three day average following casting was 74 per cent. The average relative humidity for the 15 day period after and including casting was 65 per cent with a range from a low of 36 per cent to a high of 82 per cent. On the day of casting, the average wind velocity was 6.8 miles per hour and for four days thereafter the average wind velocity was above seven miles per hour.

The amount of clear cover on M-E was measured with a C-4949 Pachometer and was found to average 1.62 inches. The low measurement was less than 3/4 inch and the high

measurement was 2.82 inches. There were 48 measurements of cover taken. The bridge is superelevated and measurements of cover were taken on the high side, the middle, and the low side of the bridge. Sixteen readings were taken in each area. The average cover on the high side was 1.05 inches with a low of less than 0.75 inch and a high of 1.85 inches. The average cover of the middle was 1.54 inches with a low of 1.0 inch and a high of 2.15 inches. The average cover on the low side of the bridge was found to be 2.26 inches with a low of 1.75 inches and a high of 2.82 inches. The depth of clear cover was found to increase in going from the high side to the low side in all sections measured. The specified clear cover for this bridge is 1.5 inches.

M-W was found to be greatly cracked on both inspections, however the cracks were generally found to be less than six feet in length. So that an indication of deterioration could be determined, the total length of all cracks per ten foot section was calculated and this was used instead of total length of cracks over six feet in length per ten foot section. This is shown in Figure 3, page 22. The cracking increased but not to a significant extent. All the cracking was in the transverse direction and no longitudinal cracks were found. The increase in cracking was due mainly to an increase in small cracks. These cracks were primarily located in the wheel paths of the vehicles

using these bridges. Most of these cracks are less than three feet in length, and very few cracks on the bridge are greater than six feet in length.

The second inspection of M-W also revealed an increase in the number of popouts. There was only one popout noted at the time of the original inspection, and there were over 30 popouts at the time of the second inspection. Several sections which showed signs of popping out were examined as on M-E and the results were the same as for M-E.

The natural frequency for M-W was measured and was found to have decreased from 3.17 cycles per second to 2.97 cycles per second. The damping was found to have increased from 0.94 per cent of critical to 1.36 per cent of critical.

M-W was cast on February 19, 1968, and the high temperature for that day was 50°F and the low temperature was 26°F. The high temperature for the next day was 54°F and the high temperatures for the next ten days were below 50°F. The low for February 20, 1968 was 36°F, and the low temperatures for the next ten days were below 30°F. On the day of casting, the average wind velocity was 7.2 miles per hour, and the average wind velocity for each of the next four days was above ten miles per hour, with an average for those days of 13.8 miles per hour. The relative humidity was 25 per cent on the afternoon of casting, and

the average for the first three days was 50 per cent. The relative humidity varied from a low of 19 per cent to a high of 81 per cent in the 14 day period after and including the day of casting.

The amount of clear cover was also measured on M-W. This bridge is superelevated and measurements were made on the high side, the middle, and the low side. A total of 53 measurements were taken and the average depth of cover was found to be 1.13 inches. The average depth of cover on the high side was found to be 1.10 inches with a range from a low of 0.80 inch to a high of 1.45 inches. The middle section had an average depth of cover of 0.92 inch with a range from a low of less than 0.75 inch to a high of 1.28 inches. The low side was found to have an average depth of cover of 1.39 inches. The measurements ranged from a low of 0.94 inch to a high of 1.84 inches. The depth of cover was always found to be greater on the high side than in the middle, but it was always greater on the low side than on the high side. The specified clear cover for this bridge was 1.5 inches.

The traffic on both M-E and M-W was construction traffic and consisted of transit mix concrete trucks. There was a concrete batching plant located nearby. These bridges were used in the hauling of concrete to other areas of the construction project.

N2-N and N2-S

N2-N and N2-S were only inspected one time, and this inspection was carried out after the two bridges had been opened to traffic for a short time. Transverse cracking was the predominate form of cracking on N2-N. There were few transverse cracks that ran the width of the bridge. The cracking was particularly heavy in the region over the support. There was some longitudinal cracking noted. The majority of the longitudinal cracking was over the first girder to the right of the longitudinal centerline in the direction of traffic. Some longitudinal cracking was noted over the first girder to the left of the longitudinal centerline facing in the direction of traffic. There was very little longitudinal cracking over the other girders.

Several transverse cracks ran the width of the bridge on N2-S. Most of the long transverse cracks were located in the region above the support. Longitudinal cracking is particularly severe over the middle girder. The cracks appear to be discontinuous, but they are part of continuous cracks over the top longitudinal reinforcement on each side of the girder. These continuous cracks run the length of the bridge. Shorter longitudinal cracks occur over the adjacent girders and are scattered along the bridge deck over the girders.

Both bridges were found to have light scale and shrinkage cracking. A check of the weather data for the

time of casting of these bridges did not show any unfavorable conditions other than the average wind velocity. The average wind velocity was above ten miles per hour on each of the days of casting and in one case was above 12 miles per hour.

Two measurements were made of the natural frequency. The first was made at the time of the inspection and the second was made almost 14 months later. During this period, the natural frequency of N2-W decreased from 5.58 cycles per second to 5.50 cycles per second. The damping increased from 1.03 per cent of critical to 1.56 per cent of critical. The natural frequency of N2-S decreased from 5.74 cycles per second to 5.17 cycles per second. The damping increased from 1.18 per cent of critical to 1.28 per cent of critical.

The bridges were used in the construction portion of the project and hauling of fill material started approximately one month after the final pour of N2-S was completed. The vehicles used were payhaulers with a load of 90,000 pounds if the vehicle was half loaded as allowed by specifications. Adjacent structures were five months old and 2 1/2 months old when hauling started. None of these four bridges were as cracked as N2-N and N2-S. A total of approximately 180,000 cubic yards of material was hauled across the two bridges with approximately 140,000 cubic yards hauled over N2-S.

The cylinder test of 28 day strength were from 3861 pounds per square inch to 5113 pounds per square inch. The concrete had a slump of from three inches to 3 3/4 inches and the air entrainment ranged from 4.5 per cent to 7.1 per cent.

Longitudinal cracking was not noted on H-E, H-W, M-E, or M-W, however longitudinal cracking was noted on N2-N and N2-S. This longitudinal cracking was located over a longitudinal girder. An approximate analysis was made to determine the transverse moments in the slab for both of these bridges. N2-S consists of five girders supporting the slab. It was found that a truck straddling the center girder produced a moment that was close to the maximum produced in the slab. Since construction traffic usually travels down the center of a bridge with no regard to the traffic lanes, the moment produced by the construction traffic would be near a maximum. It was found that the maximum moment on N2-N occurred when a vehicle straddled the exterior girder. Because of the practice of using the center of the bridge, the maximum moment would not be achieved by the construction traffic. If a vehicle straddles either of the girders nearest to the center of the transverse section, a moment is produced that is a little less than the moment produced on N2-S. From this approximate analysis, longitudinal cracking would be expected over the center girder of N2-S and over the two

interior girders of N2-N. The longitudinal cracking was found over each of these girders in the field.

2. Bridges Subjected to Normal Traffic

CO-E and CO-W

At the time of the first inspection, the cracking on CO-E was concentrated mainly in the negative moment regions although there was some cracking in the positive moment regions. Longitudinal cracking was present over the middle girder of the bridge, mostly in the negative moment region. After nine months of normal traffic, the transverse cracking had increased considerably in the negative moment regions. Transverse cracking had also increased in the positive moment regions, but not as greatly (Figure 10, page 30). The longitudinal cracking had increased and it was evident that two major longitudinal cracks ran the length of the bridge over the center girder. There were no transverse cracks found to run the width of the bridge although several cracks ran from the curb to the center of the bridge. After the first inspection, the outside lane seemed to have more cracking than the interior lane.

The natural frequency of CO-E decreased from 3.84 cycles per second to 3.79 cycles per second. The per cent of critical damping increased from 0.92 per cent to 1.28 per cent.

The negative moment regions on CO-E were cast on

April 24 and 25, 1967. The high temperature on April 24 was 62°F and the low was 39°F. On April 25, the high temperature was 58°F and the low was 35°F. During the period of 16 days after casting, a low temperature of 32°F was recorded on April 28, and a high temperature of 66°F was recorded on April 29.

CO-W showed a large amount of longitudinal cracking at the time of the second inspection after nine months of normal traffic. At the time of the original inspection, the longitudinal cracks were in the region of the supports over the center girder. There were no longitudinal cracks in the positive moment regions. The longitudinal cracks had lengthened by the time of the second inspection, and it was evident that there were two discontinuous longitudinal cracks running the length of the bridge, one on each side of the center girder. This was very prominent in the negative moment regions, and it was also evident in the positive moment regions. At the time of the first inspection, the transverse cracking was almost entirely in the pours over the supports. Cracking had increased in the pours over the supports at the time of the second inspection. Cracking in the other pours had also increased. The transverse cracking does not seem to be concentrated in either traffic lane. Many of the longer cracks run from the curbs to the center girder and no cracks run the width of the bridge. Very few transverse cracks cross the center girder.

The natural frequency of CO-W was measured and found to increase slightly from 3.79 cycles per second to 3.81 cycles per second. Damping also increased from 0.42 per cent of critical to 1.50 per cent of critical.

The negative moment regions were cast on two consecutive days. The two negative moment regions on the west end of the bridge were cast on June 5, 1967. The high temperature on that day was 76°F, and the high temperature increased to 86°F in 11 days. The low temperature was 57°F and gradually increased to 63°F in 13 days. The other two negative moment regions were cast on June 6 and the temperature information is the same as for the first two pours from the date of casting on.

There was no determination of the amount of clear cover on either bridge. These bridges are open to normal interstate traffic.

C-N and C-S

C-N was inspected 11 months before C-S and was opened to traffic one year after the initial inspection. At the time of the first inspection, there was very little cracking. The cracking was in the transverse direction and no longitudinal cracks were located. The cracking consisted mainly of small shrinkage cracks less than three feet in length, and they were concentrated on the interior ends on the exterior pours. At the time of the second inspection,

the transverse cracking had increased (Figure 6, page 25, Figure 11, page 31) and once again no longitudinal cracking was present. There was a large amount of cracking over the supports (Figure 6, page 25). The exterior pours were cracked primarily on the interior ends. Cracking of the interior pours was fairly uniform within each pour. Figure 11, page 31 shows that the cracking intensities were the highest in the negative moment regions. The increase of cracking intensity was also highest in the negative moment regions.

The natural frequency of C-N was found to increase from 2.54 cycles per second to 2.61 cycles per second. The damping was found to increase from 0.31 per cent to 0.51 per cent of critical.

The amount of clear cover in each pour was measured. The amount of clear cover ranged from a low of 0.94 inch to a high of 2.88 inches. The average amount of clear cover was found to be 1.77 inches. The average amount of clear cover was determined for each pour and this is shown in Figure 14. The amount of cover is plotted in such a manner as to reveal if there is any relationship between the depth of cover and the cracking intensity in each casting section. The specified depth of cover for this bridge is 1.5 inches.

There was nothing of significance noted in a review of the weather conditions at the time of casting and curing.

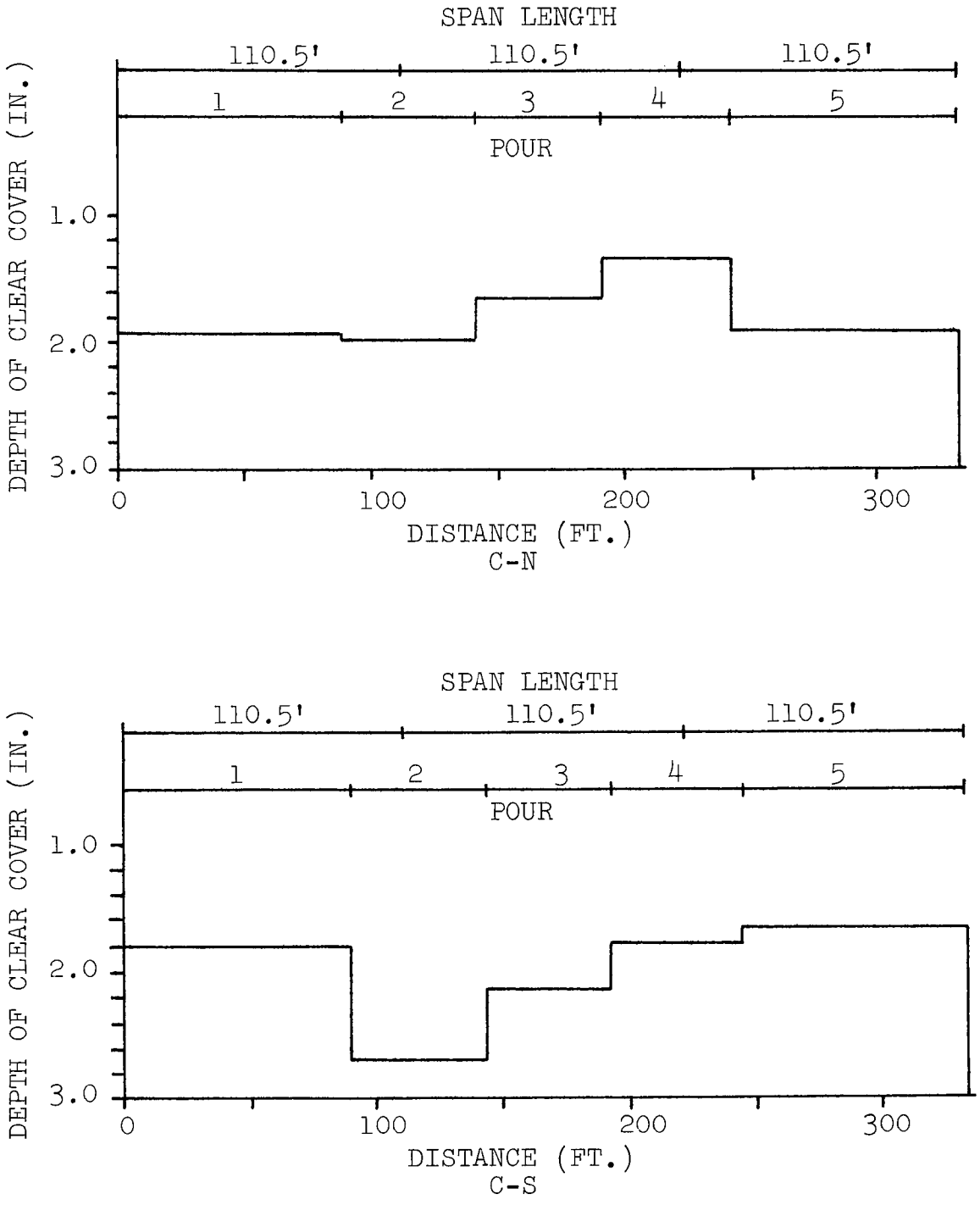


Figure 14. Depth of Clear Cover, Bridges C-N and C-S.

The maximum temperature reached above 80°F after the casting of sections two, three and four. The low temperature was seldom below 40°F and never lower than 34°F.

C-S was inspected for the first time approximately one month before the bridge was opened to traffic. At this time there were several long transverse cracks. These transverse cracks were concentrated at the interior end of the exterior pours and in one of the pours over the supports. There were no longitudinal cracks on the bridge. At the time of the second inspection, cracking had increased in all of the sections (Figure 6, page 25). The exterior sections were cracked on the interior ends, and the other sections were cracked fairly uniformly. The majority of cracks were six feet in length or greater and quite a few cracks were between three feet and six feet in length. The cracking intensity (Figure 11, page 31) increased almost uniformly over the length of the bridge except for a small increase in one negative moment region.

Several areas of scaling were found during the second inspection. The areas were small and randomly scattered over the bridge. It was possible to remove chips of concrete from the scaling area by rubbing the area with the edge of a shoe sole. This condition was not noticed on C-N.

The natural frequency of C-S decreased from 2.60 cycles

per second to 2.59 cycles per second. The damping decreased from 0.46 per cent to 0.33 per cent of critical.

The depth of cover was measured for each pour of C-S. The depth of cover varied from a low of 1.06 inches to a high of 3.17 inches. The average depth of clear cover was found to be 2.01 inches. The average depth of clear cover is shown for each section in Figure 14, page 50. The specified depth of clear cover for this bridge is 1.5 inches. There was nothing of significance noted in the weather data for this bridge.

Both C-N and C-S carry traffic over Interstate 40 in Cookeville. Most of the traffic consists of cars and light trucks.

N1-N and N1-S

At the time of the first inspection, transverse cracking on bridge N1-N was located primarily over the south support. Random cracks were found to be concentrated in the positive moment regions and particularly in the left traffic lane. Some longitudinal cracking was observed, but the length of these cracks was very short.

After N1-N had been opened to traffic for approximately nine months, a second inspection was made. It was found that the overall transverse cracking had not increased a great deal in regard to long cracks (Figure 7, page 26), and that random cracks were found to be spread more uniformly over the bridge deck. The increase in

the total length of cracks over six feet in length per ten foot section was due mainly to the lengthening of the existing cracks. The longitudinal cracking increased slightly. Only one longitudinal crack over six feet in length was found. This crack was located over the first girder to the right of the longitudinal centerline when looking in the direction of traffic.

The natural frequency of N1-N was measured and found to decrease slightly from 4.54 cycles per second to 4.51 cycles per second. The damping was found to increase from 0.63 per cent to 1.39 per cent of critical.

The major form of cracking on N1-S at the time of the first inspection was transverse cracking. The transverse cracks were primarily located in the negative moment regions and the positive moment regions of the center span. Some random and shrinkage cracks were found but no longitudinal cracks were found.

After nine months of normal traffic, the transverse cracking had increased mainly by the extension of the original cracks. Several transverse cracks that were originally less than six feet in length had increased in length to six feet or greater. Two small longitudinal were noted over the longitudinal reinforcement over two girders. These cracks were less than six feet in length.

The natural frequency of N1-S was measured and found to decrease from 4.54 cycles per second to 4.20 cycles per

second. The damping was found to increase from 0.67 per cent to 0.93 per cent of critical.

Traffic using these bridges consists of heavy truck and automobile traffic. Both bridges were used during the construction period. A concrete batch plant was located near the bridges. N1-S was used for the major part of the hauling of the concrete.

3. Relationship between Cracking Intensity and Depth of Cover

An attempt was made to correlate the cracking intensity of a pour with the depth of clear cover for that pour. Four bridges were selected and these were H-E, H-W, C-N, and C-S. The average depth of clear cover was determined for each pour on these bridges and is shown in Figure 13, page 37, for H-E and H-W, and in Figure 14, page 50, for C-N and C-S. It was assumed that the pours with shallow cover would have the highest cracking intensities and for this reason the depth of clear cover was plotted on an inverted scale (3.0 inches to 0.0 inches). If the depth of clear cover has this relationship to the cracking intensity, the shape of the depth of clear cover graph should have the same shape as the cracking intensity graphs (Figure 8, page 28 for H-E and H-W, Figure 11, page 31 for C-N and C-S). The expected positive correlation was not found and therefore cracking intensity is

primarily a function of other influences rather than depth of cover.

IV. DISCUSSION OF RESULTS

1. Bridges Subjected to Construction Traffic

H-E was found to be badly cracked in certain areas on both inspections. The section with the highest cracking intensity was in the positive moment region of the last pour. The weather date for this pour shows no adverse conditions during the time of curing. The amount of clear cover was measured and found to average 1.67 inches and the smallest amount of clear cover measured was 1.52 inches, which is greater than the 1.5 inches specified. The cracking intensity increased almost uniformly over the entire length of H-E (Figure 8, page 28). During this study, it was noted that H-W was not cracked as badly as H-E (Figure 2, page 21). H-E had the greatest cracking intensity of these two bridges and H-E was also used for most of the construction traffic.

M-W was found to be cracked greater than M-E and since both bridges were used in the hauling of concrete from a nearby batch plant, it would seem that both bridges would be cracked almost equally. It has been previously stated that some factors influencing the cracking of concrete include low humidity and windy weather.[13] Both of these bridges had average wind velocities at the time of casting of above six miles per hour, however, the relative

humidity was low on the afternoon when M-W was cast. Because of the low humidity, water in the concrete will evaporate quicker than it can be replaced by bleeding water and the surface dries out.[13] On the afternoon of casting of M-E, the relative humidity was high and retarded evaporation of the water in the concrete. M-W has many small shrinkage cracks and M-E does not. It has also been shown that inadequate clear cover increases the possibility of shrinkage cracks over the reinforcement.[6] The average amount of clear cover measured on M-W was 1.13 inches and the average amount of clear cover on M-E was found to be 1.62 inches. Since M-W had such shallow cover (often less than 0.75 inch), it is possible that inadequate cover along with the weather conditions present contributed to the many shrinkage cracks found on M-W.

The many popouts on both M-E and M-W can be traced to the use of chert gravels in the aggregate used in the concrete. A study of chert gravels in the Memphis area conducted by the Tennessee Highway Research program indicated that the popouts in concrete in the area were due to the freezing and thawing of chert gravels used in the aggregate.[18] Nine gravel pits were investigated and one of those pits, Fischer's Glasgow Pit 449, was the source of the aggregate used in the construction of M-E and M-W.

The large amount of cracking on N2-N and N2-S may be attributed to the early use of these bridges by construction traffic particularly N2-S. N2-S was more severely cracked

than N2-N and was used in the hauling of most of the roadway fill material over the two bridges. Large earthmoving equipment (Caterpillar Payhauler) were allowed to use the bridge providing it was loaded only half full. It was found to weigh 89,600 pounds when half full. The speed was limited to a maximum of 15 miles per hour and since it was reported that the contractor was cautioned about this several times it is probable that these requirements were exceeded. The early use of the bridges coupled with the heavy loads no doubt caused the extensive cracking of the bridge deck.

2. Bridges Subjected to Normal Traffic

Transverse cracking increased on CO-E and CO-W in the nine months following the opening of the bridges to traffic and this can be seen in Figure 5, page 24. The cracking intensities increased for both bridges, particularly in the negative moment regions (Figure 10, page 30). In each of the negative moment regions, the cracking intensity at least doubled while the cracking intensities of the positive moment regions did not increase as much.

Longitudinal cracking increased considerably on CO-E and CO-W during the study period. However, the worst cracking appeared to have been caused by construction traffic previous to the period of the present study. An approximate analysis was carried out in the same manner as for N2-N

and N2-S. It was found that the maximum moment in the slab occurred when a vehicle straddled the center girder. This approximate moment was twice as great as the maximum moment on N2-S. When vehicles are in their natural traffic lanes, it was found that the moment in the slab was reduced by half. Since construction traffic usually travels the center of a bridge it is assumed that the longitudinal cracking was started by the construction vehicles and then the cracking was propagated by the traffic after opening to normal traffic. From a comparison of the initial cracking intensities (Figure 10, page 30), it appears that construction traffic used both bridges about equally.

C-N was initially inspected almost eleven months before it was opened to traffic and therefore the increase in cracking is due to construction traffic as well as normal traffic. From Figure 11, page 31, it is seen that there was very little cracking initially and that the cracking intensity increased the most in the negative moment regions. The cracking intensity of C-S increased almost uniformly except for one negative moment region which only had a small increase. The increase in the cracking on C-S was due mostly to normal traffic since the bridge was inspected shortly before it was opened to normal traffic. The initial inspection of C-S shows that there was quite a lot of cracking caused by the construction traffic.

Although N1-N and N1-S were studied with regard to

normal traffic, it is evident that the construction traffic was responsible for most of the cracking prior to the bridge being opened to traffic. From Figure 12, page 32, it is seen that the cracking intensities for N1-S were much higher than for N1-N. The construction traffic was responsible for most of the cracking on N1-S. It was mentioned earlier that the construction traffic used N1-S more than N1-N and this is evident when looking at the cracking intensities of the two bridges. Because of the heavy vehicular traffic on these bridges, it would seem that the normal traffic would have had more of an effect on the cracking intensities of the two bridges. The cracking intensities on N1-N did not increase very much and it seems that the increase should have been greater since there was not very much cracking on the first inspection. This brings up an interesting point. Is there some maximum level of cracking intensity at which increased traffic loading causes very little effect? From the data gathered during this study, it is evident that there is not enough information to determine if such a level exists.

3. Relationship between Cracking Intensity and Depth of Cover

It was found that for only one bridge did the comparison between cracking intensity and the depth of cover compare reasonably well. This was C-S which was similar except for the fifth pour. C-N did not match as well since the largest cracking intensity occurred in a

pour with a depth of cover of over two inches. H-E and H-W did not show agreement except that the highest cracking intensities occurred in pours with clear cover less than the surrounding pours.

4. Relationship between Natural Frequency and Cracking Intensity

The measured natural frequency for each bridge was compared to the highest cracking intensity of that bridge and this was done for all the bridges in this study with the exception of M-E and M-W for which a cracking intensity was not defined. This was done in an attempt to establish a relationship between the natural frequency with the cracking intensity of the bridge. The bridges were ranked from the highest natural frequency to the lowest natural frequency. The highest cracking intensity for each bridge was plotted in the same order as the bridges with regards to the natural frequency as shown in Figure 15.

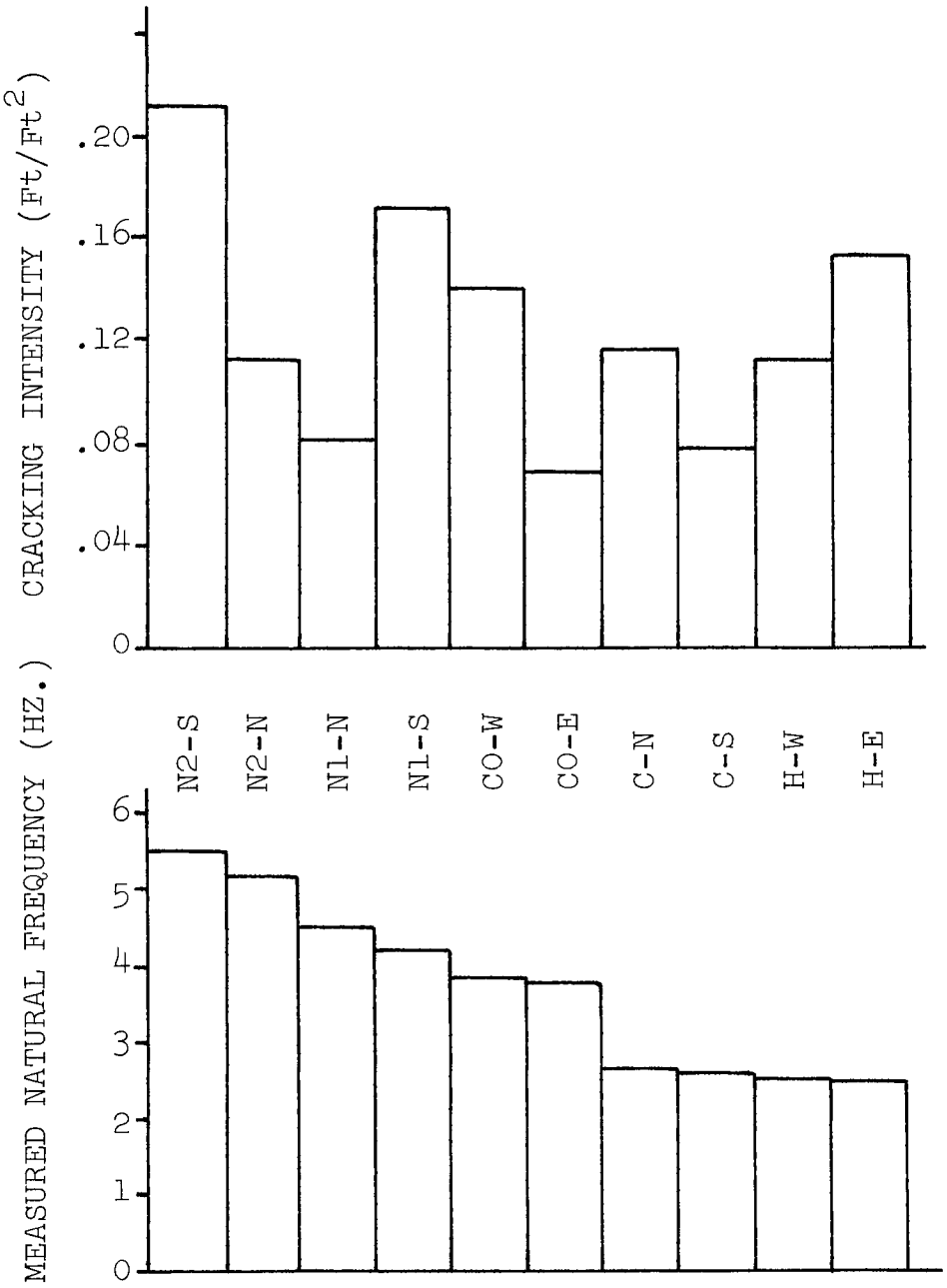


Figure 15. Comparison of Transverse Cracking Intensity and Natural Frequency.

V. CONCLUSIONS

1. Effect of Construction Traffic

One of the objectives of this study was to determine if the type of traffic influenced the severity of the cracking of the concrete bridge deck. During the course of this study, it became apparent that when two bridges were inspected, the one used by the construction traffic was more severely cracked than the other. Sometimes both bridges were used during construction and the one used most was more severely cracked. H-E was cracked more than H-W (Figure 8, page 28) and was used for the majority of the construction traffic. This is also true for N1-S and N1-N, and for N2-S and N2-N. It was mentioned previously that the construction traffic on CO-E and CO-W is thought to have started the longitudinal cracking of these bridges. The construction traffic did not cause a great deal of cracking in the transverse direction, but the position of the vehicles on the bridge had an effect on the longitudinal cracking as mentioned earlier in the approximate analysis.

Many times the maximum load a bridge is expected to resist is met or surpassed during the construction phase. It is possible for heavy earth moving equipment to exceed the design loads when transporting fill material. Earth-moving equipment on N2-N and N2-S was found to weigh 89,600 pounds when half full while the standard HS20-44 truck is

limited to 76,000 pounds. Since bridges are used before the roadway is fully in place, it is necessary to provide ramps at the ends of the bridges. These ramps provide an increase in the impact factor and this combined with speed and heavy loads can cause very high stresses. To prevent this occurrence, bridges should not be used in the hauling of fill material. If this is not possible because of the limitations of the job site extreme care must be taken to ensure that the bridges are not overloaded in any manner. More research is needed in the area of construction loads.

2. Effect of Normal Traffic

Bridges subjected to normal traffic were expected to crack more extensively in the negative moment regions between inspections than in the positive moment regions and this was found to be the case except for one case on C-S. The cracking intensities of the negative moment regions should increase since the predominate stress in these regions is tension. Even though the slabs were noncomposite in these regions some tension is developed by the adhesion of the concrete and the steel. The positive moment regions increase in cracking intensity to a lesser degree since the predominate stress is compression. It seemed that most of the cracking increase was the lengthening of earlier cracks. Most of the cracking intensities show that after traffic the shape of the graph is similar to the graph of the cracking

intensity before opening to traffic. The failure planes for the cracks may be set early in the life of a bridge and the traffic causes the cracks to extend.

3. Relationship between Cracking Intensity and Depth of Cover

Since the comparisons between depth of cover and the cracking intensity did not compare favorably, it cannot be said that clear cover is directly related to the cracking intensity, however, there may be other factors such as the curing of the slab and various construction practices that when combined with the depth of clear cover may have considerable influence on the cracking intensity.

4. Relationship between Natural Frequency and Cracking Intensity

It was hoped to find a relationship between the natural frequency and the cracking intensity of the bridges under study, however, from Figure 15, page 62 it can be seen that no definite relationship exists. Bridges with higher natural frequencies did not necessarily have the lower cracking intensities or vice versa. If the natural frequency had a direct effect on the cracking intensity, the cracking intensities would be either increasing or decreasing similar to the natural frequencies. It can be seen from Figure 15, page 62 that there is no such

relationship. It is evident that something other than the natural frequency plays a dominate role in the cracking of concrete bridge decks.

5. Quality Control

Several of the conditions noted in this study can be corrected by increasing quality control during construction. The clear cover varied greatly on several bridges. If the top of the slab is assumed to be a smooth surface, it is evident that the mat of reinforcing steel is not supported adequately since the steel must sag to vary the amount of clear cover. Earlier studies have shown that the mat of reinforcing steel is often supported by chunks of concrete or cinder blocks. The thickness of these materials can vary greatly and their use should be discontinued since adequate cover is not assured and there is no assurance that the cover will not be excessive. The mat of steel should be adequately supported so that excessive deflections do not occur.

Popouts can also be controlled by quality control. The use of chert gravel in concrete should be discontinued since the chert gravel is the cause of the popout. This may not be possible in West Tennessee since chert gravel is the predominate form of aggregate in that area, however, chert gravel could be eliminated in the other areas of the state where there are more forms of aggregate. The popouts

reduce the amount of clear cover over the reinforcement and increase the possibility of further deterioration by water and de-icing chemicals.

VI. RELATION TO PREVIOUS RESEARCH

The major form of deterioration in this study was found to be transverse cracking. Previous research[4,5,6] has shown that transverse cracking occurs over the transverse reinforcement and this was found to be the case on the bridges studied.

Spalling is considered to be the most troublesome type of bridge deck deterioration.[1] In this study, there were no instances of spalling found. Since new bridges were selected for this study, it is possible that spalling either had not started or had not progressed to a stage at which it would be visible. Since transverse cracking is usually the forerunner of spalling, it is possible that spalling may be a problem in the future.

Previous research has shown that scaling is also a problem enccuntered a great deal. In this study, scaling was found on only one bridge (C-S) and this was not severe. It is probable that these areas were caused by specific batches of concrete since the areas were scattered.

It was mentioned previously that to insure a minimum clear cover of two inches, it was necessary to specify a clear cover of 2 3/8 inches.[1] From the results of this study, it seems that a minimum clear cover greater than 1 1/2 inches must be specified to insure a clear cover of 1 1/2 inches. Several measurements in the same pour varied

by more than the 1 1/2 inches specified as the clear cover. Many of the measurements made in this study were less than 3/4 inch. Measurements were also made that were nearly three inches in clear cover. If it is assumed that the concrete bridge deck is level and smooth, it is apparent that there is quite a lot of variation in the height of the reinforcing steel. If this is the case, the reinforcing steel is inadequately supported before the concrete is poured. In the observing of construction of bridge decks,[17] it has been found that the reinforcing steel is supported by pieces of concrete. These pieces of concrete can be dislodged by the construction crews walking on the reinforcing steel. This will cause the steel to sag and the depth of cover will increase and the deck will not be as strong structurally because of the decreased moment of inertia.

VII. FUTURE RESEARCH

Further research is needed in the field of bridge deck deterioration as has been shown by this study. The major form of deterioration was found to be transverse cracking. Some bridges increased in cracking intensity more than others and this makes one wonder if some maximum value of cracking intensity is approached when cracking does not increase significantly. The bridges in this study should be inspected several more times to determine if this is the case. Since many bridges are now being overlayed it would be interesting to compare the progress of cracking intensity and overall deterioration of bridges with overlays with the bridges in this study.

This study did not find a correlationship between cracking intensity and depth of clear cover, however, this study showed that more control is needed in insuring that the proper depth of cover is obtained. More research is needed in the correlation of the cracking intensity and the depth of clear cover since this study did not present enough evidence to positively state that no relation exists.

More research is needed in regard to the effect of normal traffic on the bridges studied in regard to construction traffic. Another inspection of these bridges

would be helpful in relating the effects of the two types of traffic.

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