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USE OF SHRINKAGE COMPENSATING CEMENT IN BRIDGE DECKS

Submitted to

Highway Research Center
Auburn University

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Final Report

on

Highway Research Center Research Project

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FOREWORD

This research was supported by the Highway Research Center (HRC) and Civil Engineering Department at Auburn University, and by Blue Circle Cement Company. The PIs are grateful to these sponsors for their support of this work.

The PIs are particularly grateful to Mr. Ed Russell and Mr. John Jenkins of Blue Circle who provided technical and material support throughout the project, and to Mr. Oscar Harper of Master Builders Technologies for supplying the concrete admixtures used in the project. Also, special thanks are due to Mr. David Ransberry and Mr. Michael Phillips of the Ohio Turnpike Commission, Mr. Jim Golden of the New York Thruway Authority, and Dr. Henry Russell formerly of the Construction Technology Laboratories for generously sharing their knowledge and experiences with shrinkage-compensating concrete bridge decks.

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1. INTRODUCTION

1.1 Statement of Problem

The penetration of water into concrete over time can result in such deterioration mechanisms as freezing and thawing, wetting and drying, rebar corrosion, leaching, sulfate attack, and alkali-aggregate reaction. Cracks, poorly sealed joints, and concrete permeability are the primary sources of water penetration into and through highway bridge decks. A deck with superior water shedding and water resistance characteristics would prove superior in durability performance for all of the major bridge components.

Results of a recent survey (20) indicated that shrinkage is the leading cause of deck cracking. Hence, we have the following situation.

1. Water penetration of bridge decks is a, if not the, leading cause of premature deterioration or reduced durability.
2. Deck cracking is a primary source of water penetration (along with poorly sealed joints and concrete permeability).
3. Shrinkage is the leading cause of deck cracking.

This being the case, along with the fact that shrinkage-compensating concrete (SCC) is known to substantially reduce shrinkage cracking, it makes sense that we closely examine SCC concrete as a possible material for bridge decks. This material has the potential to significantly reduce deck cracking, and thus reduce water penetration, and thus enhance durability. This was the impetus for this research.

1.2 Objectives

The overall objective of this project was to assess the practicality and viability of using Type K cement to increase the service life/durability of bridge decks in Alabama. To achieve this objective, specific subobjectives of the project were as follows:

1) Conduct a review of what has been done in the past with SCC concrete and its applications to bridge decks, and determine the results and recommendations of these works.

2) Conduct laboratory testing to identify good SCC concrete mix designs and assess their robustness/sensitivity to environmental conditions and construction quality and compare with ordinary PC concrete mixes.

3) Plan and coordinate with ALDOT for the construction of a test bridge using SCC concrete for its deck for 1-span and a comparison span using ordinary PC concrete.

1.3 Scope

This investigation was limited to a literature review, telephone inquires, and personal visits with people and agencies having good expertise and experience with SCC and its applications to bridge decks. Additionally, a mix design laboratory testing program was executed to identify a good SCC and SCC with microsilica for use for bridge decks in Alabama. No implementation in the form of a field/test bridge testing program was undertaken in this work. It is our hope that such a testing program will follow this study in the near future.

2. BACKGROUND AND LITERATURE REVIEW

2.1 General

Cracking is a major factor that leads to premature deck deterioration via providing a pathway for the entry of moisture. This is particularly a problem in the northern U.S. where the moisture carries chloride ions from deicing salts, and where freeze/thaw is a problem. However, even in Alabama the alternate wetting/drying causes alternate swelling/contracting and microcracking and leads to premature deterioration of the concrete. Also, it may cause corrosion of rebar and in the Gulf coast region, this can be a severe problem. Additionally, early deck thermal and shrinkage cracking serve as locations of stress concentrations and crack propagations. These in turn reduce the fatigue life of the bridge deck.

Ten essential steps for producing quality concrete bridge decks are:

1. Use quality concrete ingredients.
2. Have proper blending of ingredients or mix design (including additives if appropriate).
3. Have proper air content (around 5%).
4. Have proper mixing and delivery.
5. Have proper form work and falsework.
6. Have proper rebar protective coating (if applicable).
7. Have proper rebar reinforcement, cover, and placement.
8. Have proper concrete placement conditions (environmental) and methods.
9. Have proper consolidation and finishing.
10. Have proper curing.

Parameters and characteristics which most strongly affect concrete deck durability are:

- crack mitigation
- concrete permeability

- rebar corrosion
- concrete fatigue

To achieve quality in-place concrete decks, good concrete workability must be designed into the mix. If it is not, what typically happens is (1) the contractor will achieve the good workability by adding water, or (2) the in-place concrete will be improperly distributed, consolidated, and finished.

Both of these result in concrete which is inferior to that planned in quality and durability.

2.2 Concrete Deck Cracking

The literature on cracking of concrete bridge decks is extensive. A brief summarization of the literature and what we have learned from the experiences of others regarding the cracking of concrete bridge decks is presented below.

1. Concrete decks contain a variety of internal flaws and cracks which form during the hydration process and from thermal expansions and contractions. This internal cracking becomes more extensive during drying shrinkage, and actions to mitigate this cracking include minimize the quantity of cement, reduce the w/c ratio, employ quality moist curing, and use SCC. The internal cracking also becomes more extensive with construction worker movement on adjacent continuous deck rebar and on the "green" concrete. Hence, construction activities which significantly increase early concrete deck microcracking should be investigated and actions necessary to mitigate such damages identified.

2. Many state DOTs are concerned that many of their newer bridge decks exhibited cracks within three months after construction.

3. Several state DOTs have indicated that their greatest deck cracking problems occur on their thinner decks, and on their more flexible bridge deck-girder systems (typically thin decks on steel girders). In each case the DOTs, are planning to increase their deck thickness requirements in an attempt to mitigate deck cracking. Some of the advantages and disadvantages of extra deck thickness are:

Advantages	Disadvantages
Greater deck strength, i.e., greater shear and moment capacities.	Greater support girder strength requirements.
Reduced fatigue stresses and problems.	Greater foundation support requirements.
Smaller deflections.	Greater initial cost.

Advantages	Disadvantages
Less cracking and thus less water penetration.	
Allows greater rebar cover (if needed or desired).	
Superior vibration characteristics.	
Reduced sensitivity to construction tolerances.	
Greater robustness and durability.	
Fewer construction problems.	
Probable lower life-cycle-cost.	

4. Results of personal interviews with experts throughout the U.S. and Canada on the leading causes of deck cracking, and actions they recommend to mitigate cracking are shown below.

   a. Leading causes of cracking

      - Shrinkage (the leading cause)
      - Placement sequence
      - Traffic
      - Deck flexibility
      - Rich cement mix

   b. Actions to mitigate cracking

      - Type K cement
      - waterproofing system
      - tighter construction QC
      - fiber-reinforced overlays
      - water-reducing admixtures
      - modified placement sequence

- tighter construction specs
- thicker decks
- aspect ratio between 0.5 to 2.0

5. Bridge deck cracking is commonly categorized by orientation as transverse, longitudinal, diagonal, map and random. Factors associated with each of these are as follows:

    Transverse cracks are typically caused by flexure, shrinkage, subsidence, inadequate rebar cover. While they may be found at any location throughout the bridge deck, they are more often found over transverse members or transverse reinforcement, in negative moment regions and within the interior portion of long spans. Transverse cracks are the most prevalent deck cracks.

    Longitudinal cracks are often caused by wide girder spacing that can be attributed to transverse flexure of thin deck slabs. Occasionally, on wide decks, shrinkage cracks may be oriented in the longitudinal direction. Differential flexure of longitudinal girders often causes cracking parallel to the girders, or longitudinal cracks.

    Diagonal cracks are often caused by pressure on deck corners and are more common on skewed decks. This type crack may signal a more serious problem such as differential movement of the structure.

    Map cracks are similar to the cracks produced by the hot sun on dried mud flats. Map cracks occur in concrete when the surface dries rapidly. These cracks are normally not very deep.

    Random cracks may be caused by more than one problem; however deck finishing is a common cause. Random cracks are often caused by working the plastic concrete after the initial set has begun.

6. Factors identified by Penn DOT as having a significant influence on bridge deck cracking are as follows:

    - Deck placement sequence, finishing, and construction loads;
    - Traffic-induced vibrations applied at different stages of concrete curing;
    - Structural flexibility of the decks;
    - Residual stress due to deck placement sequence;
    - Effects of admixtures and materials compatibility;
    - Environmental controls and curing;

- Bridge type;
- Aspect ratio and deck skew;
- Yielding of epoxy coatings of reinforcing bars to influence crack growth;
- Shrinkage-compensating cement;
- Freeze-thaw damage;
- Water-cement ratio;
- Expansive aggregates;
- Characteristics of cements;
- Thermal stresses;
- Subsidence;
- Fiber reinforcement.

7. Results of a Penn DOT sponsored survey of all 50 states regarding the causes of bridge deck cracking are listed below.

    - improper curing
    - thin cover
    - high temperature
    - high evaporation rate
    - excessive deflection
    - lack of retarder
    - early release of falsework
    - continuous spans
    - skewed decks
    - traffic induced vibrations
    - wooden forms
    - heat curing

- construction quality control
- long spans
- flexible spans

8. Based on our previous research on enhancing concrete bridge durability (23, 24), the primary causes of early cracking, and premature structural cracking and deterioration for bridge decks in Alabama are:

   - early drying shrinkage
   - early thermal loadings
   - heavy trucks and large numbers of truck loadings

These are shown summarized in Table 2.1 along with a listing of potential corrective actions and the ALDOT bureaus responsible for the corrective actions identified. Employment of all of the potential corrective actions is not necessary or appropriate; however employment of just one of the actions is probably not sufficient or appropriate.

Table 2.1
Primary Types and Causes of Bridge Deck Cracks and Potential Corrective Actions

Primary Types of Cracks	Primary Causes of Cracks	Potential Corrective Actions	Responsible ALDOT Bureau[1]
Significant structural cracking in both directions (cracks approximately 12" o.c. in both directions in top and bottom of many interstate decks in Birmingham)	Heavy trucks, large number of truck loadings, and considerable superstructure flexibility and deflections.	1. Design for heavier trucks, e.g., HS25 and greater number of truck loadings (reduce allowable stresses because of fatigue).	B
		2. Increase deck thickness, e.g., to 8".	B
		3. Increase deck concrete strength to f'_c = 4500 psi.	B
		4. Stagger rebars so as not to create vertical planes of weakness in the deck.	B
		5. Use more and/or stiffer support girders, and reduce LL deflections.	B
		6. Use crushed aggregate to improve tensile strength, toughness and fatigue strength.	MT
		7. Improve inspection and other QA procedures to assure proper deck thickness and rebar cover.	C
Significant drying shrinkage and thermal cracking (primarily in transverse direction) before deck is opened to traffic	Early drying and thermal shrinkage	1. Use concrete mix with lower water content to minimize shrinkage. This may require use of water reducers for workability.	MT
		2. Use concrete mix with lower heat of hydration build-up via using Type II cement, and/or mixes with more pozzolans (fly ash, ground blast furnace slag, or silica fume) and less cement.	MT
		3. Examine the use of shrinkage compensating cement with microsilica to minimize shrinkage cracking.	MT

Table 2.1- (Continued)

Primary Types of Cracks	Primary Causes of Cracks	Potential Corrective Actions	Responsible ALDOT Bureau[1]
		4. Require wet burlap curing applied very early to minimize both early drying shrinkage and thermal cracking.	MT
		5. Require immediate (after surface finishing) surface evaporation protection with CONFILM, or equivalent, when T > 80°F and/or wind > 10 mph to prevent excessive evaporation before emplacing wet burlap curing.	MT
		6. Eliminate the use of north Alabama chert aggregates, and use crushed aggregate or aggregate with a certain number/percent of angular faces.	MT
		7. Establish limits on acceptable during shrinkage characteristics and thermal expansion properties of concrete mixes for decks.	MT
		8. Examine concrete mixes to maximize aggregate packing and minimize concrete absorption to gain maximum volume stability, impermeability and durability.	MT
		9. Use small and more closely spaced deck rebar, and a higher (than present) percent of distribution and shrinkage/temperature rebar.	B
		10. Rearrange deck rebar to have longitudinal rebar on outside and transverse rebar on inside.	B
		11. Employ deck contraction joints where needed and appropriate.	B
		12. Examine concrete placement sequences for possible needed changes and ensure contractors follow the approved sequences.	B,C

Table 2.1- (Continued)

Primary Types of Cracks	Primary Causes of Cracks	Potential Corrective Actions	Responsible ALDOT Bureau[1]
		13. Require greater limitations on environmental conditions (ambient and concrete temperatures, humidity, and wind speed) at time of deck placement.	C
		14. Require a minimum rate of concrete placement for decks of 750 ft^2 per hour during hot weather ($T > 80°F$) conditions.	C
		15. Consider late afternoon or early evening placements of deck, and/or greater use of retarders for hot weather ($T > 80°F$) conditions.	C
		16. Limit construction worker movements and loadings on fresh and green concrete.	C
		17. Require greater water control in QA program for concrete from the batching plant to final finishing and curing of the deck.	C
		18. Improve inspection and other QA procedures to assure proper delivery, placement, consolidation, finishing, and curing of deck concrete.	C

[1] B = Bridge Bureau
MT = Materials & Tests Bureau
C = Construction Bureau

2.3 Shrinkage Compensating Concrete

The primary reason for considering shrinkage compensating concrete (SCC) for bridge decks is to reduce shrinkage cracking and thus to reduce water penetration and rebar corrosion, and thus enhance durability. However, there are several additional advantages of this material as well as some disadvantages relative to ordinary Portland cement (PC) concrete. The primary advantages and disadvantages of SCC are as follows:

Advantages	Disadvantages
• Reduced shrinkage cracking. • Reduced cracking due to subsidence. • Reduced rebar corrosion and concrete deterioration due to cracking. • Increased bond strength and bonding with rebars. • Increased tensile strength and ultimate tensile strain, resulting in reduced micro and macro cracking and in turn reduced water penetration. • Reduced surface capillaries and surface porosity due to reduced bleed water. • Harder and more self-sealing surface with increased strength and wear/abrasion resistance. • Requires fewer construction and contraction/control joints which will result in less water penetration. • Improved workability and easier concrete placement because of high slump. • Easier concrete finishing because of little bleed water.	• Reduced concrete density, and probably greater porosity and permeability. • Water-reducer additives make it more sensitive to time, temperature and humidity conditions at time of placing. • Greater sensitivity to curing conditions, and demanding of quality curing. • Mechanical properties in the direction of deck thickness are probably inferior. • Excessive unrestrained expansion could break the bond with coarse aggregate and destroy the integrity. • Reduced time of set and greater rate of slump loss. • Higher initial cost.

The Advantage listing above is quite impressive. However, it is not clear whether the advantages will outweigh the disadvantages and provide a more durable bridge deck. Let us examine the Disadvantage listing in a little greater detail.

1. Reduced Density. The expansive nature of SCC renders it less dense than PC concrete. However, with the forming of the ettringite in SCC, it is unclear if its porosity and permeability will be worse or better than PC concrete. It is anticipated that it will be worse. However, even if this is the case, its reduced surface capillaries and more self-sealing surface, along with its reduced cracking, may render decks of SCC more resistant to water penetration than PC concrete decks.

2. Construction Difficulties from Additives. Use of water-reducers as additives renders SCC more sensitive to mixing, handling and finishing time. This can cause some construction problems, and in turn some in-place concrete quality problems. However, the positive aspect of using water reducers is that good concrete workability (i.e., with a slump of approximately 6 in.) will be achieved during placement. This effect is beneficial, because good workability is essential to achieving good quality in-place concrete. With PC concrete, good workability is often not achieved in the mixture proportioning process, and what typically happens is:

- the contractor achieves good workability on-site by adding water, which reduces the strength and durability of the hardened concrete;

- the in-place concrete is improperly placed, consolidated, and finished.

3. Greater Sensitivity to Curing. A key to successful applications of SCC is to achieve the proper moist curing expansion (see Figure 2.1) in order to achieve a proper shrinkage-compensated and crack free product. Early initiation of the curing process, which needs to be a continuous and quality curing, is required. This may require early fog-spraying (most of the expansion takes place in first 24-36 hours), and maintaining a wet burlap covering for 7 days. The curing demands for SCC are somewhat greater than those for PC concrete. However, it should be noted that even with PC concrete, proper curing is an area where greater attention during construction is essential for achieving good quality and durable concrete.

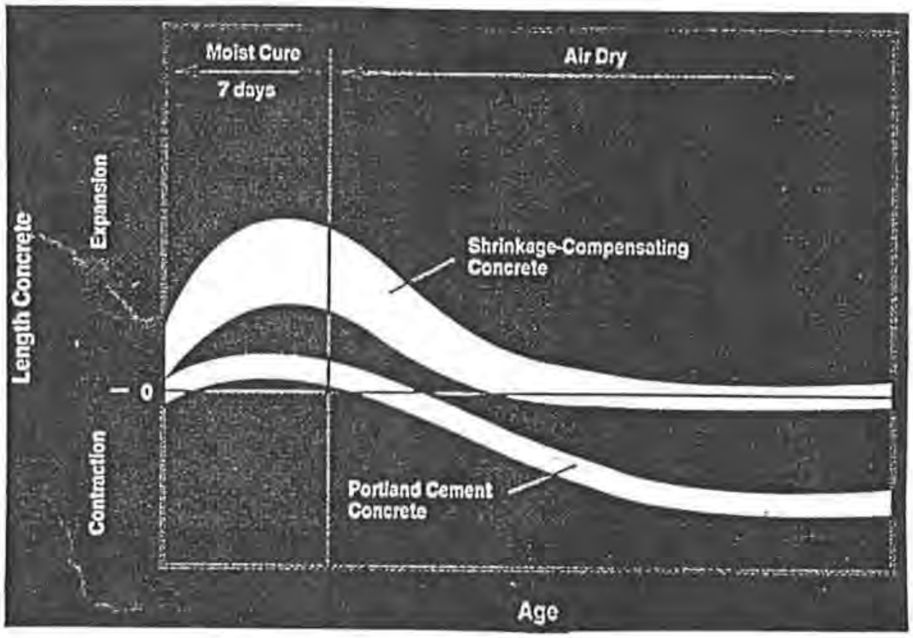


Figure 2.1. Typical Length Change Characteristics of Shrinkage Compensating and Portland Cement Concretes

4. Mechanical Properties in Direction of Deck Thickness. Bertero (7) reports that for thin surface components such as decks that are restrained (by steel rebar) biaxially only in their plane, that the mechanical properties of the SCC in the direction of the thickness of the deck may be inferior to those in the in-plane directions. This is probably the case, but it is also probably not very important. For example, the mechanical properties of plywood are inferior in the direction of its thickness, but this does not prevent it from being an excellent decking material.

5. Excessive Expansion Destroying Integrity. Bertero (7) reports that the magnitude of transverse (unrestrained directions in uniaxial restrained specimens) expansions can be so great that bond between coarse aggregates and the expansive-cement concrete

mortar matrix will be broken and voids between these two components of the expansive-cement concrete will be visible in cut sections. The existence of these voids can jeopardize soundness of the concrete. This must be prevented by using the proper amount of expansive cement (Type K cement) and having a proper mix design. Ohio and other states using expansive cement in bridge decks do not report any soundness problems with SCC.

6. Higher Initial Cost. Whereas the initial concrete cost will be typically 10 to 25 percent higher due to using Type K cement and water-reducers, it is anticipated that the life-cycle-cost of SCC should be lower due to reduced cracking and subsequent lower maintenance cost and greater bridge durability/longevity.

The literature on SCC is fairly extensive and other information extracted from the literature which is pertinent to our investigation is briefly summarized below.

1. Expansive cement - A cement which when mixed with water forms a paste that, during and after setting and hardening, increases significantly in volume. Expansive cements are Types K, M, or S. Type K has been available commercially since 1963. Type S was developed by PCA and became available commercially in 1968. Application of Type K have been primarily restricted to production of shrinkage-compensating concrete. Type M is not commercially available in the U.S.

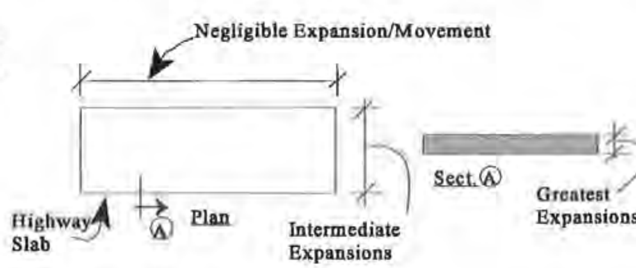
2. Expansive cement concrete is made with Type K, Type M or Type S expansive cement. Its objective is to minimize cracks caused by drying shrinkage. To induce compressive stresses (-25 psi < $\sigma_{CONCRETE}$ < -100 psi and + 5000 psi < σ_{STEEL} < + 15,000 psi), shrinkage-compensating concretes must be restrained via internal steel rebar, subgrade friction, forms, adjacent structures, or other means.

3. There are two basic formulations of expansive cement

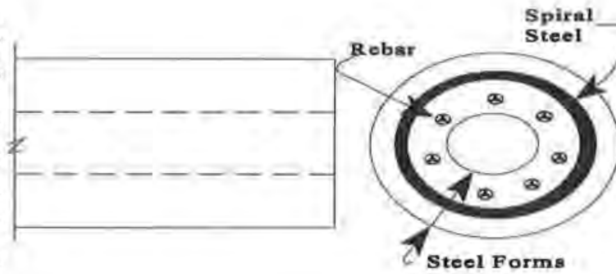
   - The first formulation is proportioned to expand and compensate for shrinkage, and thus eliminates shrinkage cracks. This formulation is referenced to as "shrinkage-compensating cement."

   - The second formulation is proportioned to expand more and have a "net" expansion when it hardens so as to prestress the embedded steel rebar. It produced concrete which prestresses itself and also mitigates cracking. This formulation is referred to as "self-stressing cement."

4. Some California test results with SCC are as shown at the right.



5. For pipe application, we can get beneficial effects of significant expansions of expansive cement, i.e., to a level where we are using "self-stressing cement" (see figure at right). It should be noted that this is a common application in the old USSR.



6. In the absence of restrain, expansive concrete exhibited free expansion up to 6%. If restrained, it developed compressive stresses in the concrete.

7. Unrestrained expansion affects compressive strength, and the greater the unrestrained expansion, the lower the compressive strength. However, if the expansive concrete is adequately restrained, high compressive strength can be attained.

8. A PCA study showed free expansion of several percent may be produced by expansive cement mixtures of Portland cement. The magnitude of expansion may be controlled by the composition of the mixture and by curing temperature. Expansion before shrinkage is a few percent. Later shrinkage is approximately 0.1-0.2% which is about the same as normal cement. A net expansion induces tension stress in the steel rebar, and if prevented, induces compressive stresses in the concrete.

9. Magnitudes of self-stress are affected by

    - amount of expansive component (the greater the expansive component, the greater the expansion and self-stress)

    - w/c ratio (the lower the w/c ratio, the greater the subsequent expansion)

    - curing conditions (early and continuous wet curing is essential for proper expansion)

    - percentage of steel used in restraining the expansion (the optimum percent of steel seems to be 1-2%).

10. Bertero (7) reported that successful use of expansive concrete in the fabrication of SCC prismatic members seems to require the restraint of the expansion in <u>all directions</u>. However, field results of bridge deck applications by the Ohio Turnpike Commission seem to contradict this.

11. A large amount of the chemical reaction which causes expansion in SCC takes place during the early age of the concrete (within the first 2 days).

12. A major part of ettringite (causes the expansion) must form after attainment of a certain degree of strength, otherwise the expansive forces will dissipate in

deformation of a still plastic or semi-plastic concrete and thus place no stress on the restraint provided.

13. Normal strength concrete can be obtained with shrinkage-compensated cement, and the strengths obtained show a direct relationship with the amount of Portland cement in the mix.

14. Some key attributes of shrinkage-compensating cement are:

    - shrinkage compensation
    - crack resistance
    - water resistance
    - wear resistant
    - chemical prestressing

15. The advantages of shrinkage - compensated cement can be obtained in the field without departure from standard engineering and construction techniques and practices.

16. The success of SCC is judged on its reduction of shrinkage cracks in the field environment and as constructed by conventional construction work forces. However, more R&D is needed on field related construction problems of

    - placement
    - early hardening/finishing
    - curing
    - temperature at placement

    as well as design problems of

    - percent of restraining steel
    - type cement and how much to use
    - sensitivity to w/c ratio
    - affects of admixtures.

17. Collepardi, etc. (9) indicated that the combination of a superplasticizer and an expansive agent may be more advantageous than the use of an expansive agent alone.

18. The upper limit of linear expansion for SCC appears to be around 2% without the concrete becoming unsound, i.e., disrupted.

19. Approximately 60-80% of the total expansion of SCC occurs in the first 24-36 hours.

20. Concrete made with Type K cement requires more water for hydration purposes than ordinary PC concrete. It normally requires a water reducer.

21. The ideal bridge deck would be (1) crack-free, (2) a low permeability concrete, and (3) joint-free or a minimum number of sealed joints. SCC allows improvements on (1) and (3) and perhaps on (2); however, additional testing of in place decks is needed to verify that Type K concrete does yield decreased permeability.

22. The performance of expansive cement in minimizing cracking depends in large measure on early curing i.e., on the timing and quality of the early curing.

23. It is recommended that the amount of expansion, which is as important as strength in the performance of shrinkage-compensating concrete, be determined by measuring the length change of 3 x 3 x 10 in. restrained concrete prisms (ASTM Test Method C878).

24. The water requirement of SCC is greater than that of PC concrete for a given consistency. The extra water is needed for the ettringite growth and the SCC expansion. As with PC concrete, the lower the w/c ratio, the greater compressive strength of SCC.

25. The tensile, flexural, compressive, and shear strength developments of SCC are very similar to those of PC concrete under similar situations as are the modulus of elasticity, Poisson's ratio, coefficient of thermal expansion, creep, and freeze-thaw durability.

26. The drying shrinkage volume change characteristics of SCC appear to be far superior to those of PC concrete. Also, the surface abrasion resistance of SCC is reported to be 30-40 percent greater than that of PC concrete.

27. The splitting tensile strength of SCC is reported to be $0.08\,f'_c < f'_t < 0.14f'_c$ and ACI states

$$f'_t = 7.5\sqrt{f'_c}$$

28. Crushed coarse aggregate with its greater angularity, surface area and surface roughness improves f'_t, impact strength, and fatigue strength more than it does f'_c. Thus crushed coarse aggregate should be used for bridge decks.

29. Air curing reduces f'_t more than it does f'_c and thus f'_c should not be used to judge the quality of curing if serviceability issues such as cracking are of concern. SCC requires quality wet curing, and PC concrete should be wet cured for good serviceability/durability.

30. Compaction and air content affect f'_c more than f'_t.

31. The shear strength of SCC is

$$f'_v \approx 0.20\ f'_c$$

32. Because of its improved tensile strength, slightly lower modulus of elasticity, and reduced macro and micro early cracking, the fatigue strength of SCC should be somewhat superior to that of PC concrete. The commonly used value (for PC concrete) of

$$f'_{fatigue} = 0.5\ f'_c \text{ (at } 10^7 \text{ cycles)}$$

seems appropriate for SCC. However, high traffic volume bridges currently are loaded with ADTT of 3000. For a 75 year service life, this translates to 82^M cycles of truck loadings. If one allows for a growth in ADTT of 100 percent, this would yield 164^M cycles during the bridge's service life. Hence, for SCC, appropriate limiting fatigue stresses may be

$$f'_{fatigue} = 0.5\ f'_c \text{ (for } 10^M \text{ cycles or low traffic volume)}$$

$$f'_{fatigue} = 0.45\ f'_c \text{ (for } 200^M \text{ cycles or high traffic volume)}$$

33. Because of its reduced shrinkage, the distance between construction joints and the length of single deck placements can be increased considerably with SCC. This coupled with its easier finishing are attractive construction features of SCC. However, for decks requiring multiple placements, checker boarded placement of SCC needs to be avoided unless special provisions are made to allow the SCC to expand during its early curing expansion phase.

34. No special construction equipment or techniques are required for satisfactory placement of SCC. However, SCC is much more sensitive to continuous wet curing which is initiated very early. Also, the rate of slump loss is greater for SCC and thus renders it more sensitive to delays at the job site when using ready-mixed concrete.

35. ACI Committee 345 Report 345R-91 on "Guide for Concrete Highway Bridge Deck Construction," states the following:

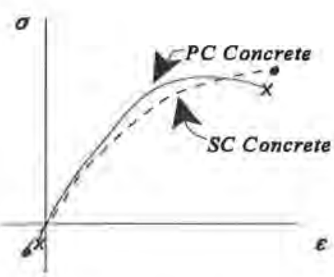
> Casting SCC decks to precast or prestressed girders and beams is to be avoided as this will present excessive external restraint against potential longitudinal expansion that will prevent the needed internal resilient steel strain required for the shrinkage- compensating action. Casting SCC decks to steel beams and girders has been more successful with appropriate potential concrete expansion levels attained.

36. Dr. Henry Russell, formerly of CTL and a renown expert on SCC, feels that the statement in No. 35 above is the result of the facts that Ohio and New York both experience excellent nonshrinkage cracking with SCC decks on steel girder bridges.

And, in the 1970s, California experimented with SCC decks on concrete girder bridges and reported no significant beneficial effects. Coupled with the fact that concrete girders are typically stiffer than steel girders, Dr. Russell feels that the experiences in California, Ohio, and New York led ACI Committee 345 to the statement in No. 35. Dr. Russell also indicated that consistent with the statement was the fact that in New York, only 2 SCC deck bridges did not perform superior in mitigating shrinkage cracking. These same 2 bridges were the only integral bridges (integral deck, superstructure and substructure and thus additional external restraint of the SCC deck) that were rehabilitated with SCC decks. The bridges were also skewed, but several other bridges were skewed and they performed well.

37. ACI 345R-91 also indicated that SCC can significantly reduce deck shrinkage cracking, has higher flexural tensile strength, and significantly higher abrasion resistance relative to PC concrete.

38. Dr. Henry Russell indicated that his research, as well as others', have shown improved tensile strength, greater tensile strain at failure, and somewhat reduced modulus of elasticity for SCC relative to PC concrete as illustrated in the figure below.



Qualitative σ-ε Curves for SC and PC Concretes

39. The improved tensile characteristics of SCC, i.e., f'_t, ε'_t and probable improved fatigue strength, should result in a concrete with reduced macro and micro cracking and superior fatigue resistance. In turn, the reduced cracking should result in reduced macro permeability, and this along with improved fatigue resistance should render the concrete more durable.

40. Shrinkage compensating cement was first developed in England, and was used there at a rate of about 400,000 tons/year shortly after its development. Currently its use in England is at a rate of about 100,000 tons/year.

41. Having been used since the late 1960s in the U.S., Type K cement is not a new or experimental material. Many specifications are available for the materials used for construction with Type K cement. Some of these specifications are ASTM C-845, ACI 223-83 *Standard Practice for the Use of Shrinkage-Compensating Concrete*, and the Department of the Army Corps of Engineers Manual EM1110-2-2000, dated September 25, 1985, Engineering and Design Standard Practice for Concrete.

42. No special equipment is required for placing and finishing concrete made with Type K cement. Generally speaking, normal, sound practices of concreting can be followed for mixing, placing, finishing, and curing the concrete.

43. Even though Type K cement is more expensive than Portland cement, Type K cement has several advantages that can mitigate these costs. For example, it is easier to place and finish Type K cement; therefore, the contractor can save labor costs in placing decks, especially those of 300 cubic yards or more.

44. With a few simple precautions, Type K concrete produces higher slump and cohesiveness--or "fat" characteristics. These characteristics provide good finishing qualities. However, because these characteristics make Type K cement different from other cements, owners and contractors must consider the following factors when using Type K cement:

- Fog-spraying should be available, especially for placement during hot weather, if plastic shrinkage cracking develops.

- Delaying placing activities at the job site should be avoided since shrinkage-compensating cement concrete loses slump faster than normal concrete.

- Concrete made with shrinkage-compensating cement will exhibit little or no bleed water; therefore, care must be taken to begin finishing operations at the proper time.

- Placement should be scheduled early in the day before temperatures rise; the deck should not be placed if the ambient temperature is above 80°F.

- Where possible, evening placement has advantages in that the greatest exothermic heat will occur during the cool nighttime ambient temperatures.

- Water curing should always be used with Type K concrete decks.

- A pre-construction and/or preplacement meeting should be held involving the inspectors, contractor, test lab, and ready-mix producer to discuss the above factors in detail before construction begins. This meeting is especially important if any of the participating "players" is inexperienced in using shrinkage-compensating cement.

45. Ohio Turnpike Commission - Have used SCC exclusively for their bridge decks since January 1985 and are extremely pleased with their performances. A more detailed description of their experiences and recommendations are given in Chapter 4.

46. Ohio DOT - Have emplaced some SCC decks, but were not pleased with the permeability properties, and currently do not use SCC.

47. New York Thruway Authority - Have emplaced some 30 plus SCC decks and were very pleased with the reduced early shrinkage cracking, but later experienced scaling and surface deteriorations. Construction Technology Laboratory (CTL), investigated

their decks and problem and determined that they did not have proper air content in the concrete and because of this, any concrete would have exhibited similar damage and deterioration. The Thruway Authority has more recently been pleased with a combination of microsilica and type K cement in a modified SCC mix. A more detailed description of their experiences and recommendations regarding SCC are given in Chapter 4.

48. New York DOT - Have emplaced 29 SCC bridge decks and are not pleased with their performance. They found the rapid chloride ion permeability of their SCC to be significantly inferior to that of PC concrete. They no longer use SCC. They are reported to be fast moving toward the use of HPC for bridge decks. They strongly believe in maximizing the "packing" of concrete for bridge decks, i.e., having good gradation of

- coarse aggregate
- fine aggregate
- cementitious materials (cement, fly ash, microsilica)

49. Most, if not all users of SCC concrete have found significantly reduced shrinkage cracking with SCC. Thus it appears that SCC bridge decks would have greater water and chloride ion penetration of the uncracked concrete regions, and less penetration via cracks in the deck.

2.4 Summary

In summary, the primary reason for considering Type K cement for bridge decks is to reduce early drying shrinkage cracking. In turn, this will reduce water penetration and rebar corrosion, and reduce crack propagation and growth due to truck/fatigue loadings. Both of these lead to premature deck replacement, but, it is the latter problem which primarily leads to deck nondurability in Alabama.

Considerable results of the effectiveness of using Type K concrete in bridge deck concrete have been documented, and some conclusions that can be drawn from past studies are:

1. Transverse shrinkage cracking has occurred in bridge decks in many states using mix designs without Type K cement.
2. Not only are these cracks unsightly, they are also detrimental to the deck's endurance and performance.
3. Type K cement concrete in bridge decks has proven it virtually eliminates transverse cracking when properly cured.

4. Type K cement concrete has been used successfully by ready-mix producers and contractors when a few simple procedures are followed.

5. Type K cement concrete may be more or less permeable than concrete made with other cement and the same mix design. Further study and research are needed to determine the effects of Type K cement on the concrete's permeability and performance with special mix designs.

Specifying bridge deck concrete based only on strength is not sufficient to attain overall quality and long term durability. Type K cement offers the potential for enhanced deck durability over ordinary Portland cement via the following benefits:

- reduced shrinkage cracking due to expansion/shrinkage-compensation
- reduced cracking due to subsidence
- reduced rebar corrosion due to reduced cracking
- reduced surface capillaries and surface porosity due to very little bleed water
- reduced cracking due to increased tensile strength and ultimate tensile strain
- increased bond strength and thus rebar bonding
- fewer construction and contraction/control joints resulting in less deck leakage
- increased wear and abrasion resistance
- easier concrete placement because of high slump.

The above is an impressive list of improved characteristics over ordinary Portland cement, and thus type K cement should be studied closely as an attractive material option to enhance bridge deck durability. Particularly valuable would be a field testing program to compare shrinkage/cracking/permeability performances of

- Current standard concrete used for ALDOT bridge decks
- Modified design to reduce shrinkage and permeability, e.g., lower cement and water contents, increased coarse aggregate size and increase aggregate content.
- Shrinkage compensating concrete
- Shrinkage compensating concrete with microsilica.

3. RESEARCH PLAN

3.1 General

Service life/durability performances of concrete mixes and bridge deck designs are difficult to assess over a short period of time, and testing over extended periods loses focus, enthusiasm and interest. A rational research plan is (1) to initially learn as much as possible about SCC from the literature and from the experiences of others, (2) to develop a good candidate SCC mix design and determine its properties and characteristics via laboratory testing, and (3) to emplace a SCC deck on a bridge for field evaluation. This was the basic approach taken in this study with the exception that funding did not permit us to go to the field test bridge evaluation.

In order to achieve the objectives of the project as stated in Chapter 1 in a systematic and efficient manner, four specific work tasks were identified. These tasks along with brief descriptions of the research plan for each constitute the project research plan, and are presented below.

3.2 Research Task 1

Whereas the use of shrinkage compensating concrete (SCC) in this country is relatively new, still many applications in industrial floor slabs and highway bridge decks have been undertaken. Hence, the first task of our research plan was to glean as much information as possible from these applications. This was accomplished via a "literature" review which included,

- Reviewing technical published literature on the characteristics and properties of SCC and on its applications in highway decks, floor slabs, and water/sewage treatment plants.
- Reviewing standards of practice, specifications, and testing methods which are applicable to SCC.
- Visiting with a manufacturer of Type K cement (Blue Circle Cement Co.) to get their guidance and recommendations on the characteristics, use, limitations, and special construction considerations for SCC.
- Visiting via phone with select people in the U.S. that have had first hand experience with SCC applications to bridge decks.

- Visiting the Ohio Turnpike Commission and the New York Thruway Authority to observe their SCC bridge decks, and to discuss the merits and demerits of SCC applications to bridge decks.

3.3 Research Task 2

The second task of our research plan was to perform trial concrete mix designs to identify four (4) candidate mixes to be use in a later follow-up field evaluation program. The candidate mixes sought were:

1. Standard ALDOT bridge deck PC concrete (for control).
2. SCC mix.
3. High Performance Concrete (HPC) mix of PC concrete with microsilica.
4. SCC with microsilica.

This task was started shortly after beginning Task 1 and to a large extent the two tasks proceeded concurrently.

In performing the mix designs, the plan was to hold the ingredients below constant for all four mixes.

Coarse Aggregate - No. 57

Fine Aggregate - No. 100

Weight of Cementitious Material

- Type I PC
- Type I PC + Micro Silica
- Type K Cement
- Type K Cement + Micro Silica

and, to use two values, one low and one high, for the

w/c ratio

Water reducing agent

Air entraining agent

This would allow us to determine the sensitivity of the strength, shrinkage and other characteristics to these parameters for each mix, and to determine the best design for each of the four mixes. Thus, the trial mix design procedure for each of the 4 mixes was as indicated in Figure 3.1.

To assess the characteristics and relative performances of the mix designs, the following Standard ASTM Tests were performed on each mixture.

- Slump - C143
- Unit Weight - C138
- Air Content - C231
- Compressive Strength - C39
- Splitting Tensile Strength - C496

Additionally, the following Standard ASTM Tests were performed on four mixtures, i.e., one mixture with each of the four cementitious material combinations and no admixtures.

- Time of Setting - C807
- Restrained Expansion of Cement Mortar - C806

Last, the following Standard ASTM Test was performed on each of the mixes containing Type K cement.

- Restrained Expansion of SCC - C878

3.4 Research Task 3

The third task of our research plan was to use the four mix designs emerging from Task 2, and test them in the same manner as in Task 2, i.e., test

- Slump
- Unit Weight
- Time of Setting
- Air Content
- Compressive Strength

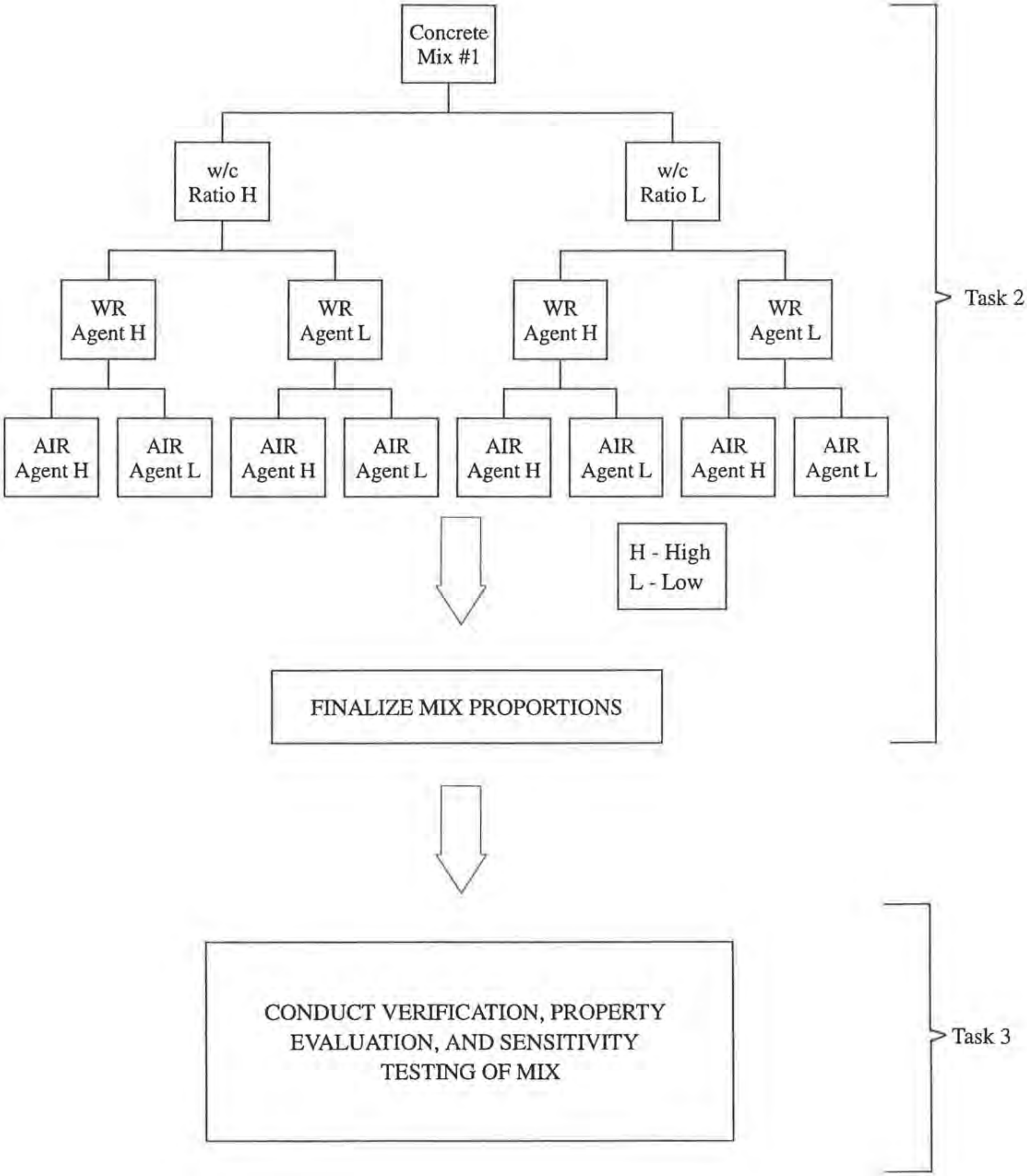


Figure 3.1. Typical Mix Design Procedure

- Splitting Tensile Strength
- Restrained Expansion of SCC mixes
- Restrained Expansion of Cement Mortar

This testing allowed us to make relative comparisons of the four mix designs as well as positioned us to later compare and assess the relative performances of these same mixes in field test bridge applications.

Additionally, the standard PC (control) and the SCC mix were tested for sensitivity of

- Slump
- Compressive Strength
- Splitting Tensile Strength
- Restrained Expansion

to simulated hot (90ºF) and cold (40ºF) field conditions. This testing was done with the aid of a laboratory environmental control chamber.

Last, the standard PC (control) and SCC mix were also tested for sensitivity of

- Compressive Strength
- Splitting Tensile Strength
- Restrained Expansion

to the following curing conditions

- Normal Quality Curing (7 day wet curing)
- Poor Quality Curing (dry curing)

3.5 Research Task 4

The fourth task of our research plan was to try to coordinate the placement of some bridge test decks using the 4 mix designs of the second phase work. Emplacement of the test decks requires development of construction document changes (bridge deck construction drawings and specifications), and identification and development of the changes needed was a part of this work.

The test decks are to be placed on one or more bridges in the ALDOT system.

Since service life/durability performance assessment by direct means is not reasonable from a time standpoint, crack developments, concrete and steel strain levels, material compressive strength and permeability values, material robustness and sensitivities to concrete mix variations, construction variations, and variations in environmental conditions should be monitored and recorded for the test bridge. These will be used, along with engineering judgement, to estimate service life/durability enhancement or nonenhancement of SCC, SCC with microsilica, and HPC bridge decks relative to ordinary PC concrete decks.

4. RESULTS OF VISITS WITH SCC USERS

4.1 General

The Ohio Turnpike Commission (OTC) is the greatest user of SCC by far in the U.S., with approximately 500 SCC bridge decks. The New York Thruway Authority (NYTA) is probably the second largest user, and had recently emplaced some 30 plus bridge decks of SCC. As part of the research plan for the project, the principal investigators (PIs) arranged a visit with these largest users to learn first hand from their experiences. Both visits were made in May 1995 and 2 days were spent with each agency in discussing their mix design, construction requirements, special specifications, inspection procedures, etc., pertaining to the use of SCC. At both agencies, we visited many bridge sites and had an opportunity to observe SCC decks from 10 years to 1 year old. Brief summaries from each visit are given below.

4.2 Ohio Turnpike Commission Visit

At the Ohio Turnpike Commission (OTC) in Cleveland, Ohio, we met and discussed SCC bridge decks with the following personnel:

Mr. David Ransberry, Chief Engineer

Mr. Michael Phillips, Construction Engineer

Mr. Kerry Ferrier, Design Engineer

Mr. Ransberry had earlier made a presentation of OTC's use of SCC to mitigate early shrinkage cracking of replacement bridge decks in their system. He gave us a copy of his presentation, and also verbally made the same points to us. An abridged version of Mr. Ransberry's presentation follows.

> Cracks in concrete bridge decks will become a subject of great concern and worry if you are an agency that has just spent anywhere from \$500,000 to \$5,000,000 on a reinforced concrete bridge deck replacement that has transverse, full-depth cracks neatly spaced along the entire length of the deck before the first vehicle ever uses the new bridge deck. Photographs of this early cracking problem are shown in Figures 4.1 - 4.3. This scenario happened to the OTC in 1984 after the second year of their bridge program that called for the bridge deck widening, replacement and rehabilitation of the 519 structures

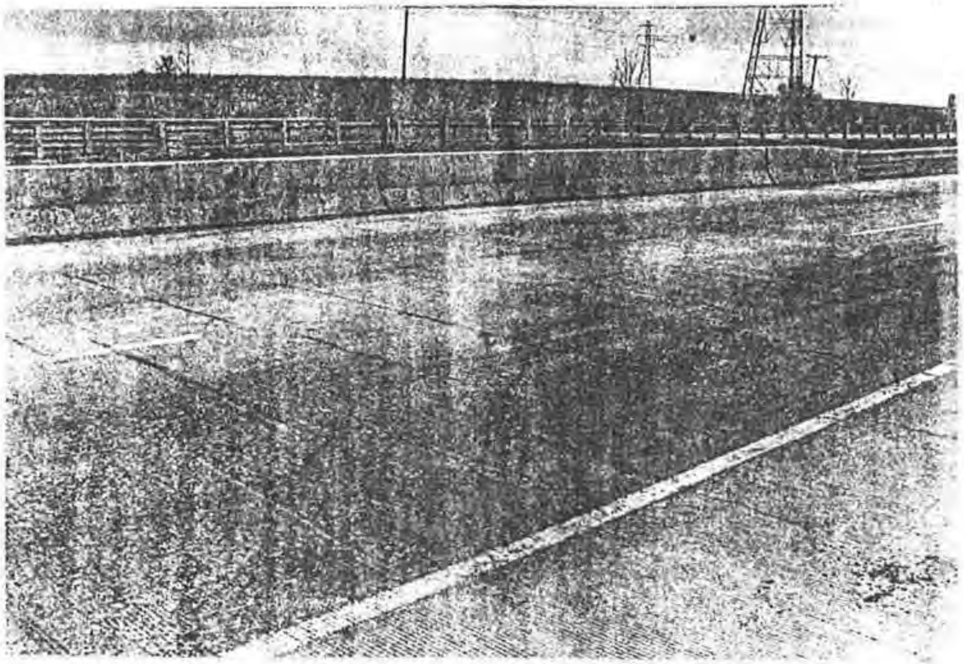


Figure 4.1. View of Top Surface at Poland-Unity Crossing (25)

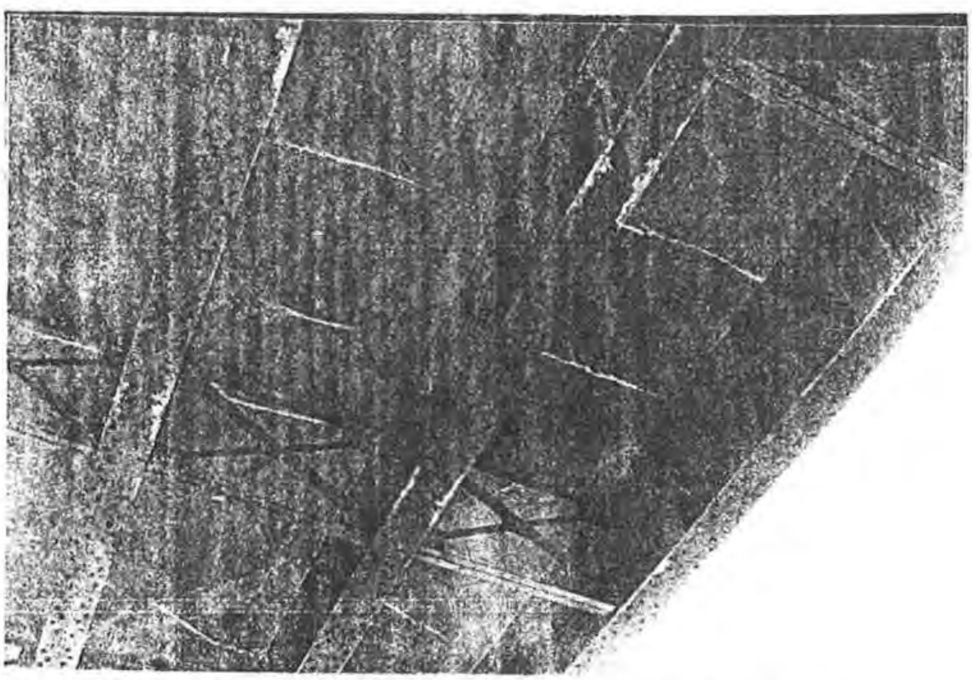


Figure 4.2. View of Underside of Deck for Conrail Crossing (25)

Figure 4.3. View of Underside of Deck at Unknown OTC Crossing

under their jurisdiction. A summary of the OTC's deck replacement program, and the resulting drying shrinkage cracking is shown in Table 4.1. As reflected in the table, during the 1984 construction program, the OTC began experimenting with using shrinkage compensating cement for bridge decks. They emplaced 2 Type K cement decks in the total of 67 that they replaced that year, and by the end of the year had not observed any deck cracking in these trial decks.

At the end of the 1984 construction season, the OTC was at the point where they felt something had to be done. They simply could not continue to reconstruct 50 to 60 bridges per year using the same Ohio DOT specifications, and have nearly 75 percent of them crack. The first step OTC took was to find out why a high percentage of their replacement decks placed in 1983 and 1984 had various degrees of drying shrinkage cracks (DSC). It did not appear that workmanship or specification compliance was the problem as cracking occurred with different contractors and with different inspection and testing groups. A check with the Ohio DOT bridge engineers revealed that they too had various degrees of cracking problems, particularly since the higher cement concrete requirements were introduced in the 1979 specification. The Ohio DOT had just completed, in 1983, the redecking of the 3,000 foot long Hope Memorial bridge in the city of Cleveland using their specifications and the results were severe drying shrinkage cracks throughout the entire length.

Thus, the OTC authorized the Construction Technology Laboratories (CTL) in December 1984, to perform an accelerated study on "how and why their decks were cracking and to recommend some corrective actions." The study was to be completed in January 1985 so that OTC could incorporate any changes into their 1985 construction program. CTL performed the study and delivered their final report in January 1985. The primary findings of their study are briefly summarized below.

- The deck cracks ran transverse to the direction of traffic and were generally spaced 4 to 8 feet on centers.

Table 4.1
Ohio Trunpike Bridge Deck Replacement Program and Summary of
Deck Cracking due to Drying Shrinkage

YEAR	DEGREE OF CRACKING NONE	MINOR	MODERATE	SEVERE	TOTAL DECKS REPLACED
1983	3 50%	2 33%	1 17%	- 0%	6
1984	17▲ 25%	15 22%	16 24%	19 29%	67
1985*	33 87%	4 10%	1 3%	- 0%	38
1986	43 96%	2 4%	- 0%	- 0%	45
1987	72 97%	1 1.5%	1 1.5%	- 0%	74
1988	57 96%	1 2%	1 2%	- 0%	59
1989	53 100%	- 0%	- 0%	- 0%	53

▲Includes 2 bridge decks using Type K cement (this was the trial project using Type K cement)
*All bridge decks replaced after 1984 contained Type K cement only.
+271 deck replacements were with Type K cement, and cement 71 with Type I or II cement.

Of the total 271 decks using Type K cement, 11 decks show some degree of drying shrinkage cracks (DSC). 5 of the 11 Type K (DSC) decks are ramp structures poured while subjected to live load vibrations.

Of the total 71 decks using Type I or II cement, 53 decks show some degree of (DSC). 3 of the 53 Type I or II (DSC) decks are structures poured while subjected to live load vibrations.

Excluding structures poured under live load conditions, our experience shows DSC occurred in 2.3% of decks using Type K cement whereas DSC occurred in 73.5% of decks using other cements.

Minor Cracking - 1 to 5+ total cracks

Severe Cracking - Cracking spaced 6' ± across entire deck or several spans

Moderate Cracking - Limited cracking which exceeds minor limits and less than severe

- The concrete was high quality with tight paste-aggregate bond and compressive strengths ranging from 5,500 P.S.I. to 7,300 P.S.I.

- Vertical cracks passed primarily around the large aggregate which suggests these cracks were formed early during the hydration process prior to development of sufficient paste-aggregate bond. The cracks were, therefore, confirmed as associated with drying shrinkage.

- Drying shrinkage may have been compounded by thermal stresses due to heat of hydration generated during early curing.

- Petrographic examination indicated a water cement ratio ranged from 0.30 to 0.45.

- Air content ranged from 5.5 to 8.5%

- Maximum coarse aggregate size ranged from ½ to ⅝ inch.

- Coarse aggregate was primarily crushed limestone and dolomite, which reduces the potential for shrinkage.

- Cement contents were at 715 lbs. per cubic yard.

- The data confirmed that the decks had good quality, structurally sound concrete which; however, had severe drying shrinkage cracks that would require increased maintenance and possibly shorten the deck life cycle.

- The following options, or combinations of these options, could be used to reduce the potential for shrinkage cracks:

    1. Reduce the cement content and increase the maximum coarse aggregate size to 1 inch.

    2. Use the lower heat of hydration Type II cement.

    3. Restrict curing to continuous water cure over burlap for a minimum of 7 days or 14 days for Type II cement.

    4. Use shrinkage compensating cements.

The OTC's reaction to CTL four corrective options was as follows:

1. Recommendation #1: suggesting reduced cement and larger aggregate, although may very well reduce or eliminate shrinkage cracks, did not seem acceptable. It would be sacrificing the superior, long term durability qualities known to exist with the richer, small aggregate mixes.

2. Recommendation #2: using Type II cements with its low hydration heat could be a solution; however, the recommended 14 day curing would not fit into our required tight construction schedules and was therefore rejected.

3. Recommendation #3: concerning timely and continuous water curing over burlap was adopted as the only curing permitted. The use of burlene and other plastic sheeting was eliminated from the specifications.

4. The final recommendation: using shrinkage compensating cement seemed to offer the best alternative to solving the cracking problem. True, there is an increase in material cost over Type I cements; however, there was every reason to believe that cracking could be reduced or eliminated and we could keep the rich mix, small aggregate design that was necessary for long term durability. From a contractor's view, the higher slump (5" ± 1") makes it easy to handle and pump. With the exception of a possible slight cost increase, it appeared this material offered all the advantages we were looking for without sacrificing anything. In January of 1985 a decision was made to use Type K shrinkage compensating cement in all bridge reconstruction work.

Gradually, over the last 10 years, the OTC has made various refinements in the Type K concrete specifications based on their experience. Great emphasis is placed on curing. They use fog spraying under certain weather conditions, always use a monolecular film to retard evaporation, and even control the curing water temperature to avoid thermal shock. These curing details are necessary to insure the expansion works to its maximum plus control any plastic surface cracks which can occur where surface evaporation exceeds the concrete bleeding rate. Beginning with the two trial bridges in 1984 and continuing through the current year, the OTC has completed a total of 294 bridges using over 100,000 cubic yards of concrete, with Type K shrinkage compensating cement. This probably makes the OTC the single largest user of Type K cement in the United States.

In closing, based on our experience with Type K cement over the past 10 years, I strongly recommend that any agency planning a deck replacement program consider the use of Type K cement. In 1984 we approached each final deck inspection prepared to count and locate cracks - now we hardly ever give it a thought.

As evident from the above summary, and from our discussions, Mr. Ransberry has a very strong positive opinion about the use of SCC for bridge decks.

Following our meeting with Mr. Ransberry, we met with OTC's Construction Engineer, Mr. Michael Phillips who provided us with detailed information on SCC as related to bridge deck construction. Much of the information that Mr. Phillips passed onto us from his experiences with SCC are included in the literature review information in Chapter 2. However, we combined other specific construction information that he provided with that provided by the NYTA in the form of a brief set of Guidelines for Use of SCC and these are given in Appendix A. Also, Mr. Phillips showed us a very enlightening set of photographs taken at a new SCC deck emplacement the year before. He allowed us to copy the photographs, and these are presented and discussed below.

The photographs in Figures 4.4 - 4.19 show the chronological sequence of construction activities on an OTC access bridge deck which was constructed of Type K cement in 1994. The bridge provides access to an industrial park and was equivalent in width to a four lane bridge, and was approximately 250 feet in length. The deck was placed, finished, and wet burlap curing emplaced in 1-day.

Figure 4.4 shows the bridge just prior to beginning placement of the concrete. Note a worker wetting down the forms and rebar in the background, the plywood walkway down the center, and the walkway/workway along each side of the bridge. Figure 4.5 shows the concrete pumper in place with a ready mix truck ready for discharging. Figures 4.6 and 4.7 show the initial placements of the concrete at one end of the bridge. The placement was continuous from one end to the other. Figures 4.8 and 4.9 show close-ups of the placement by pump and consolidation by a vibrator. The SCC requires a rather large amount of water for growth of the ettringite (the expansive component). This makes it friendly to pumping and requires little vibration to assist in placing and consolidating. Note in Figure 4.9 that the longitudinal steel is on top of the transverse steel in the top mat.

Figure 4.10 shows continuing placement consolidation of the concrete along with roller screeding, and Figure 4.11 shows roller screeding followed by bull floating of the surface. Only one pass of the bull float is allowed by the OTC. Note in the background of Figure 4.11 that workers are already preparing the wetted double layer of burlap required for curing. Figures 4.12 and 4.13 show close-up views of the roller screeding and bull floating operations and the smoothness of the resulting deck surface. There is very little bleed water when using SCC, and this is evident in Figures 4.12 and 4.13.

Figures 4.14 shows hand screeding/smoothing work being performed at the deck edge from the edge walkway. Note in this figure, the closeness of the concrete placement (in the background), screeding, floating, and tining (see right-lower corner deck surface). Figure 4.15 shows a wheel-barrow sampling of concrete being taken at the end of the pump line for preparation of concrete test specimens. Figure 4.16 shows the closeness of the bull floating and surface tining operations, and Figure 4.17 is a close-up of the tining operation and the tined surface. The cohesiveness/flattyness of SCC make it a good material for tining because it minimizes unraveling and gives a quality tined surface. Even so, the OTC indicated that the contractor must keep his tining instrument sharp to minimize unraveling.

Figure 4.18 shows the continuing screeding, floating, tining, and the wetting (see worker in background) of burlap cover to be used for curing. Figure 4.19 shows application of an evaporation retarder to the deck surface. Note in this figure how closely the retarder is being used behind the tining operations. Figures 4.20 and 4.21 show the two layers of wetted burlap being prepared and placed on the deck for curing. Early wet curing which is maintained for a minimum of 7-days is essential when using SCC.

Figure 4.22 shows the deck approximately 1-year later at the time of our visit of OTC. The surface exhibited minor spalling of some tining ridges as can be seen in the photograph. However, there was no evidence of any cracking on the top or bottom surfaces of the deck.

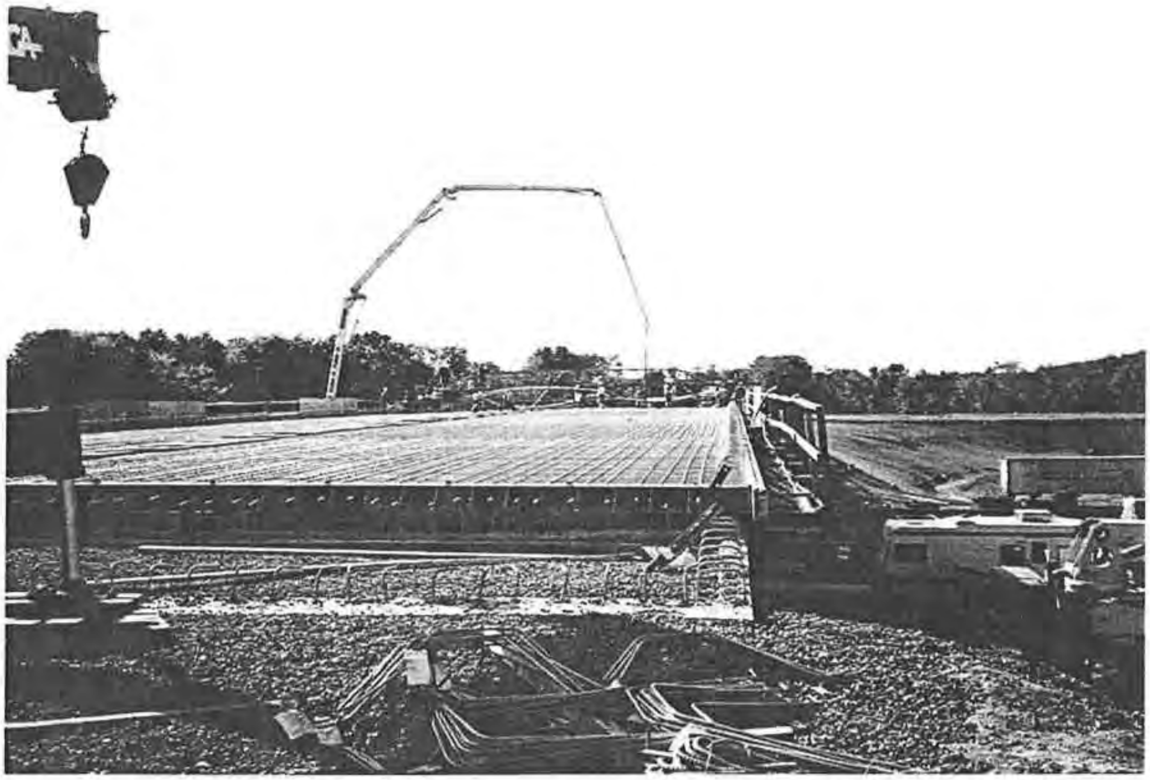


Figure 4.4. Entire Bridge Deck Ready for Concrete Placement Via Pumping

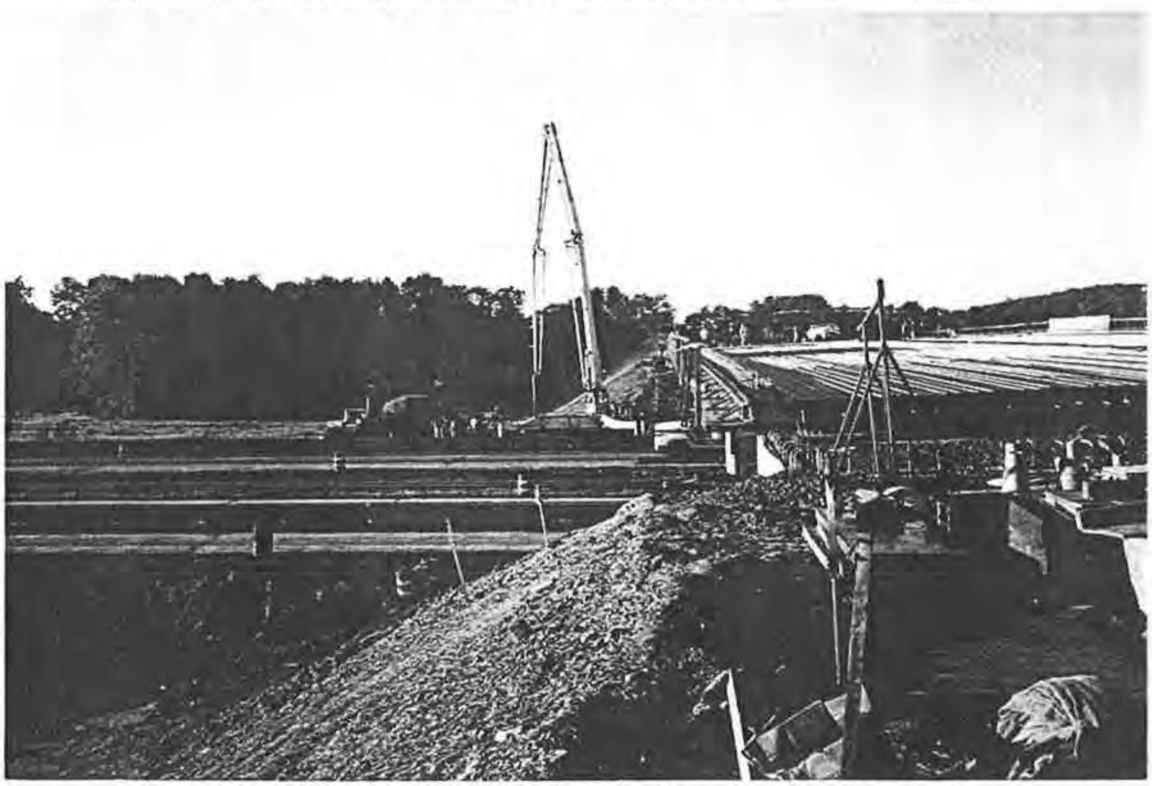


Figure 4.5. Ready Mix Truck and Concrete Pumper Adjacent to Deck

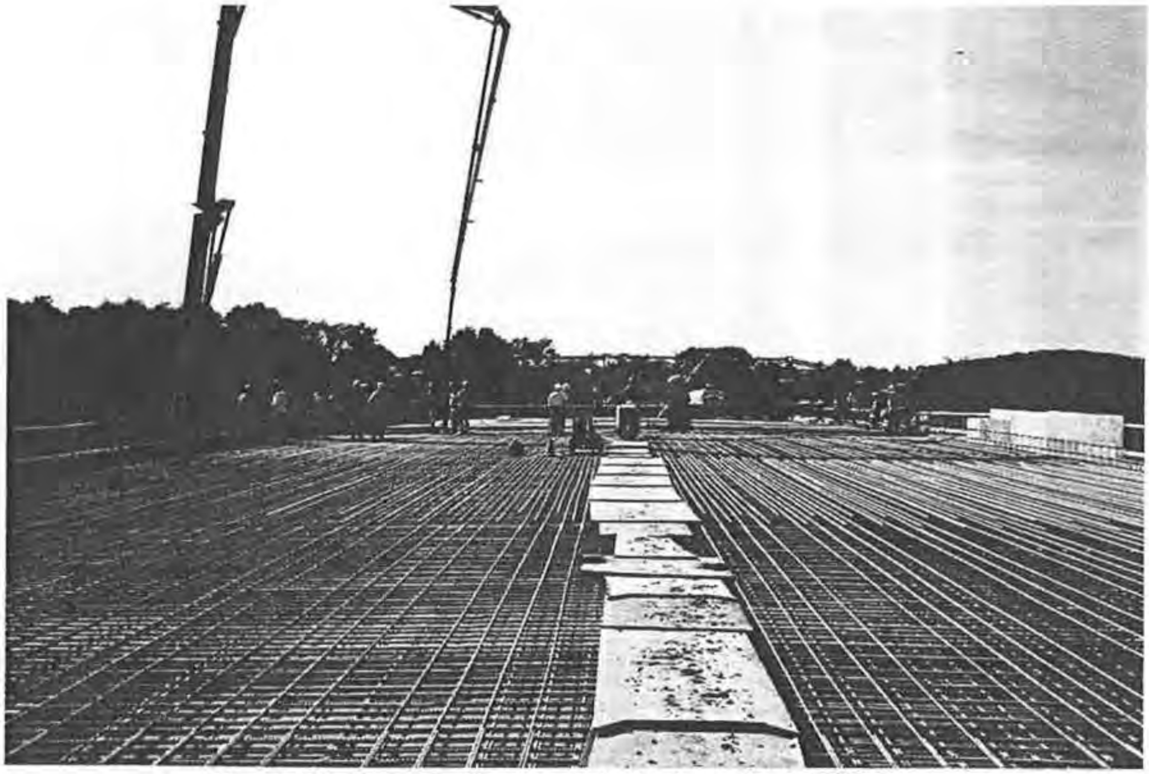


Figure 4.6. Concrete Placement Beginning at End of Bridge

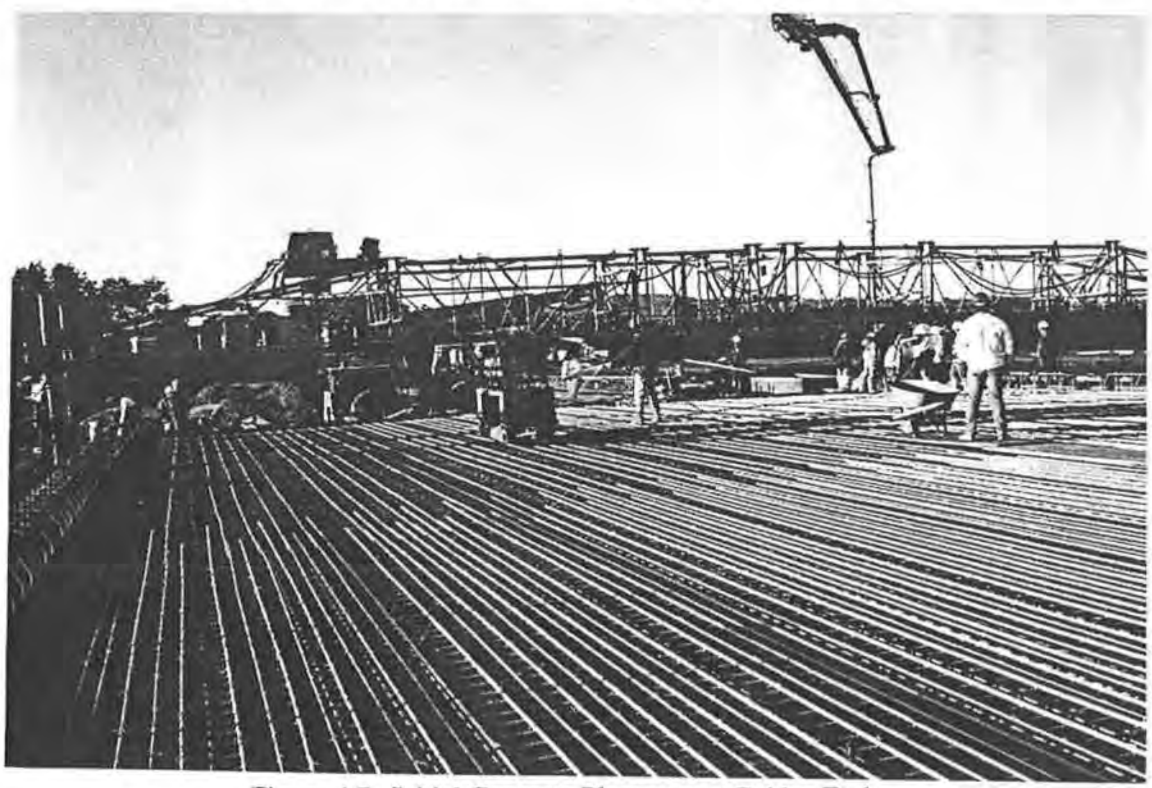


Figure 4.7. Initial Concrete Placement at Bridge End

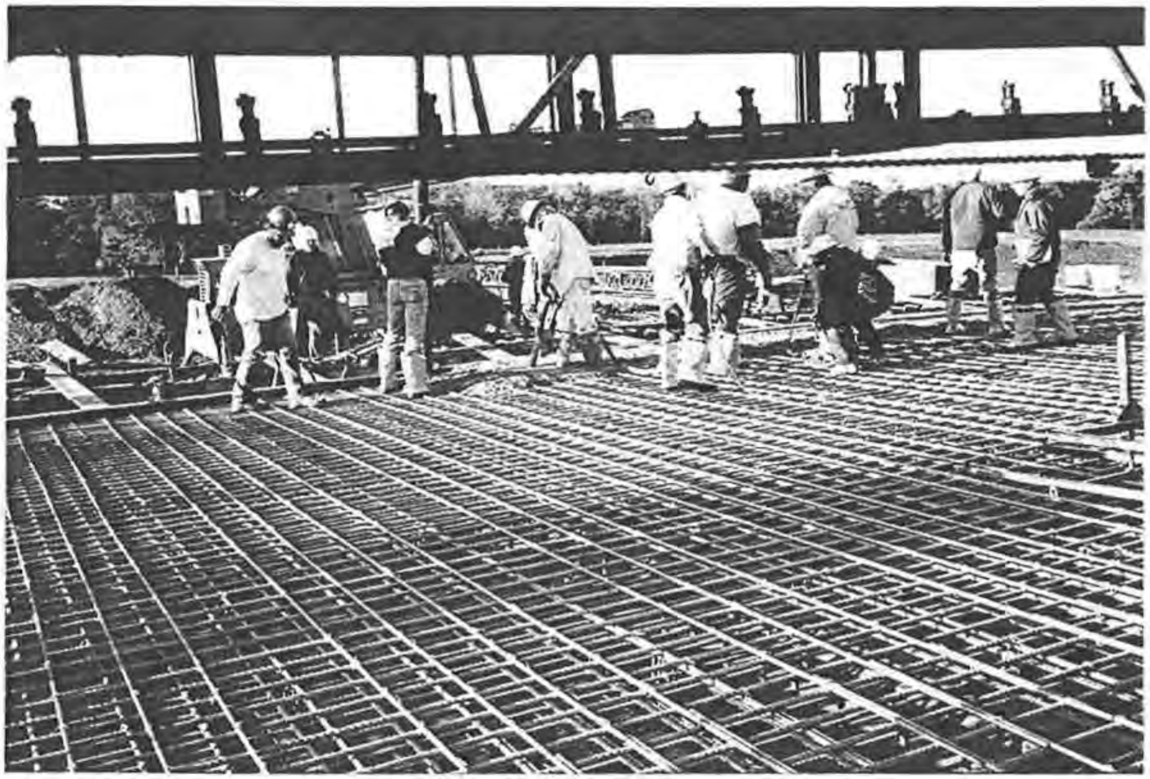


Figure 4.8. Close-Up of Initial Concrete Placement

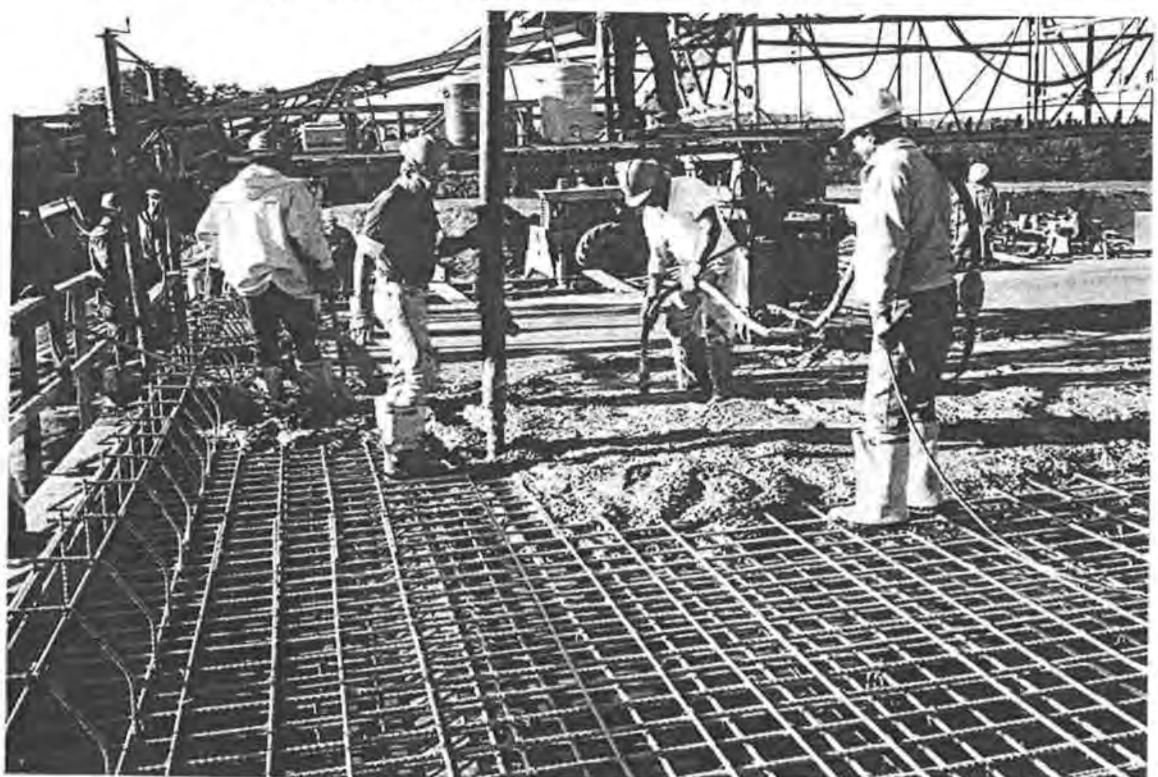


Figure 4.9. Concrete Placement Via Pumping

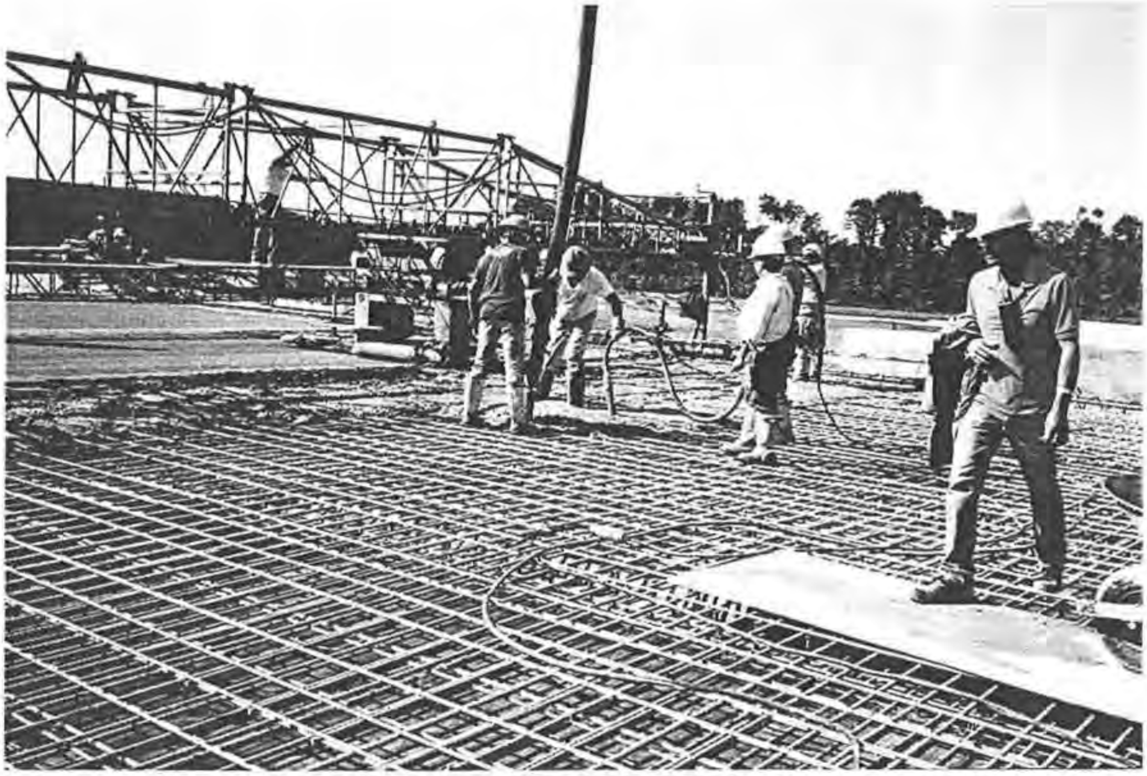


Figure 4.10. Concrete Placement and Roller Screeding

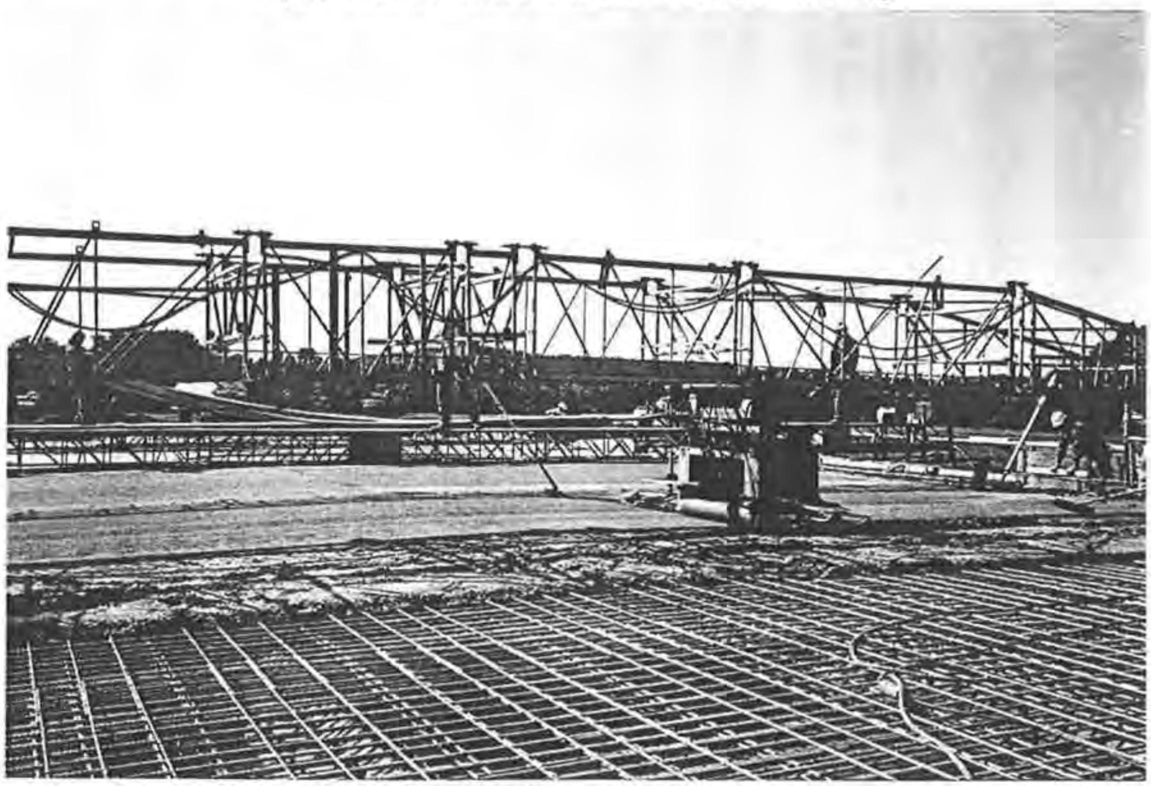


Figure 4.11. Roller Screeding and Bull Floating of Concrete

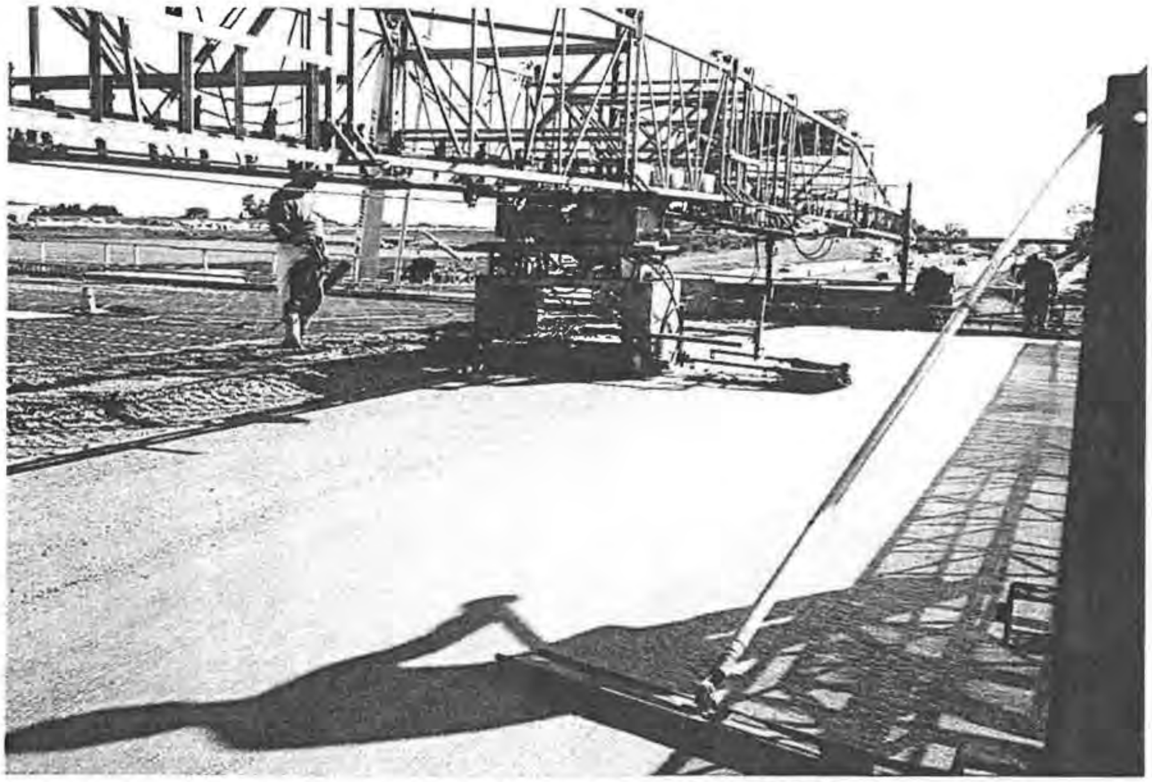


Figure 4.12. Close-Up of Roller Screeding and Bull Floating

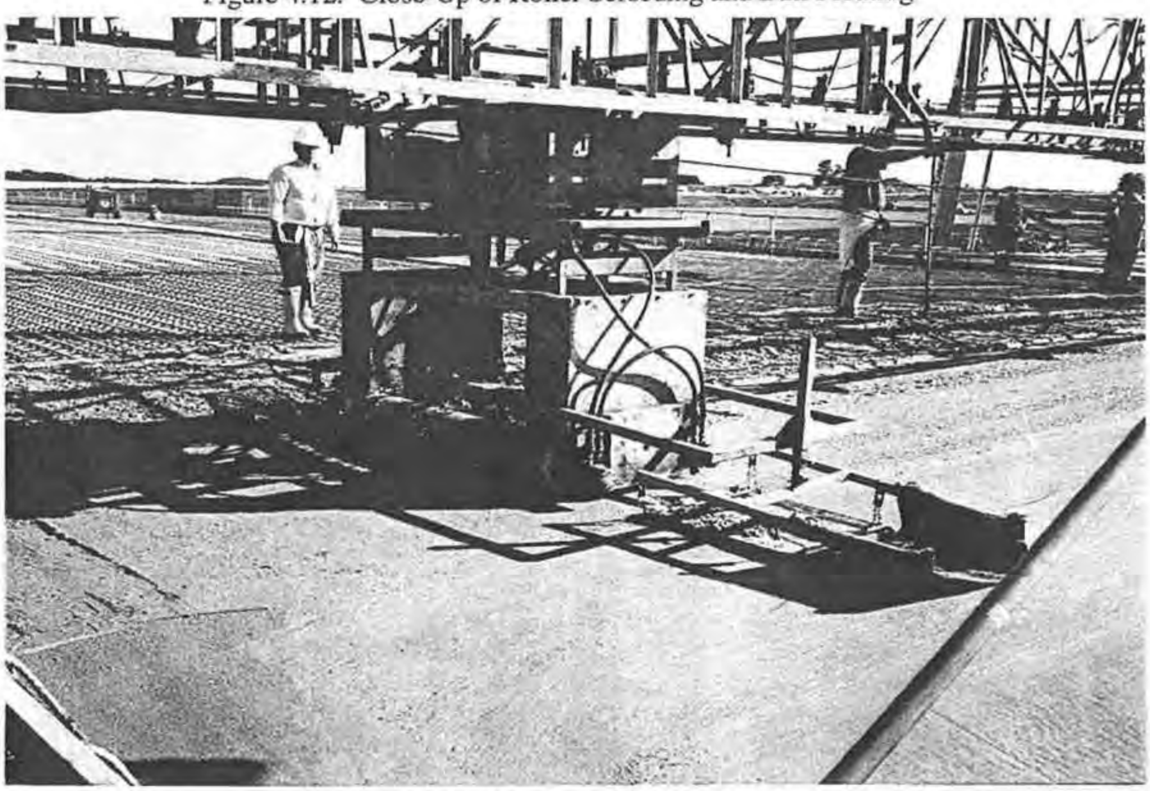


Figure 4.13. Close-up of Roller Screed

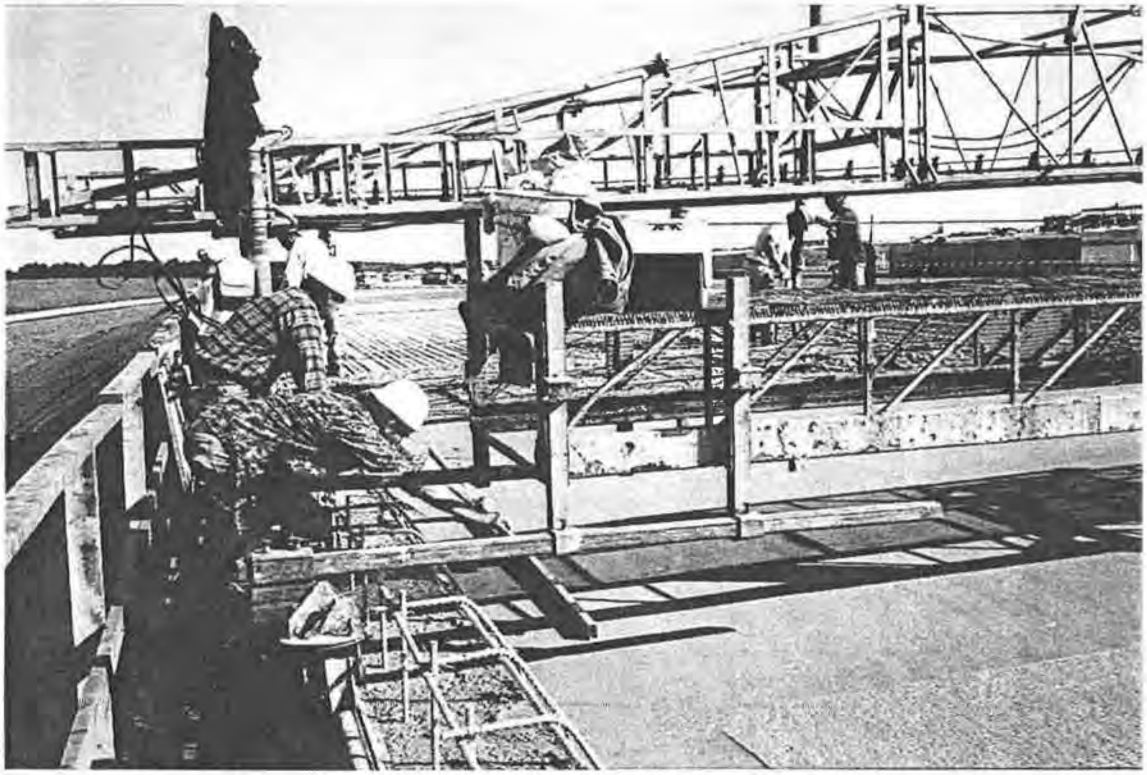


Figure 4.14. Hand Screeding at Deck Edge/Guardrail

Figure 4.15. Concrete Sampling at End of Pump Line

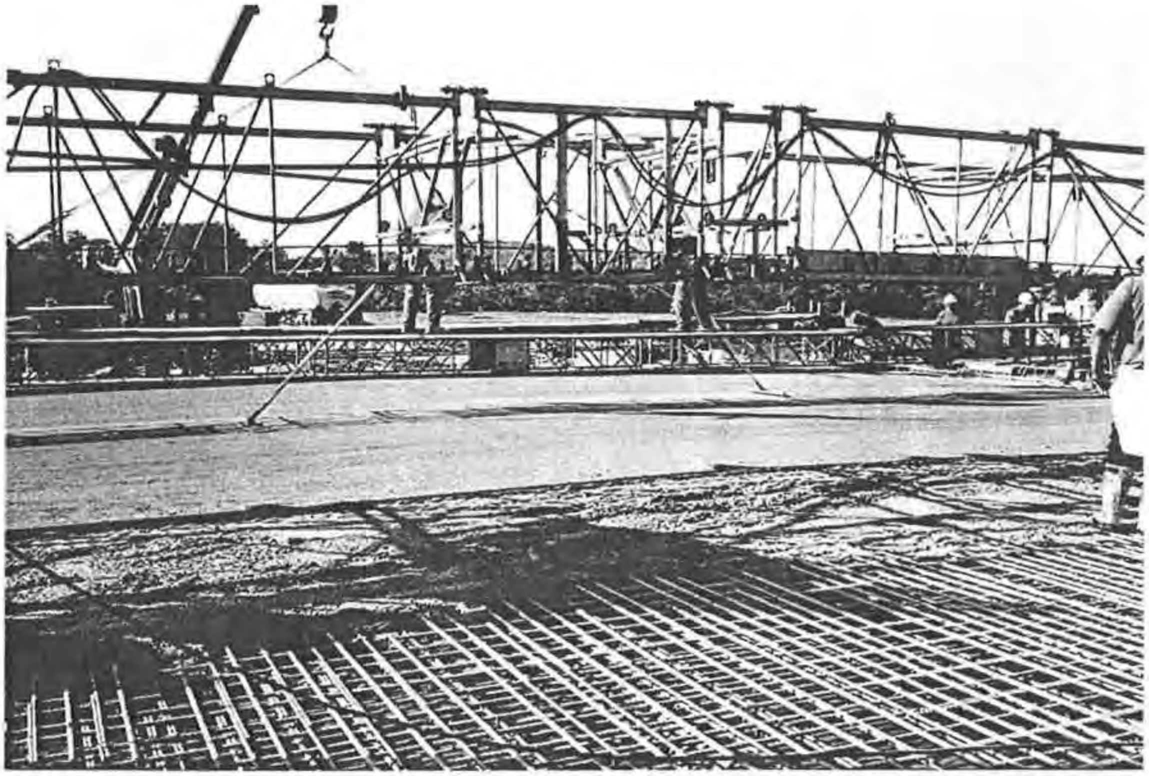


Figure 4.16. Bull Floating and Tining of Deck Surface

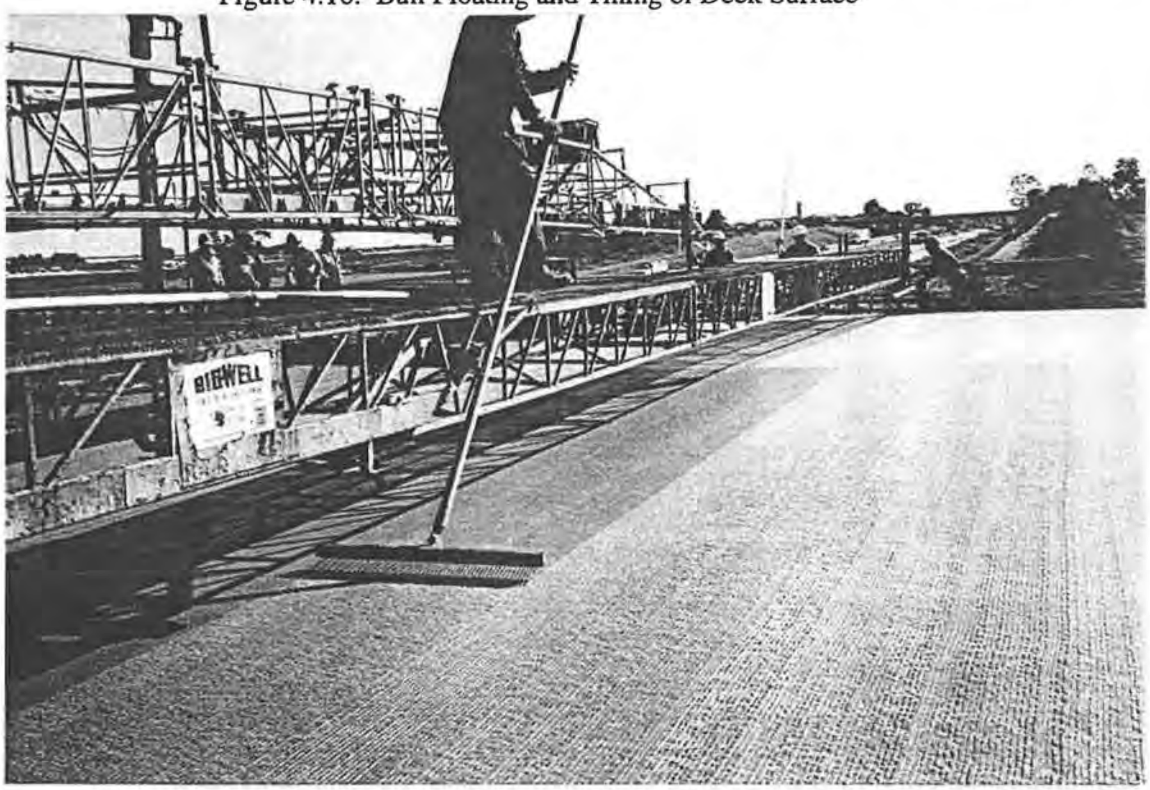


Figure 4.17. Close-up of Tining of Deck Surface

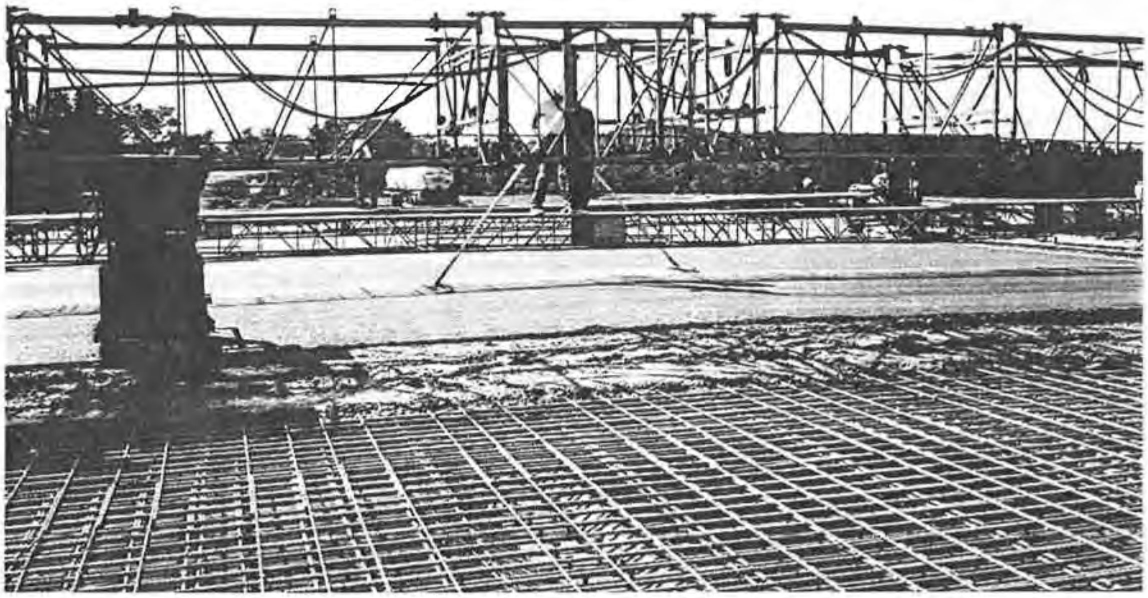


Figure 4.18. Screeding, Floating, Tining and Wetting of Burlap

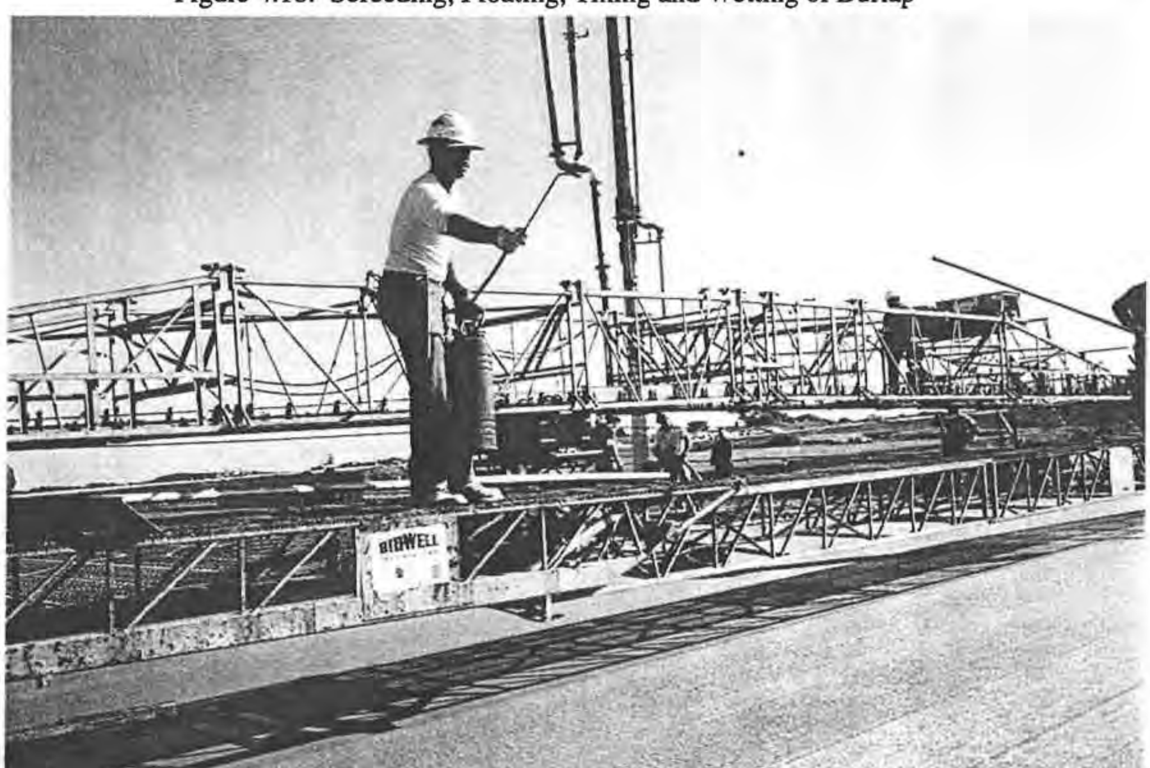


Figure 4.19. Close-up of Applying Evaporation Retarder

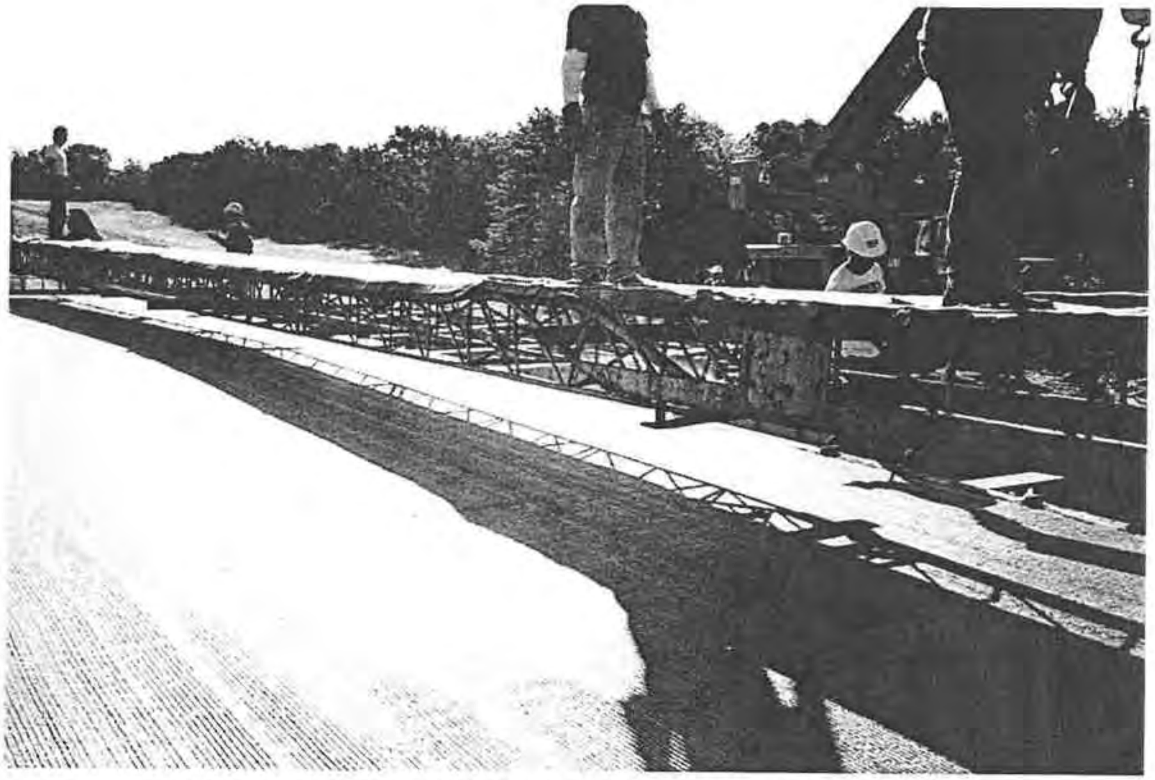


Figure 4.20. Preparing Wet Burlap for Covering/Curing

Figure 4.21. Placing Wet Burlap for Curing

Figure 4.22. Finished Bridge 1-Year Later

At our meeting, Mr. Phillips indicted that he was aware that the ODOT and the NYTA had poor experiences with SCC decks. And, in an effort to understand why this was so in light of OTC's excellent experience, he contacted Mr. Randy Over of ODOT and Mr. Jim Golden of NYTA to discuss what they may have done differently from OTC. He found that many critical factors were treated the same way by all three agencies. However, a number of critical factors were not treated the same, and these are summarized in Table 4.2.

Following our meeting with Mr. Phillips, we were given a comprehensive and detailed site visit/tour of many SCC bridge decks in OTC's system. The decks were from 1 to 11 years old and some were of normal weight and some of lightweight concrete. Almost without exception, the decks looked new and crack free on both the top and bottom. Photographs of some of the bridges and decks which we visited and inspected are shown in Figures 4.23 - 4.38.

Table 4.2
Differences of OTC, NYTA, and ODOT on Critical Issues for SCC Decks*

Factor/Issue	ODOT (Randy Over)	NYTA (Jim Golden)	OTC (Mike Phillips)
Record Test Samples	Before Pump	Before Pump	End of Pump
Require Test Lab at Batching Plant	Maybe	No	Yes
Batch Out of Spec so In Spec at Placement	No	No	Yes
Use Concrete Finishing Aides	Yes	Maybe	No
Adjust Mix Throughout Placement for Pump and Hose Locations	No	No	Yes
Inspectors Work For	Some of the Participating Players	NYTA	None of Players
Require 3 c.y. Test Batching & Placement	No	No	Yes
Require Pre-placement Meeting	Maybe	Maybe	Yes
Require a Concrete Sealer	No	No	Yes
w/c Ratio	0.50 max	Over 0.50	0.50 max

*Difference as assessed by Mike Phillips of OTC.

The photographs in Figures 4.23 - 4.28 are typical of the Ohio Turnpike bridges, i.e., continuous steel girder bridges with SCC decks which are in excellent condition (see Figures 4.23 and 4.24). The deck on this bridge was constructed (as a replacement) in 1991, and was four years old at the time of these photographs. There was no evidence of any deck cracking on the top or the bottom of the deck. There was some minor spalling at the tined grooves as can be seen in Figures 4.25 and 4.26. Note in Figure 4.24 the significant rusting of the bottom flange of the outside steel girder due to deicing salts. However, as can be seen in Figures 4.27 and 4.28 rusting was not occurring on the inside girders. Also, these figures show the underside of the deck to be in mint condition.

Figure 4.23. Typical Ohio Turnpike SCC Bridge Deck

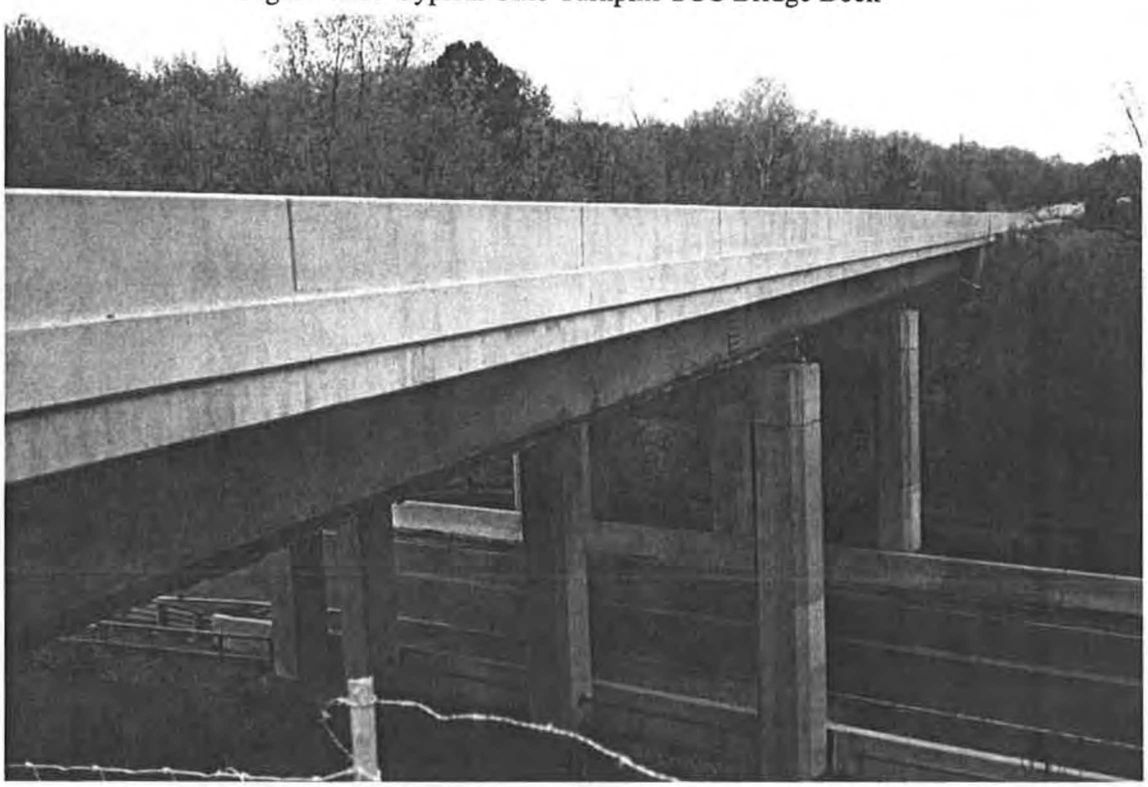


Figure 4.24. Typical Ohio Turnpike Continuous Steel Girder Bridge

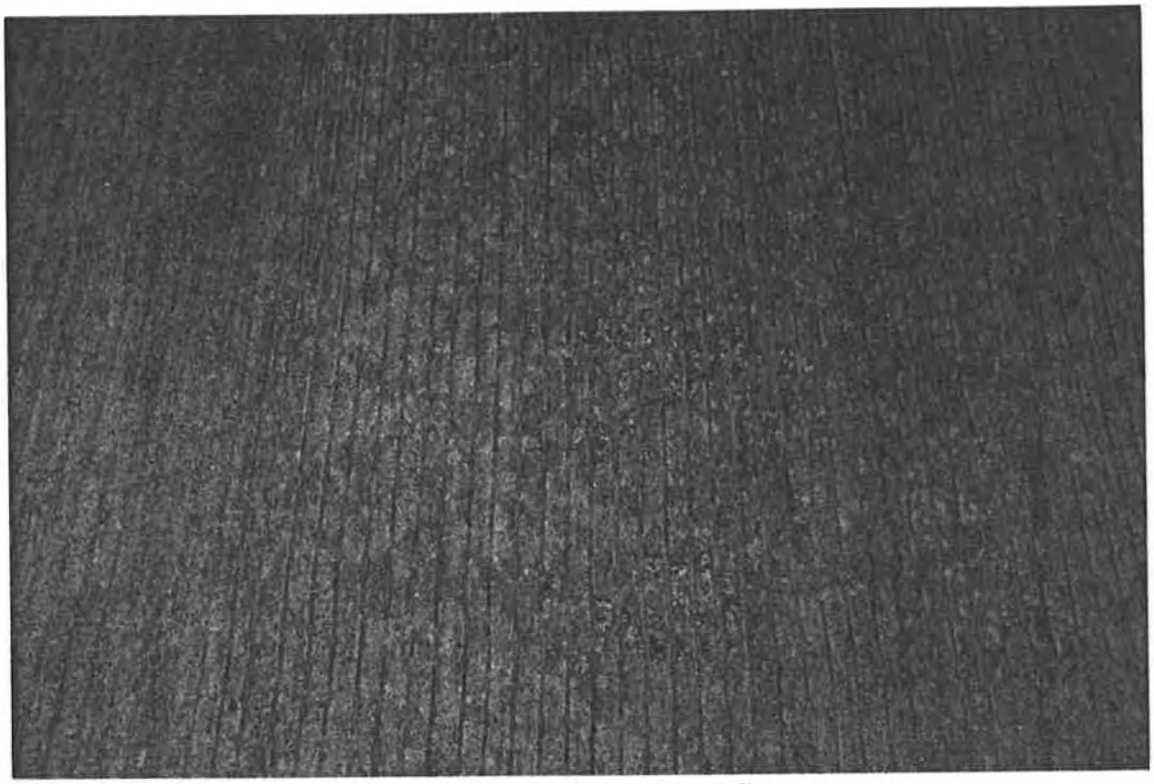


Figure 4.25. Minor Spalling at Tined Grooves

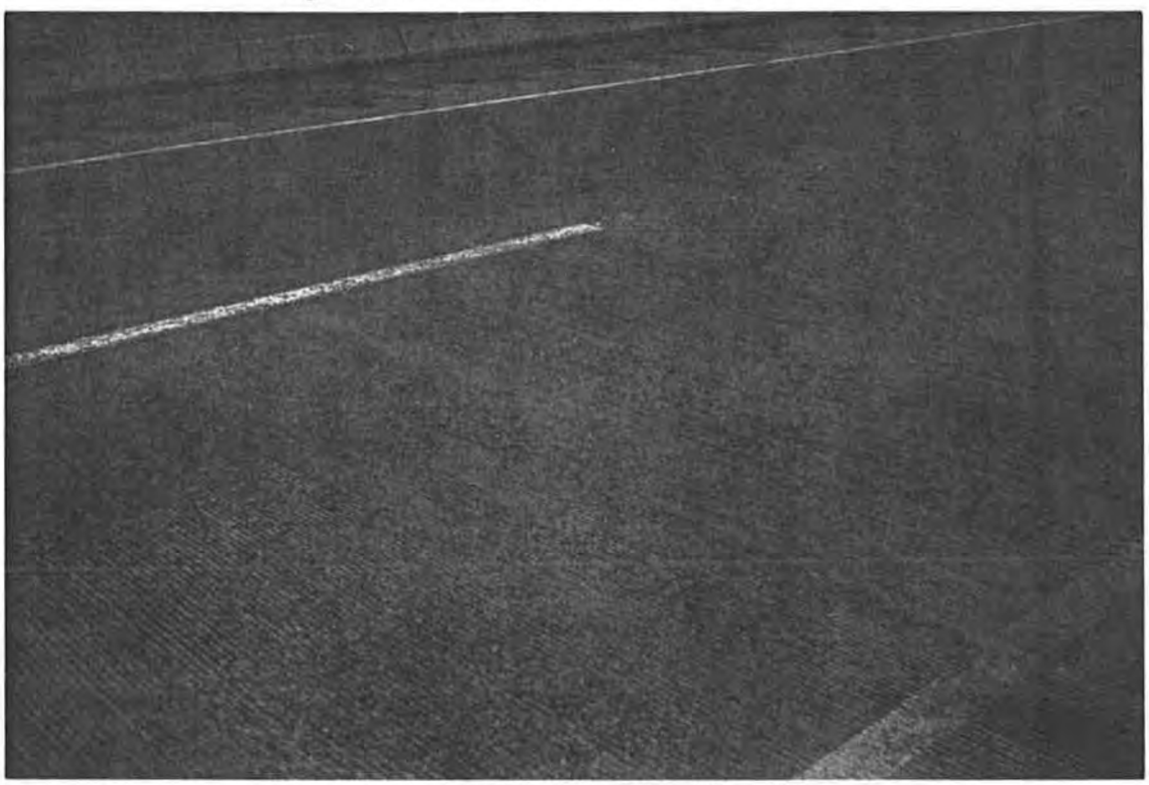


Figure 4.26. Close-Up of Tined Grooves

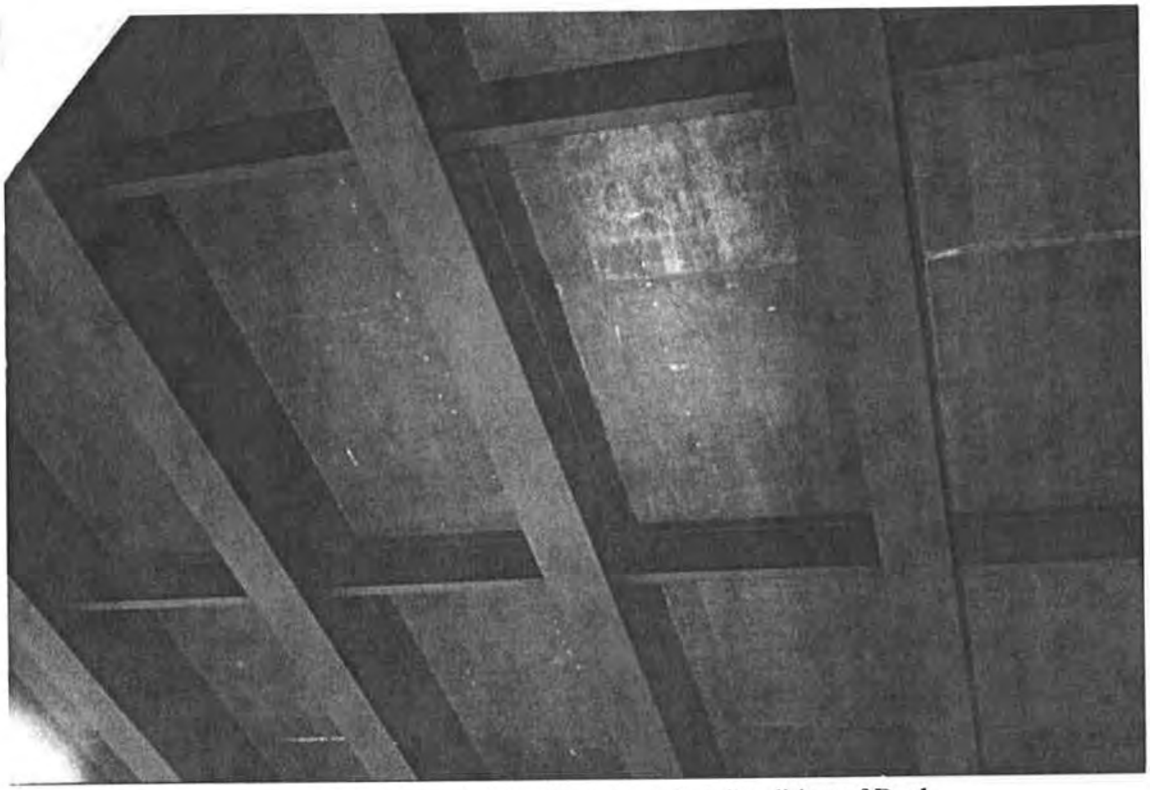


Figure 4.27. Underside of Bridge Showing Mint Condition of Deck

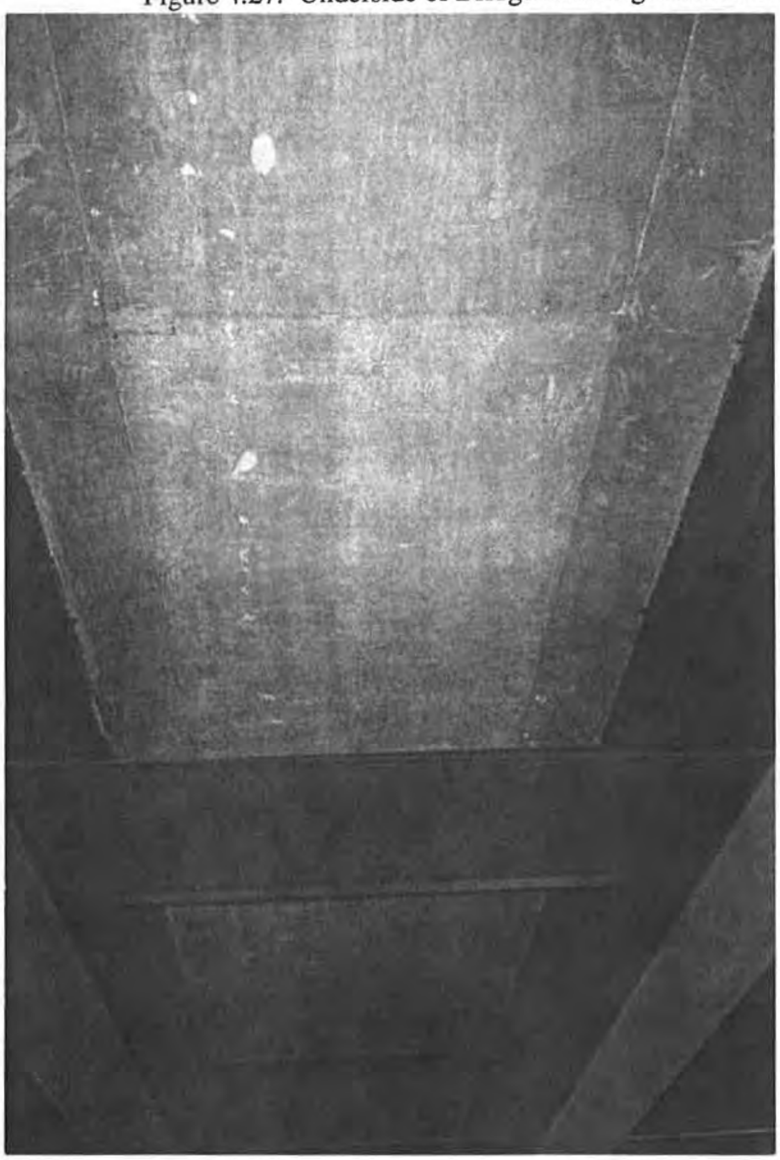


Figure 4.28. Close-Up of Underside of Deck

The photographs in Figures 4.29 - 4.34 show the oldest SCC deck in OTC's system. The deck constructed in 1984 and thus was 11 years old at the taking of the photographs. As with most bridges, expansion joint deterioration leaves something to be desired as evident in Figure 4.30. Figures 4.31 and 4.32 show some traffic wearing of the tined grooved deck surface. Figures 4.33 and 4.34 show bad scaling of the SCC guardrail. OTC's guardrails are cast-in-place after the deck and are also of SCC. The scaling problem of this rail appeared to be unique to this bridge and the reason was not known. Both the top and bottom surfaces of the deck were in excellent condition and essentially crack free.

Figure 4.29. Eleven Year Old SCC Bridge Deck

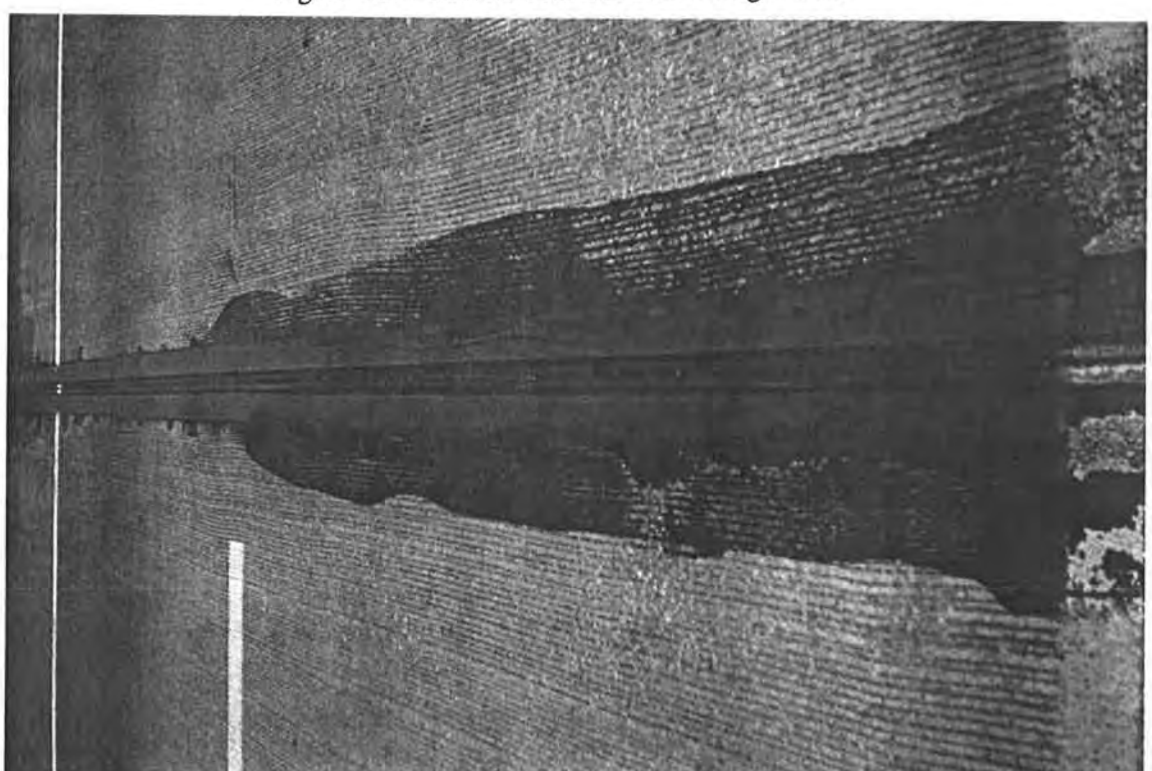


Figure 4.30. Expansion Joint Deterioration and Maintenance

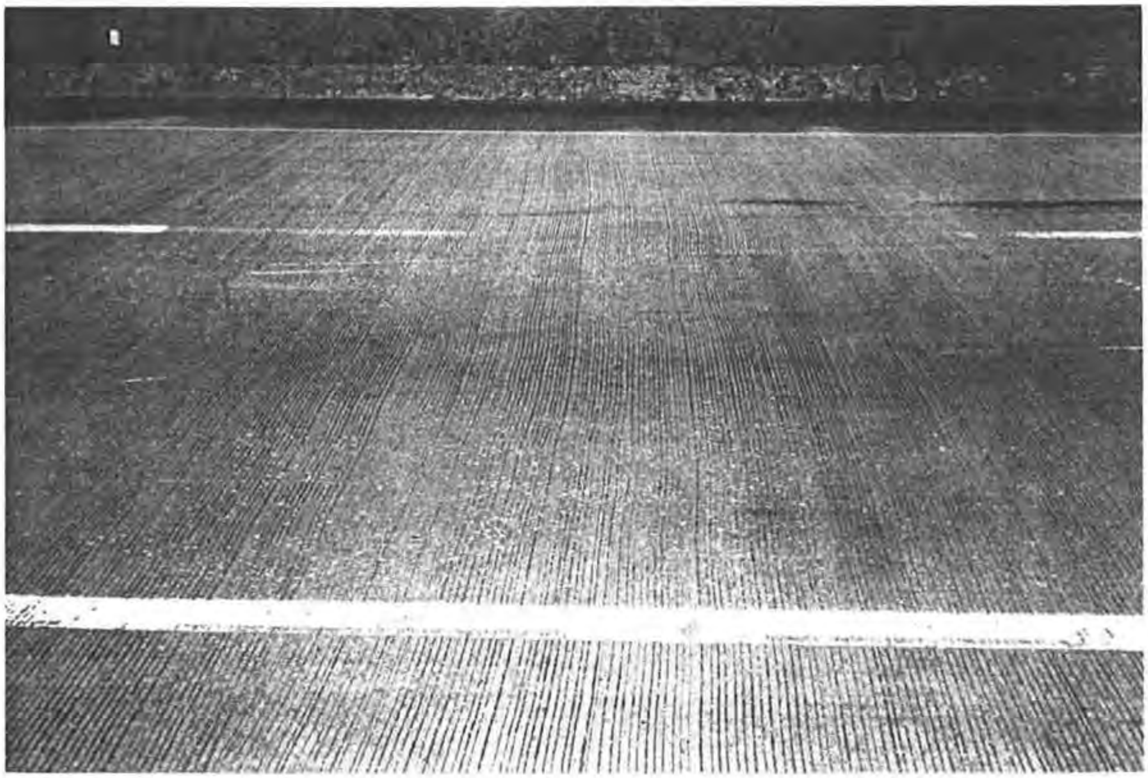


Figure 4.31. Evidence of Traffic Wear of Grooved Deck

Figure 4.32. Close-Up of Traffic Wear

Figure 4.33. Bad Scaling of SCC Guardrail

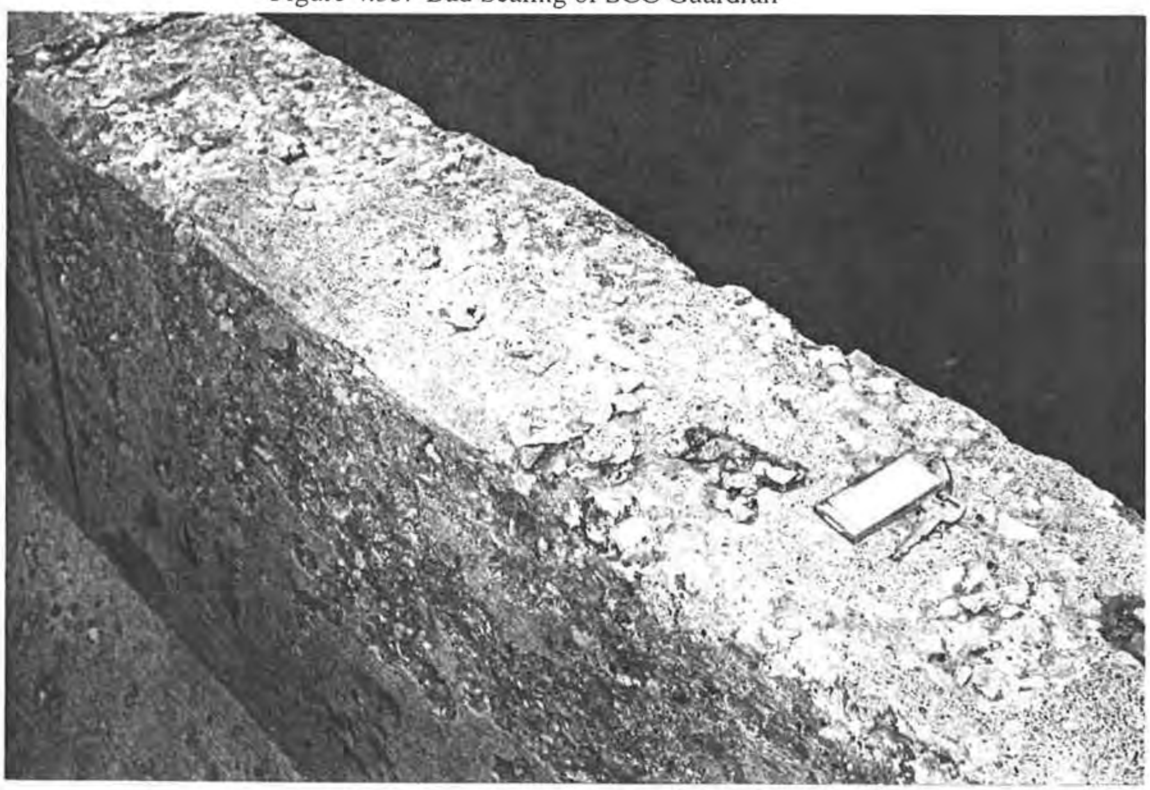


Figure 4.34. Close-Up of Guardrail Scaling

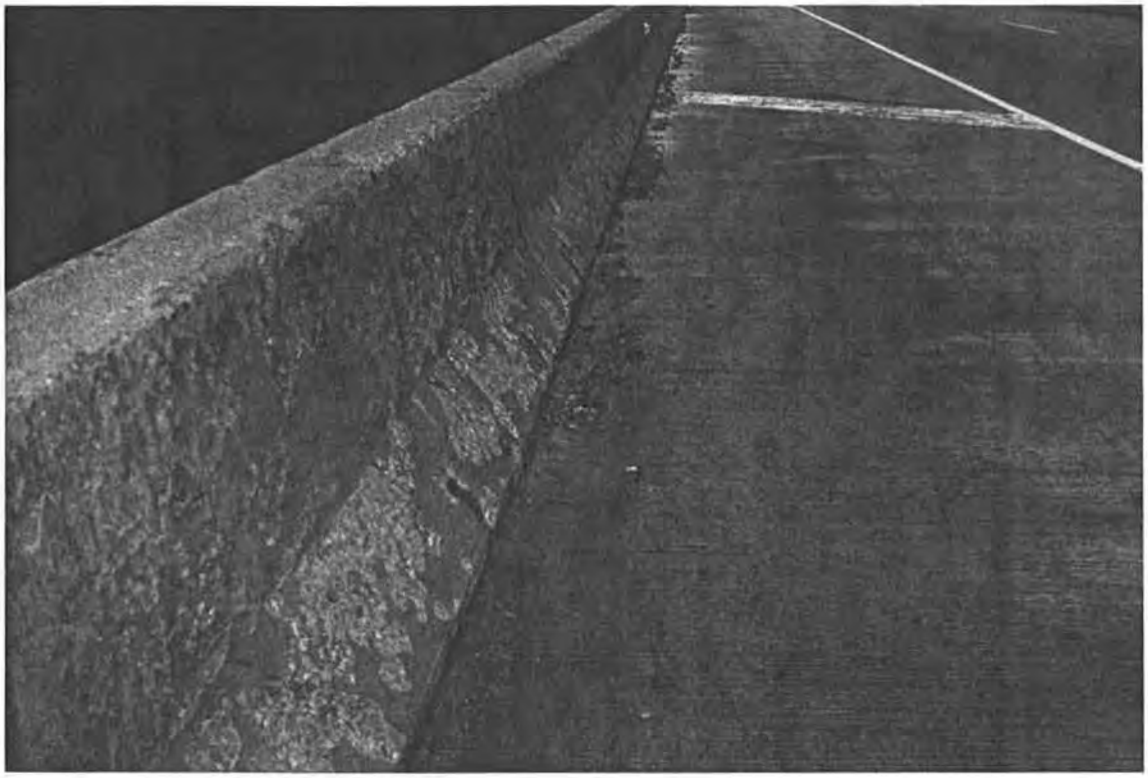


Figure 4.33. Bad Scaling of SCC Guardrail

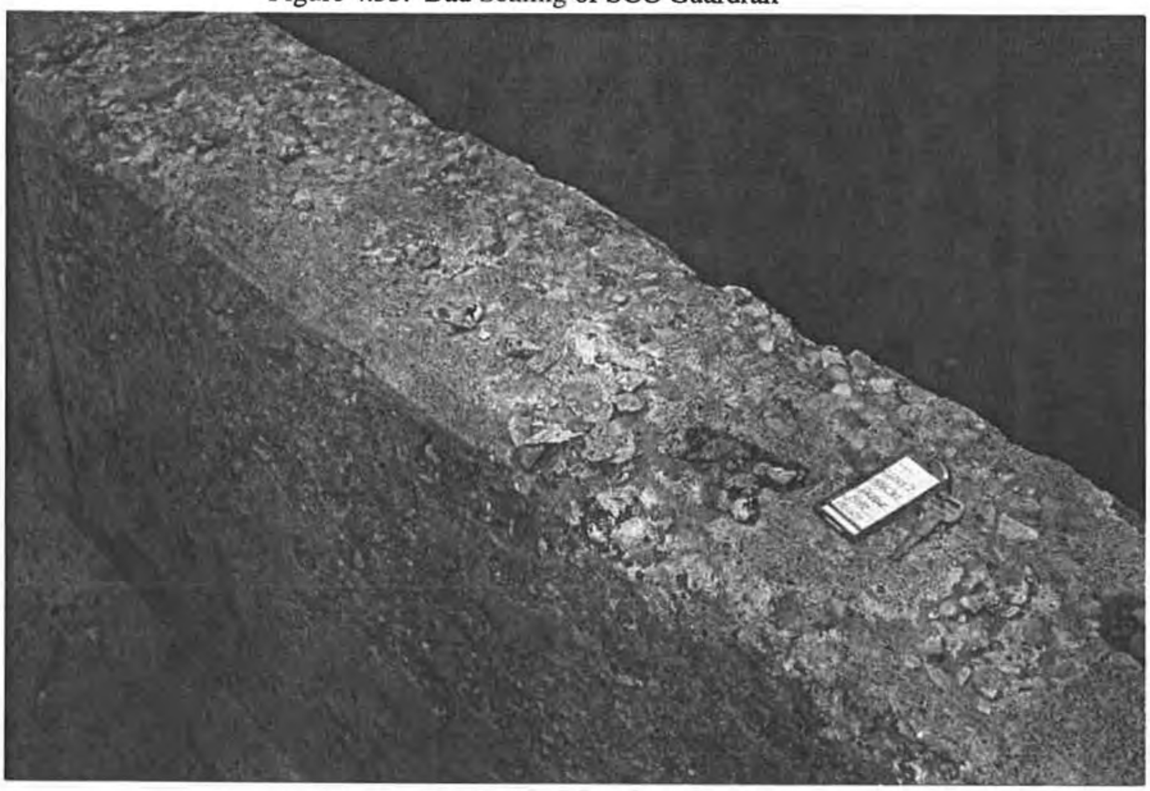


Figure 4.34. Close-Up of Guardrail Scaling

The photographs in Figures 4.35 - 4.38 show an older OTC bridge of unknown age. The bridge was highly skewed and the transverse deck crack near the end (probably caused by the large skew angle) shown in Figures 4.36 was one of the few cracks which we saw in all of the OTC bridge sites that we visited (approximately 10 sites). Figures 4.37 shows the underside of the bridge which appeared to be in mint condition. Figure 4.38 shows some severe corrosion of rebar and spalling at one of the PCC abutments due most likely to the use of deicing salts.

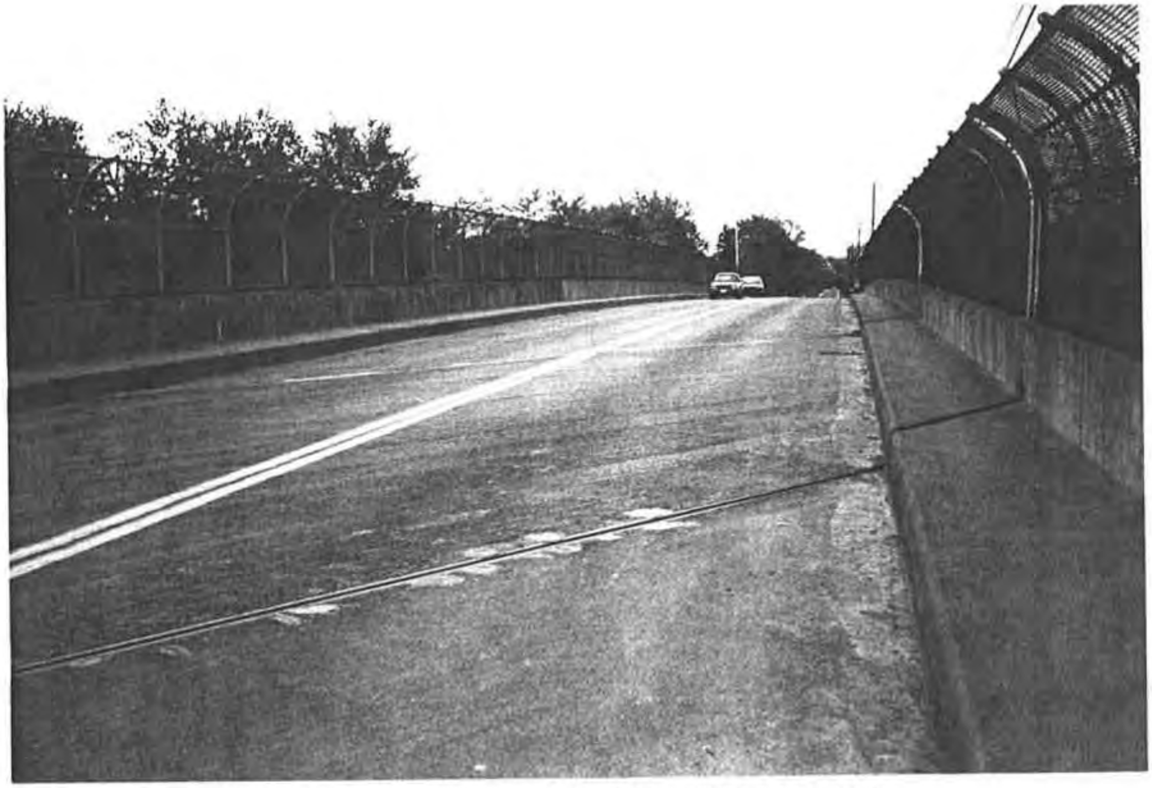


Figure 4.35. Highly Skewed and Older SCC Bridge Deck

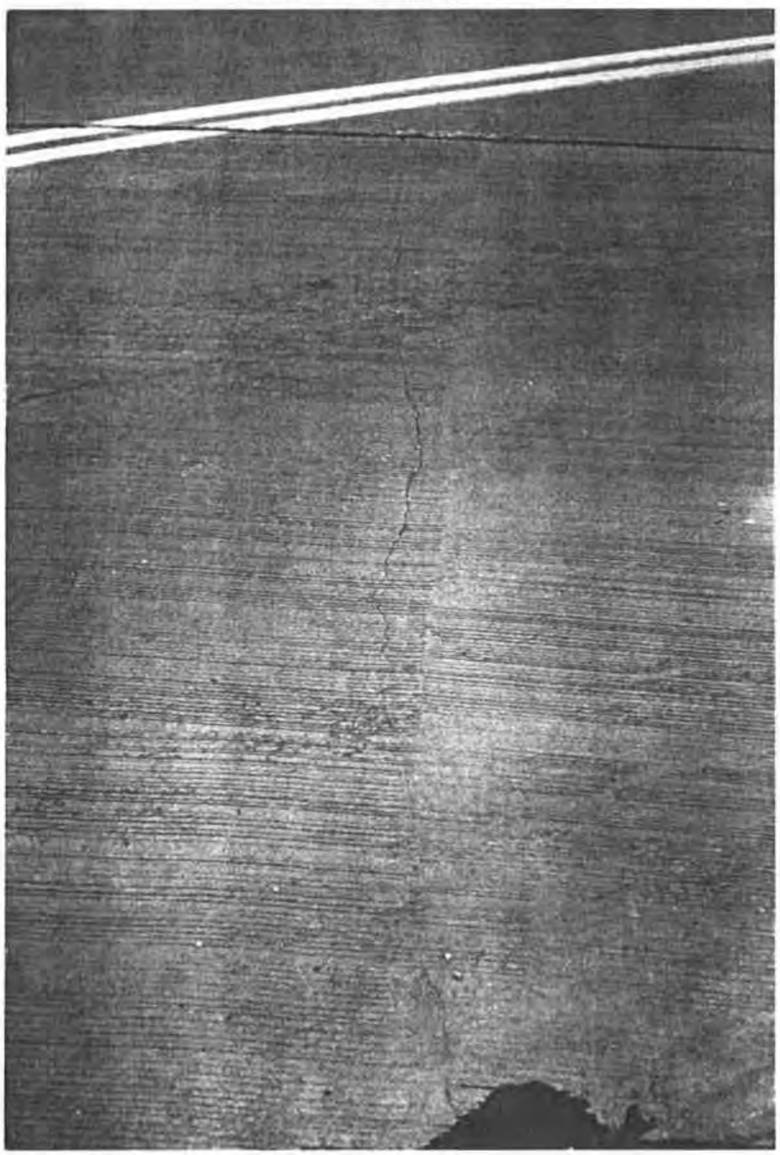


Figure 4.36. Transverse Crack
Near End of Skewed Deck

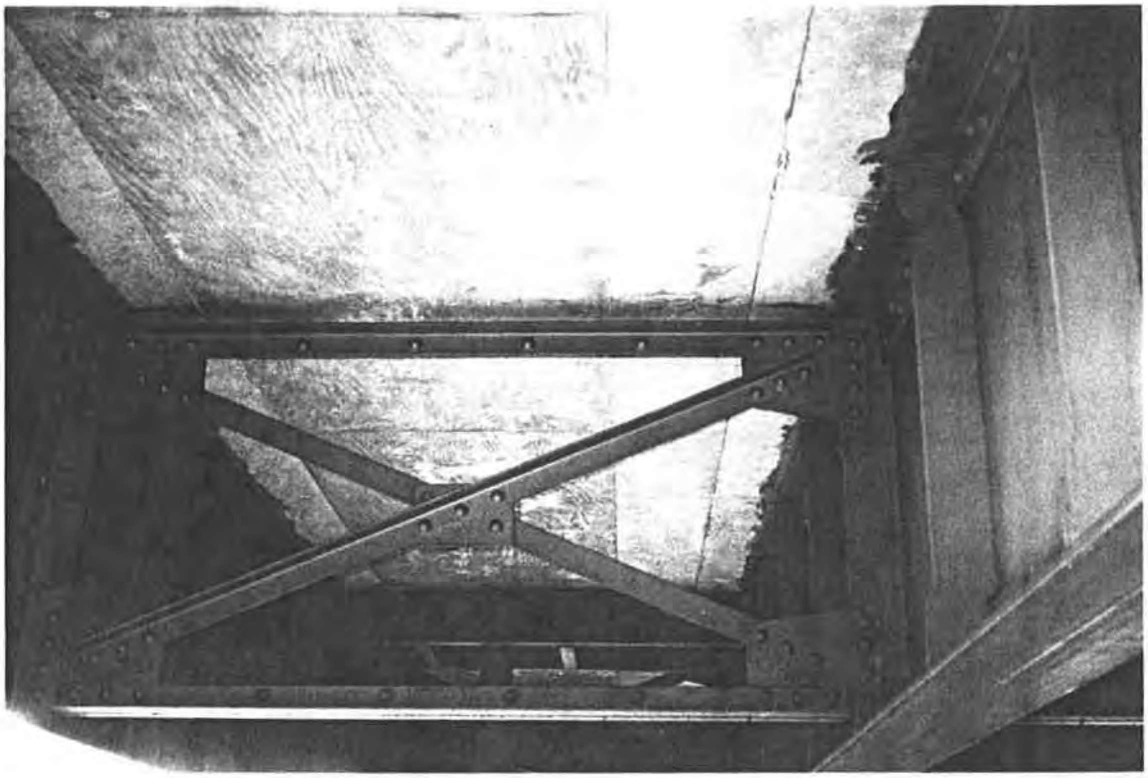


Figure 4.37. Excellent Condition of Underside of SCC Deck

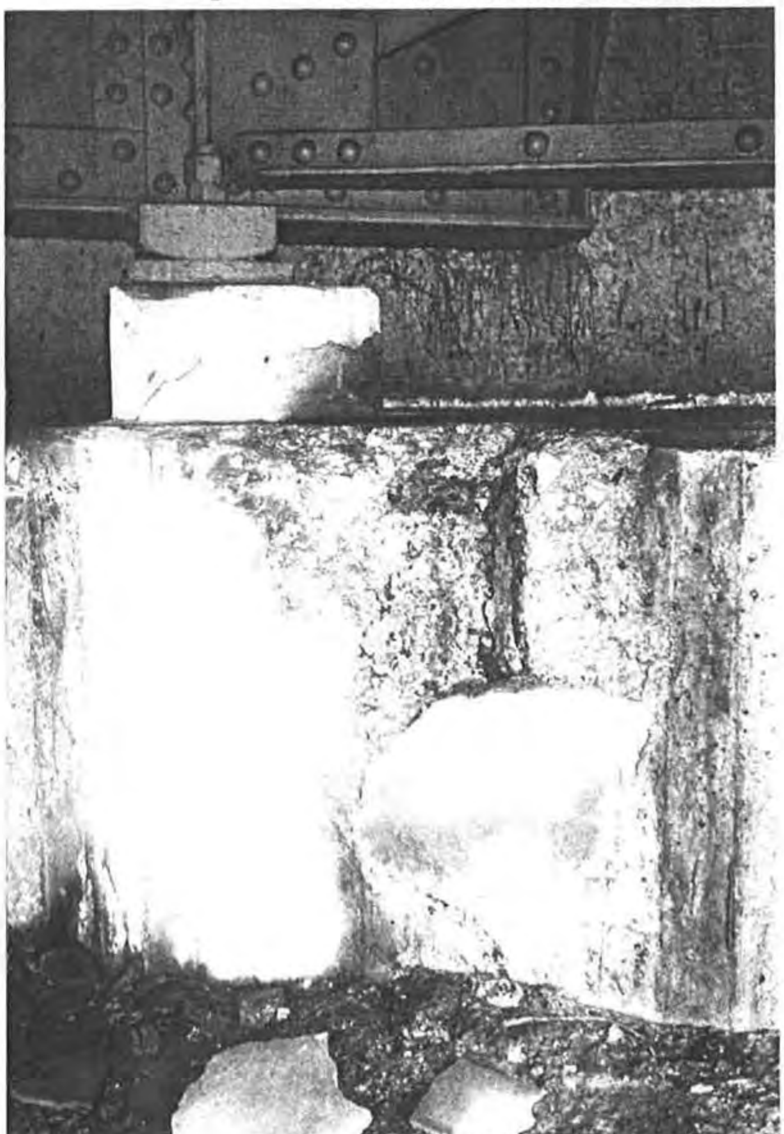


Figure 4.38. Rebar Corrosion and Spalling of PCC Abutment

We visited and photographed approximately seven other SCC bridge decks in OTC's system. However, they are not presented here because there was essentially no damage to photograph or report. With the exceptions of the guardrail scaling on the bridge shown in Figures 4.33 and 4.34, and the transverse deck crack shown in Figure 4.36, we saw no significant signs of other damage/deterioration. What we observed in our inspection of about ten SCC bridge decks versus the photographs that David Ransberry showed us earlier (see Figures 4.1 - 4.3) were dramatically different and very encouraging.

Following our field inspection, we met with OTC's Design Engineer, Mr. Kerry Ferrier who spoke to issues related to design considerations with SCC. He provided us with a copy of OTC's Supplementary Specifications which they use when using SCC decks. The PIs have modified these to make them compatible with ALDOT's specifications, and these are given in Appendix B.

4.3 New York Thruway Authority Visit

At the New York Thruway Authority (NYTA) Office in Albany, NY, we met with Research Engineer, Mr. Jim Golden. Mr. Golden had once worked for the Portland Cement Association, and this background along with his years of experience in research and development provided him with a unique background and experience related to SCC bridge decks. Mr. Golden indicated that the NYTA had placed 47 SCC bridge decks from 1991 - 1994 as follows:

1991 - 1

1992 - 1

1993 - 44

1994 - 1

He also indicated that he had visited with the OTC prior to placing the SCC, and basically adopted their mix design and most of their specifications. He also developed a 1- page set of Guidelines for Use of SCC, and gave us a copy. As indicated earlier, we combined the recommendations of the

OTC with the NYTA Guidelines to form a modified set of Guidelines and these are given in Appendix A.

Mr. Golden indicated that NYTA's early shrinkage cracking results were excellent, with minimal early cracking. Two of the bridges were of jointless/integral design, and these bridges did exhibit shrinkage cracking with the SCC as would be expected since the early expansion of the SCC could not take place because of boundary restraints. Subsequently, however, most of the decks developed severe scaling/durability problems to such an extent that the NYTA declared a moratorium on the use of SCC for bridge decks.

The NYTA brought in Construction Technology Laboratories (CTL) as a consultant to determine the cause of their severe deck scaling deteriorations, and CTL determined that they had very nondurable concrete and it was caused by

1. having a reduced air content due to pumping
2. having too high a slump and w/c ratio
3. not placing sufficient emphasis on curing.

CTL concluded that the poor durability properties of their deck concrete was due to (1) - (3) above and had nothing to do with the Type K cement or the fact that it was SCC.

Even though CTL's assessment of the problem and final report seemed to vindicate SCC, the NYTA has continued to not use SCC in its decks. Recent reports seem to indicate that the NYTA is experimenting with the use of SCC with microsilica as a mineral admixture for bridge deck concrete, and are encouraged by its excellent performance.

Mr. Golden took us to visit many of their SCC bridge decks and his earlier summation of their experiences with the performances of the SCC decks proved to be quite accurate. That is, what we saw were decks that showed virtually no shrinkage cracking, but some with very severe surface scaling damage. Photographs of typical deterioration or nondeterioration are shown in Figures 4.39 - 4.69.

The photographs in Figures 4.39 - 4.44 show the Dunnsville Bridge over the NY Thruway. The deck was replaced in stages, i.e., one lane in June 1993 and the other in October 1993. The centerline closure placement/concrete in Figures 4.39 and 4.40 is regular PCC and the lighter colored lanes are SCC. Some traffic lane wear is evident in Figure 4.39. There were a couple of minor cracks such as the one shown in Figure 4.41 in the top of the deck, and some minor spalling as shown in Figure 4.42. The spalling was only in the side drainage regions where deicing salts probably tend to accumulate. Also, there was some spalling of the SCC adjacent to transverse joints as indicated in Figures 4.43 and 4.44.

Figure 4.39. Dunnsville Bridge with Replacement Deck of SCC

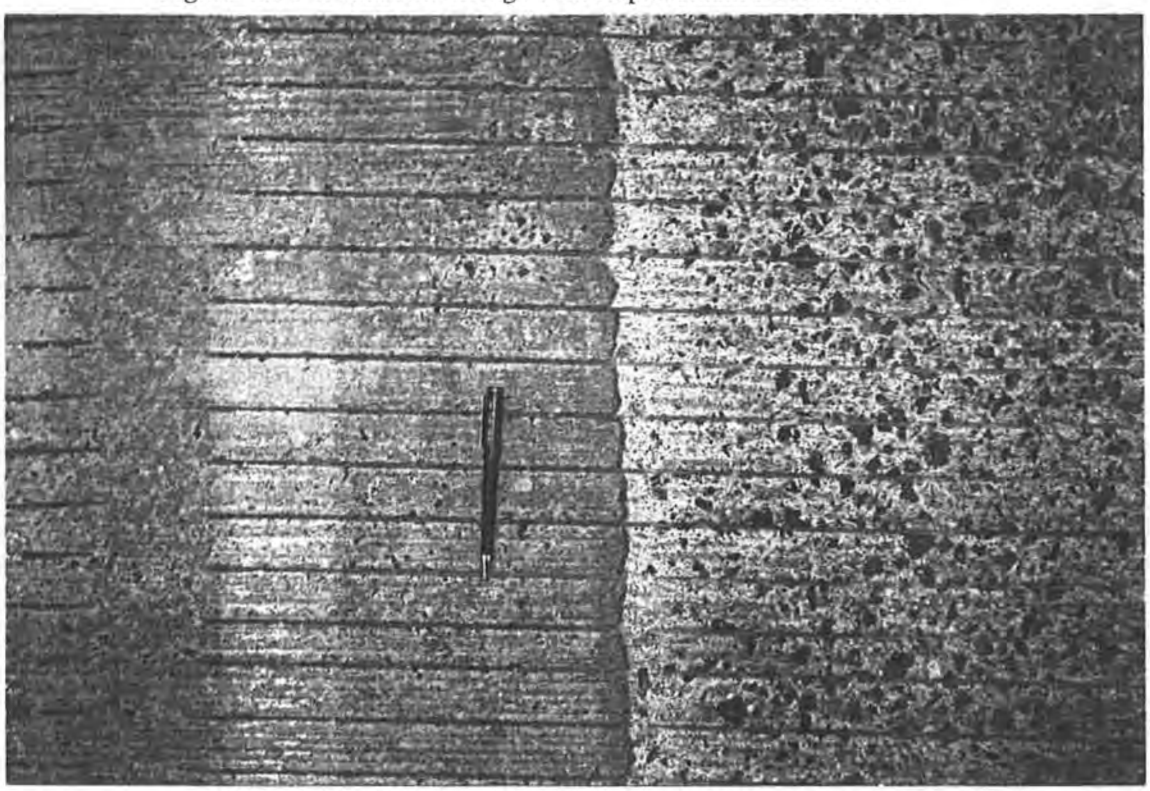


Figure 4.40. Close-Up of Longitudinal PCC/SCC Joint

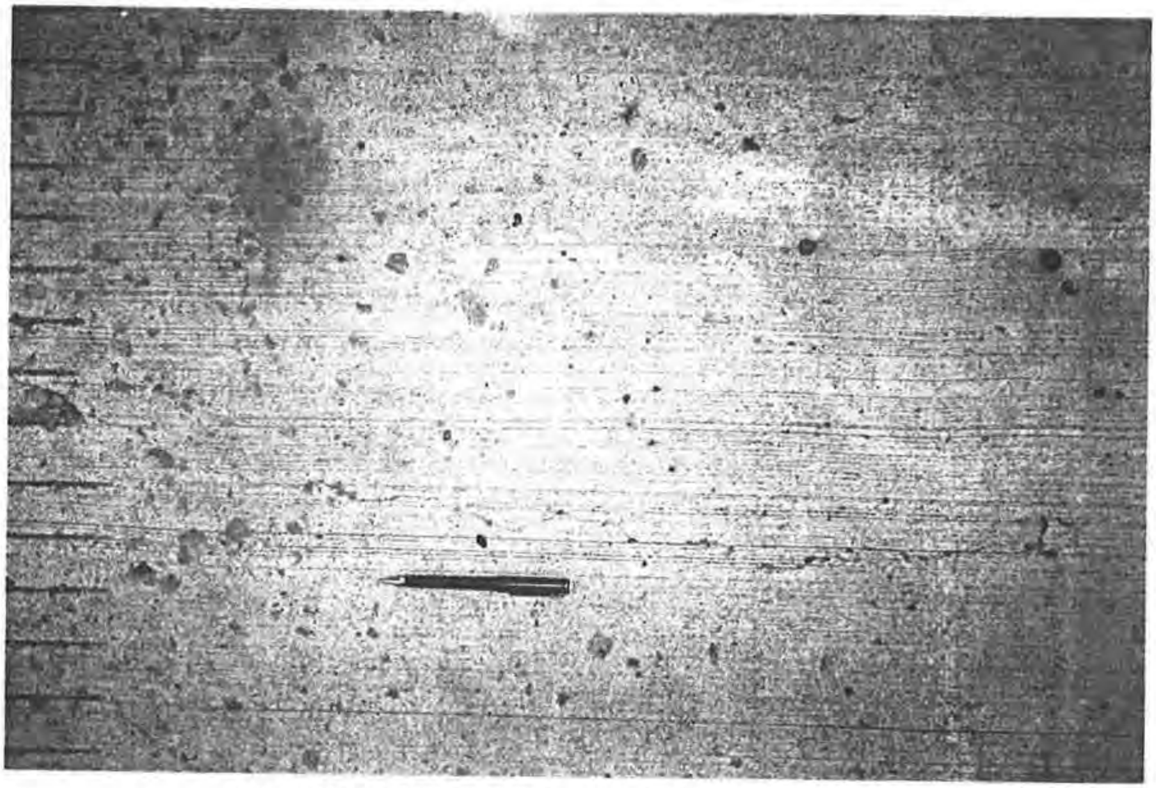


Figure 4.41. One of Very Few Light Transverse Cracks

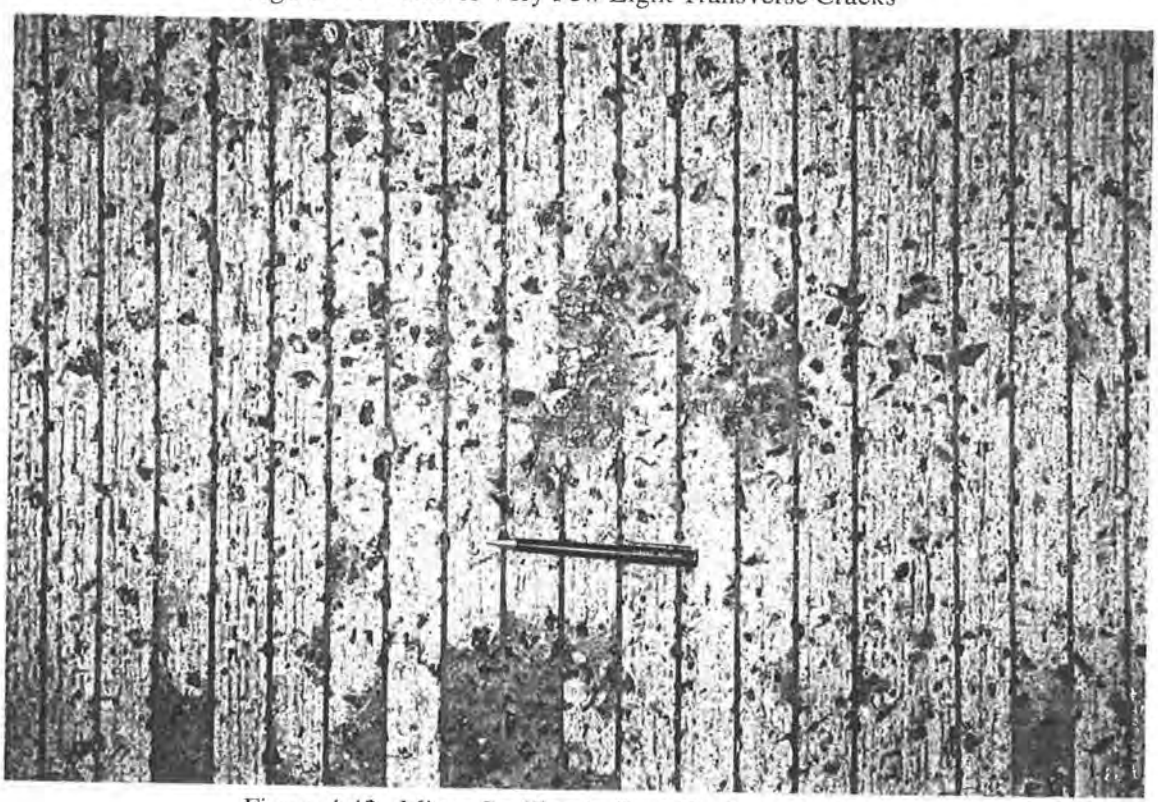


Figure 4.42. Minor Spalling in Deck Drainage Region

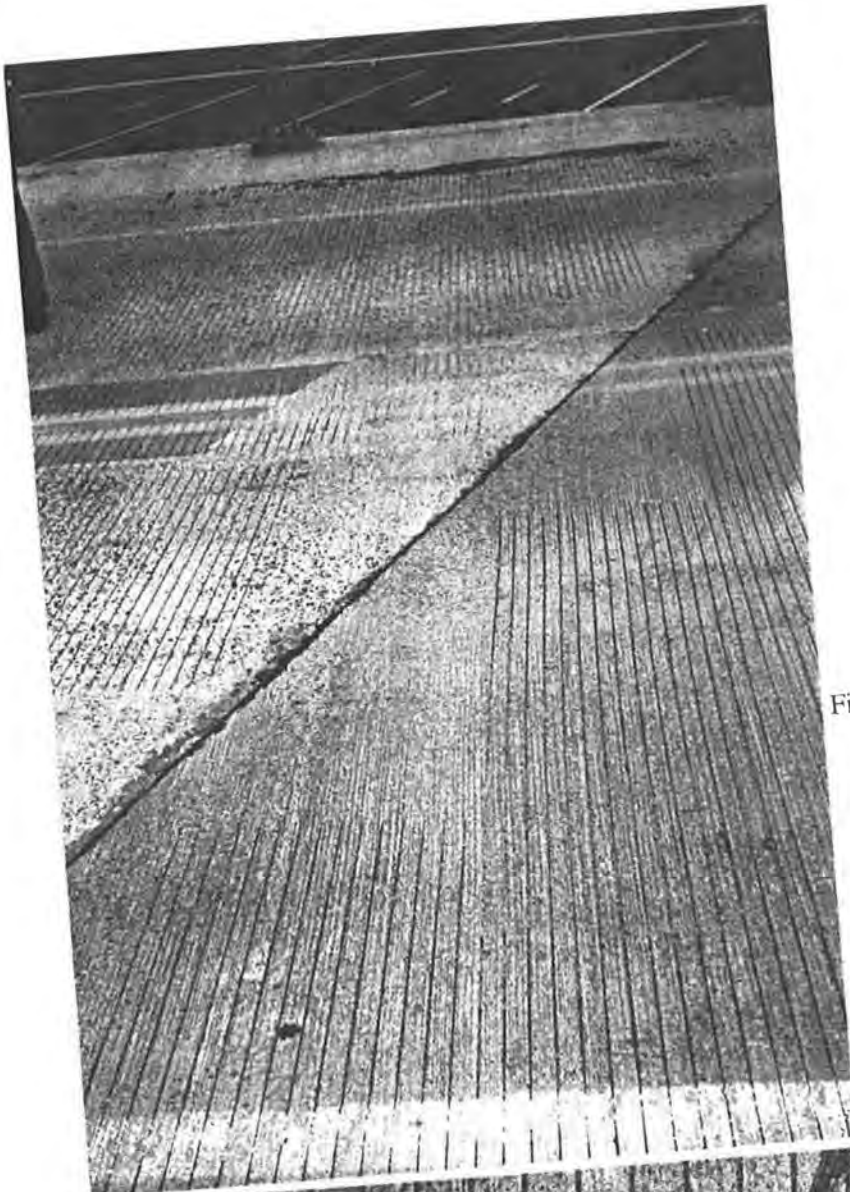


Figure 4.43. Spalling at Deck - Approach Slap Joint

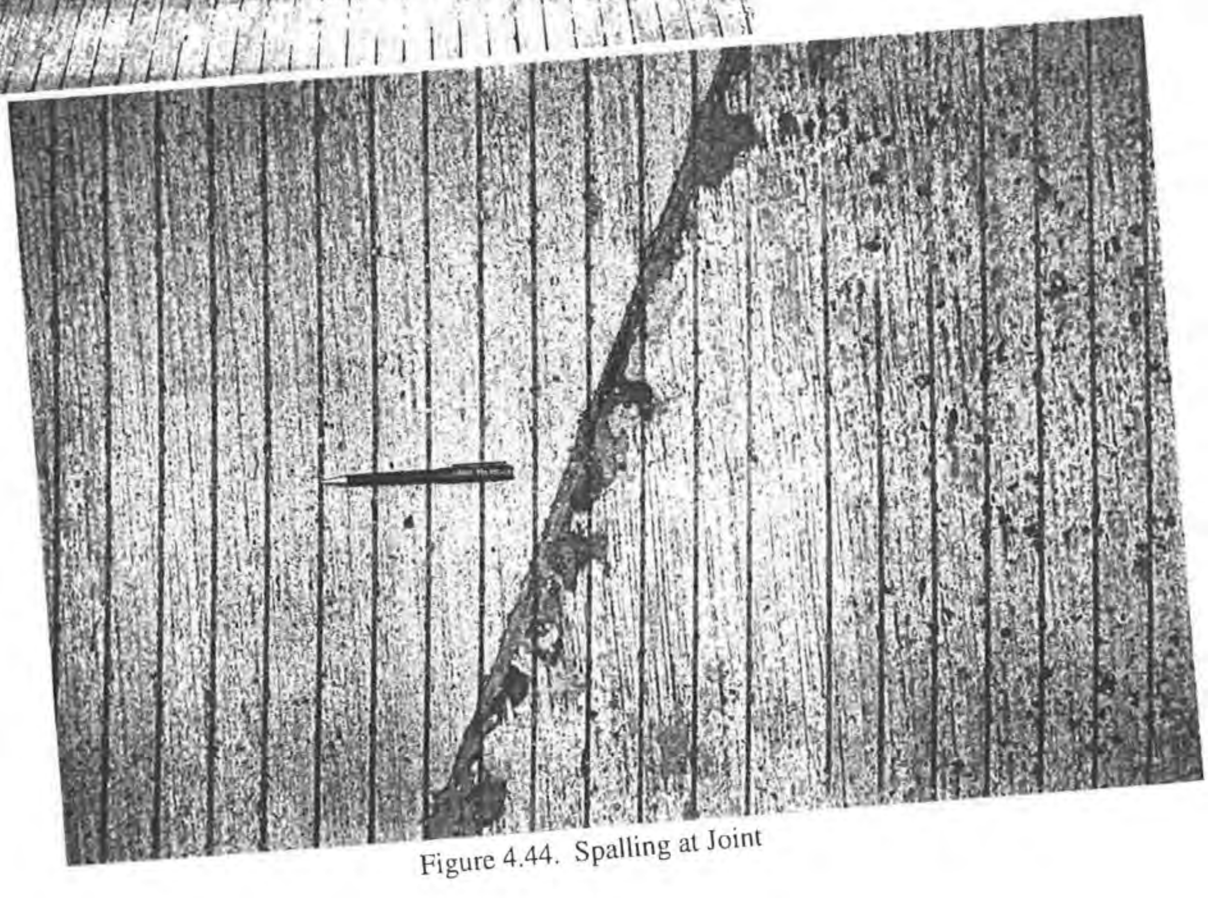


Figure 4.44. Spalling at Joint

The photographs of Figures 4.45 - 4.50 show the bridge carrying State Route 146 over the NY Thruway. This was also a staged deck replacement with SCC, where the deck for one lane was replaced in October 1992, and the other in June 1993. Figures 4.45 and 4.46 show overall views of the deck with the center closure concrete placement more evident in Figure 4.46. Figure 4.47 shows a medium width transverse crack close to and paralleling the abutment joint. Figure 4.48 shows some minor surface spalling and major paint wearing/debond. Figures 4.49 and 4.50 show some significant surface spalling in the wheel path.

Figure 4.45. Overall View of Route 146 Bridge Over the NY Thruway

Figure 4.46. SCC Deck Showing Center Closure Concrete

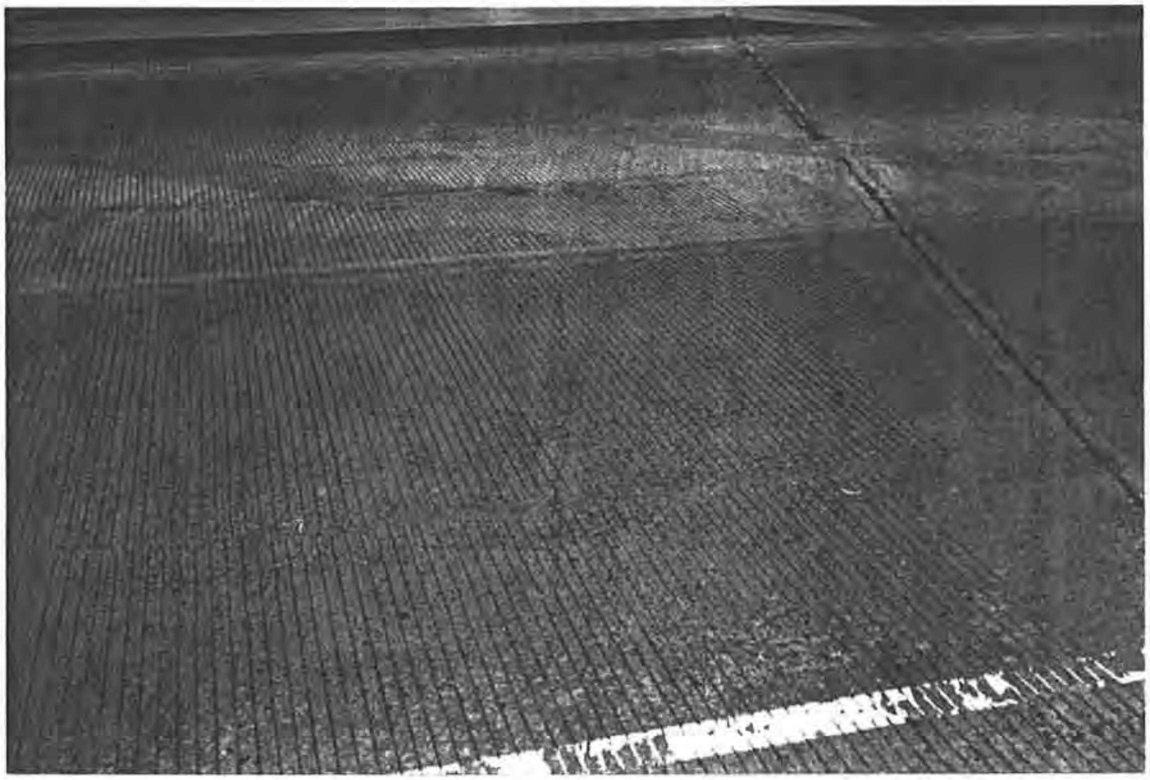


Figure 4.47. Transverse Crack Near Abutment Joint

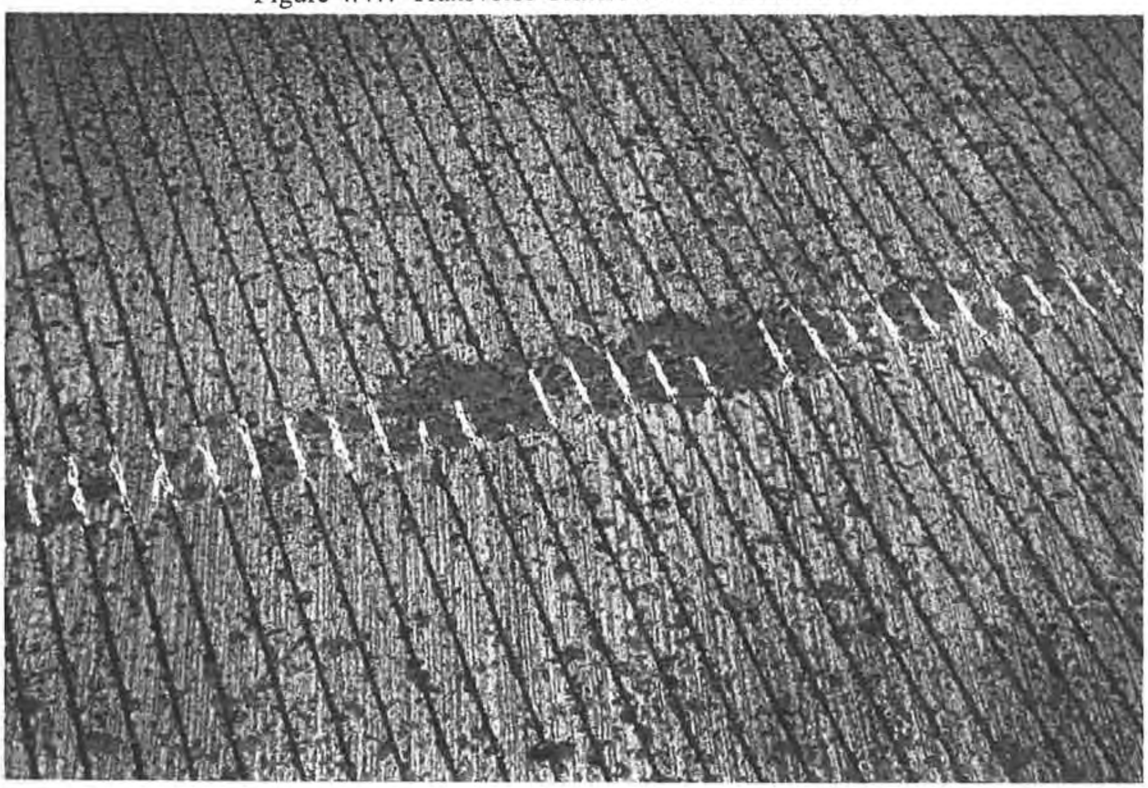


Figure 4.48. Surface Spalling and Paint Wearing

Figure 4.49. Surface Spalling in Wheel Path

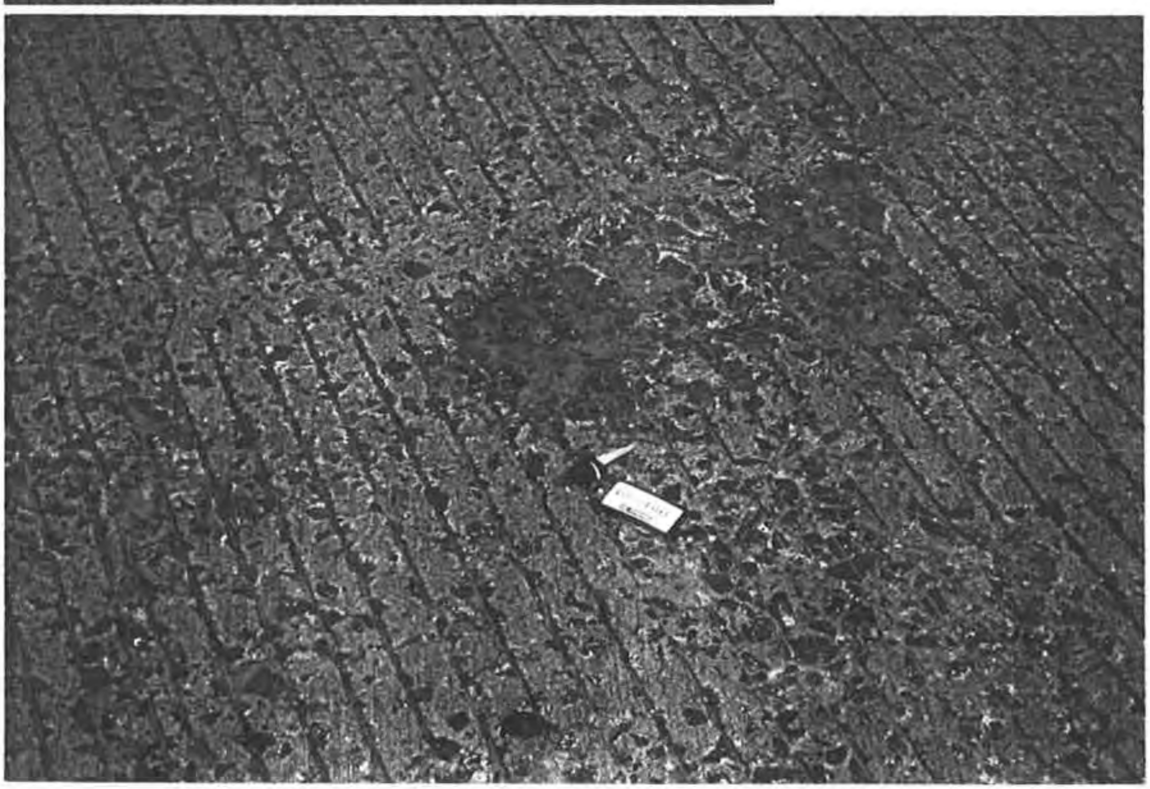


Figure 4.50. Close-Up of Surface Spalling in Wheel Path

The photographs in Figures 4.51 - 4.56 show one traffic direction of the NY Thruway over State Route 9. The original PCC deck was replaced in 1993 with a SCC deck with an asphalt protective/wearing surface. Overall views of the 3 simple span-steel girder bridge are shown in Figures 4.51 and 4.52. Figures 4.53 and 4.54 show severe spalling of the outer curbs of the bridge. Figure 4.54 is at the abutment which is of painted (peeling) PCC. Figure 4.55 shows some spalling and deterioration of the SCC along a bridge expansion joint. Other than the side curbs and strips adjacent to the expansion joints, the SCC deck was only visible from underneath the bridge. An inspection of the decks underside showed no evidence of cracking or deterioration. On the underside, the deck appeared to be in mint condition as evident in Figure 4.56.

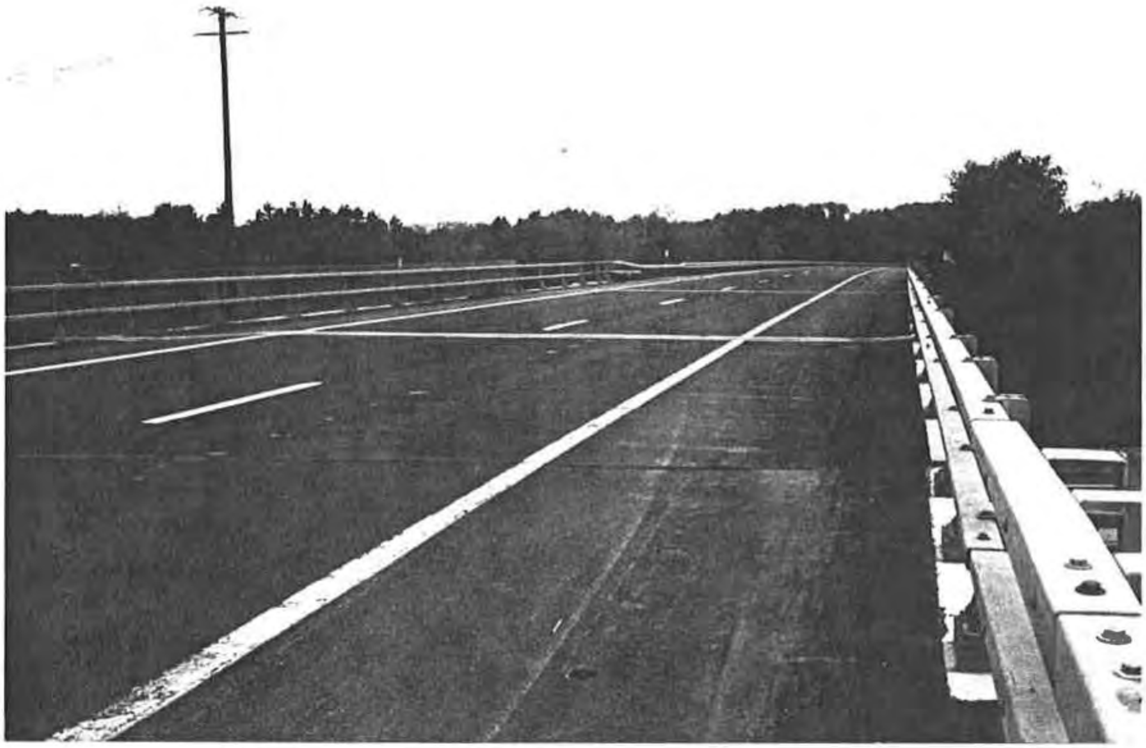


Figure 4.51. Overall View of NY Thruway Bridge with Asphalt Wearing Surface

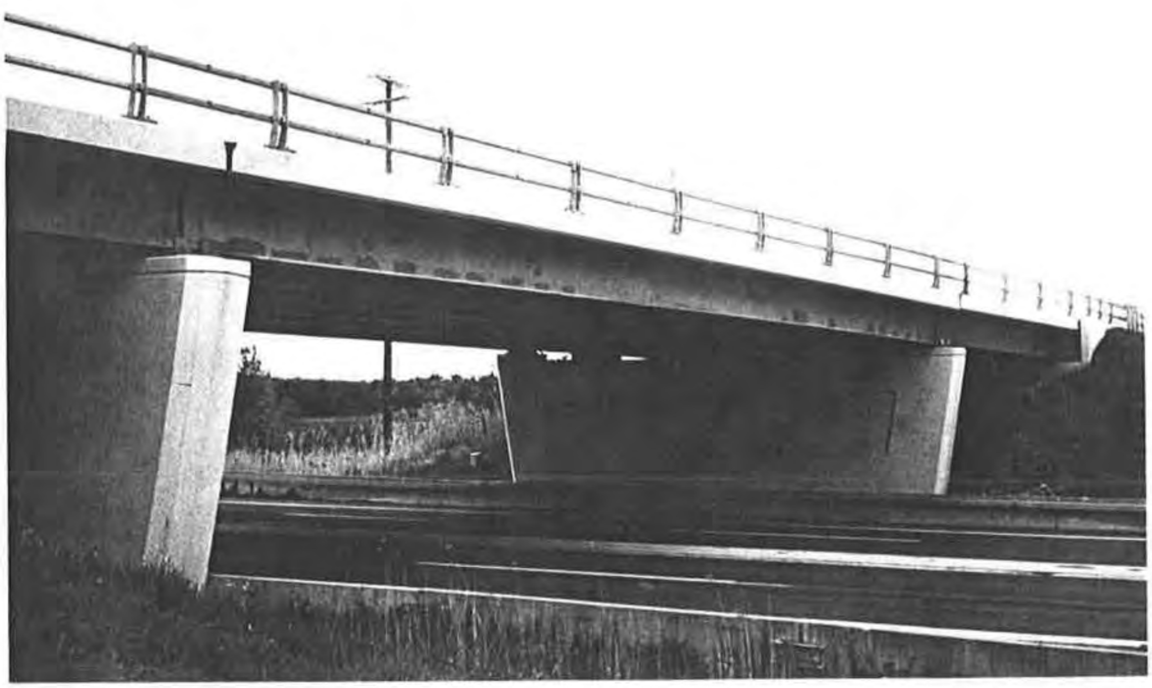


Figure 4.52. NY Thruway Bridge Over State Route 9

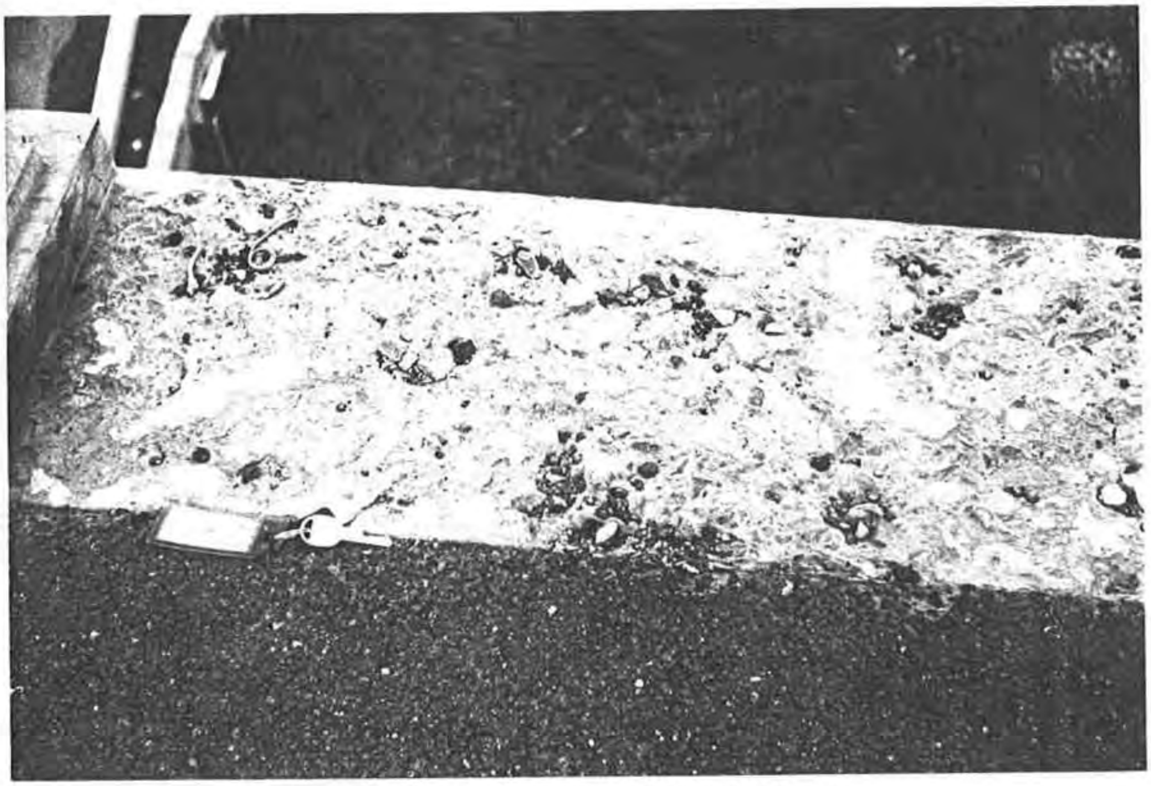


Figure 4.53. Severe Spalling of SCC Deck Curb

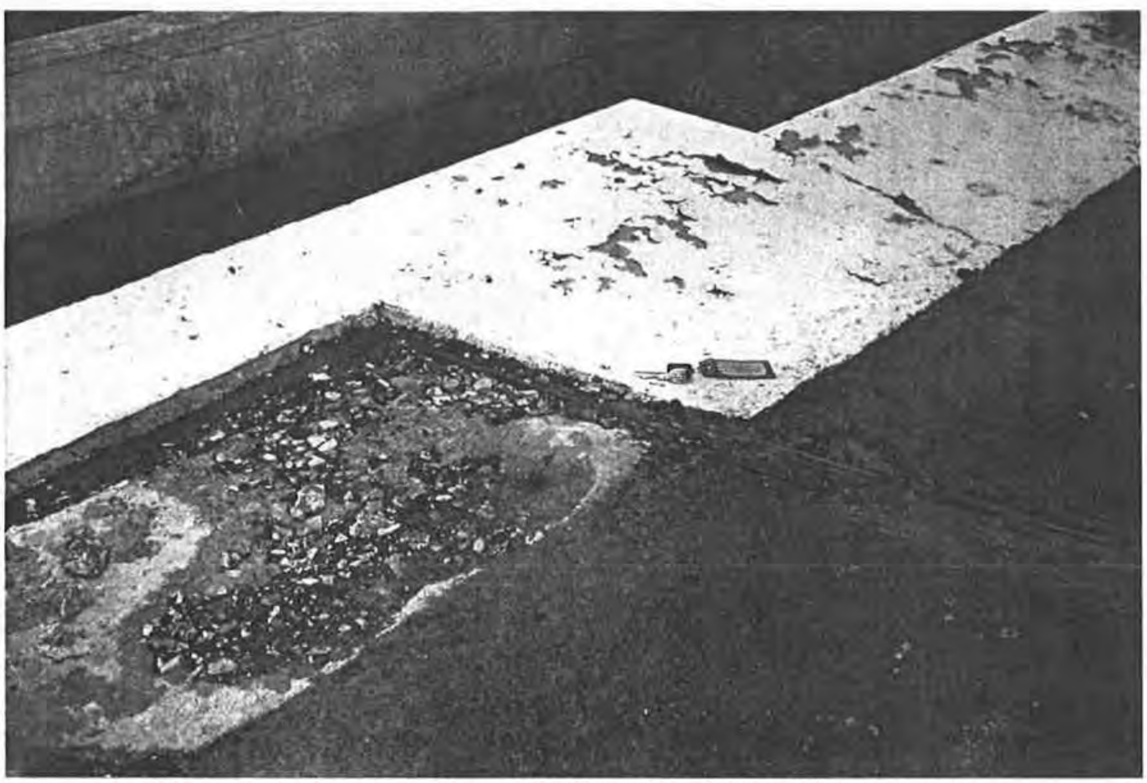


Figure 4.54. Severe Spalling of SCC Deck Curb Near PCC Abutment

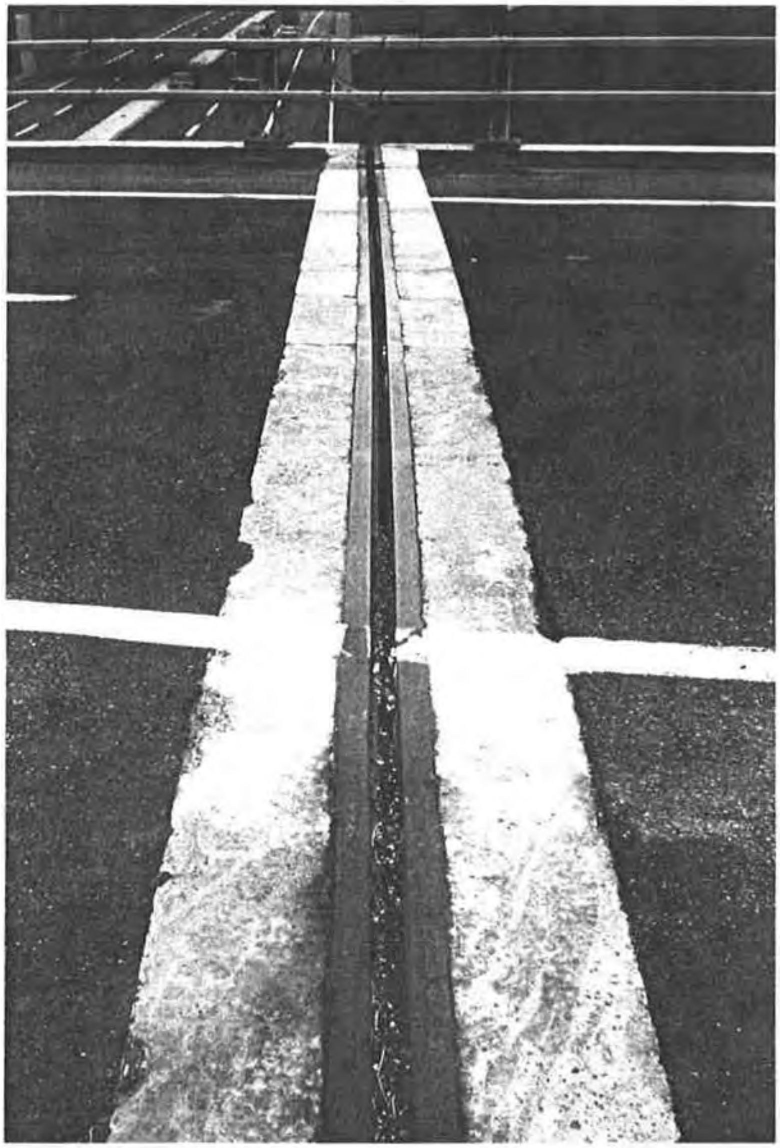


Figure 4.55. Spalling and Deterioration of SCC Adjacent to Joint

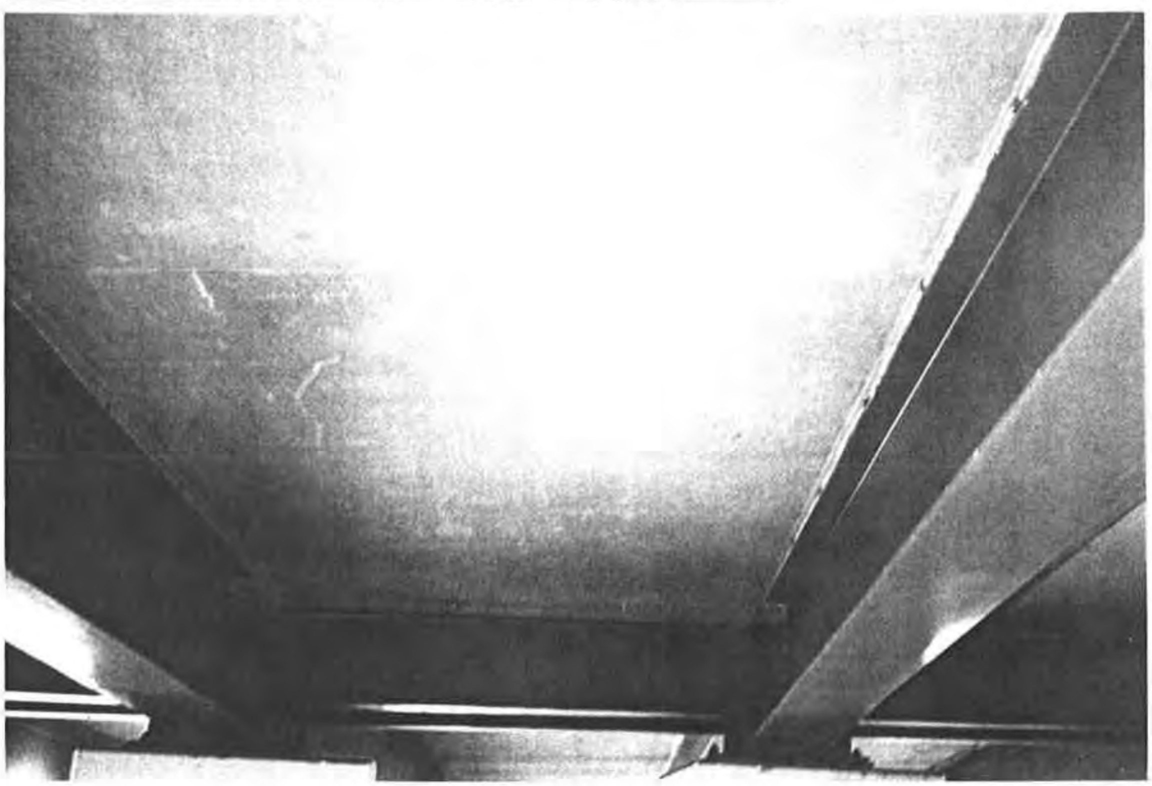


Figure 4.56. Mint Condition of Underside of Deck

The photographs in Figures 4.57 - 4.66 show one traffic direction of the Muitzes Kill Bridge of the NY Thruway. As with the previous set of photographs, this bridge's PCC deck was replaced with a SCC deck in 1993. The bridge was designed to have an asphalt protective/wearing surface, and thus the only concrete visible for inspection was at the curbs, expansion joints, and the underneath side of the deck. This bridge had even greater spalling deteriorations then the bridge over State Route 146. As can be seen in Figure 4.57, the bridge was of the same design and very similar to the previous bridge. However on this bridge, there was some spalling of the SCC concrete at locations along the bottom of the curb as well as along its top as can be seen in Figure 4.58. Figures 4.59 and 4.60 show severe spalling all along the curb. Figures 4.61 and 4.62 show some of the underside of curb spalling which was quite severe at a couple of locations. Also, severe spalling deterioration was evident at the transverse expansion joints as can be seen in Figures 4.63 and 4.65. Again, a close inspection of the underside of the deck found it to be devoid of cracks and from all appearances to be in mint condition as evident in Figures 4.65 and 4.66.

Figure 4.57. Muitzes Kill Bridge on NY Thruway

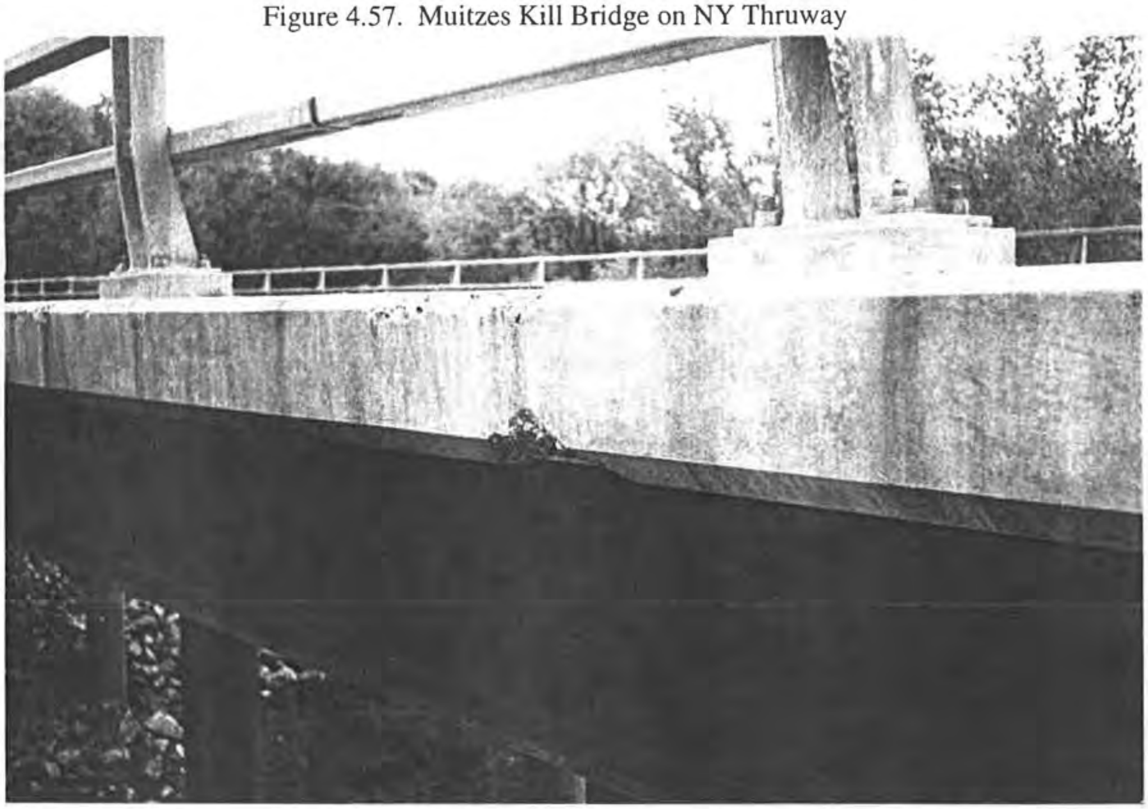


Figure 4.58. Spalling Deterioration at Top and Bottom of SCC Curb

Figure 4.59. Severe Spalling Along Curb

Figure 4.60. Additional Spalling Along Curb

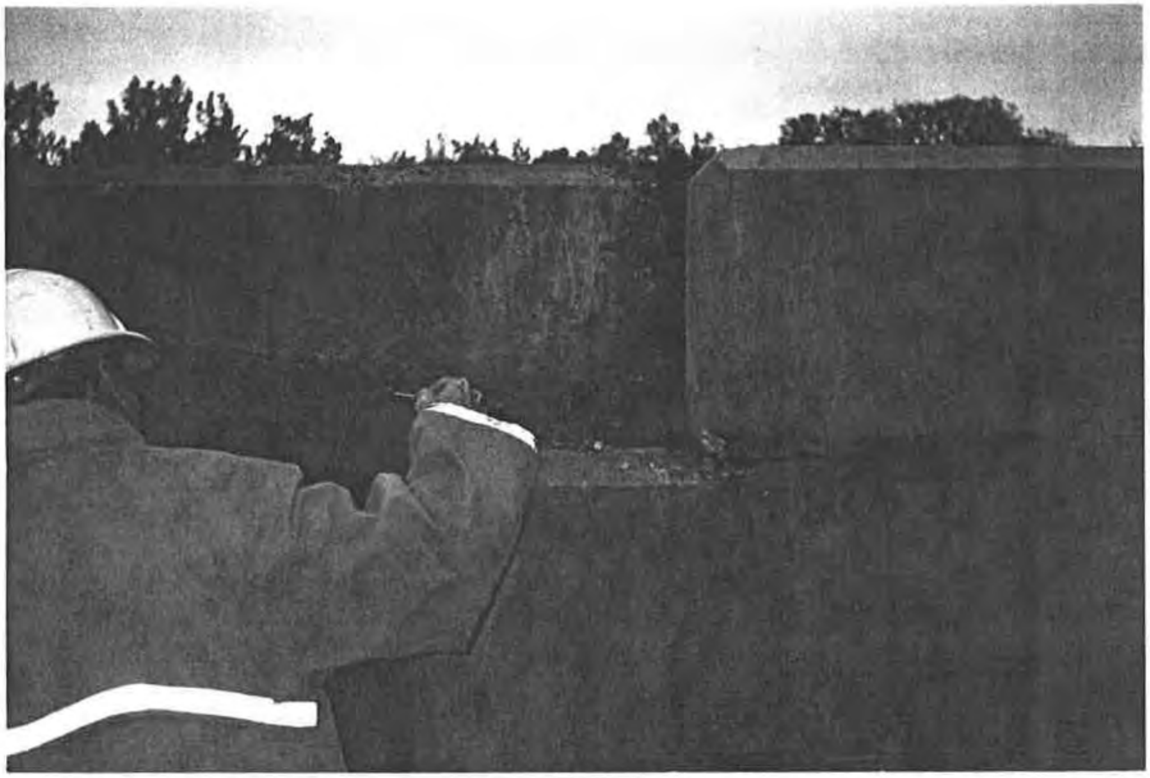


Figure 4.61. Severe Spalling at Underside of Curb Near Abutment

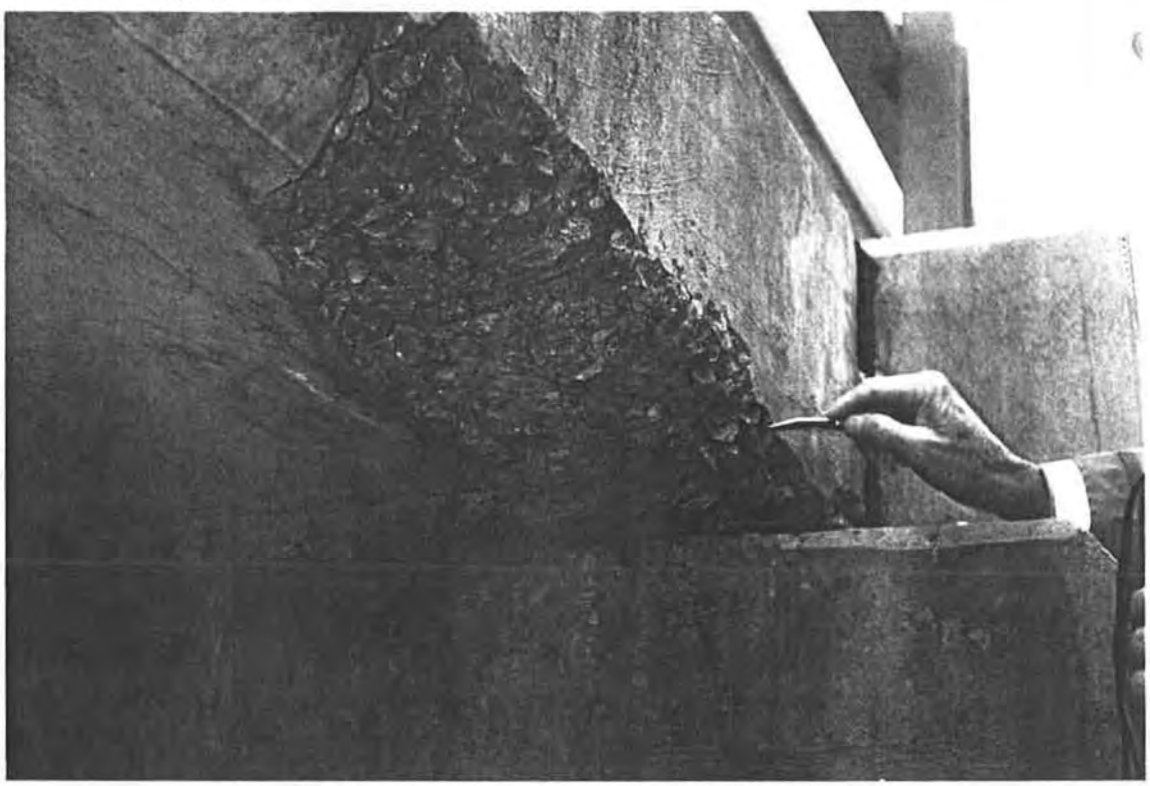


Figure 4.62. Close-Up of Underside of Curb Spalling

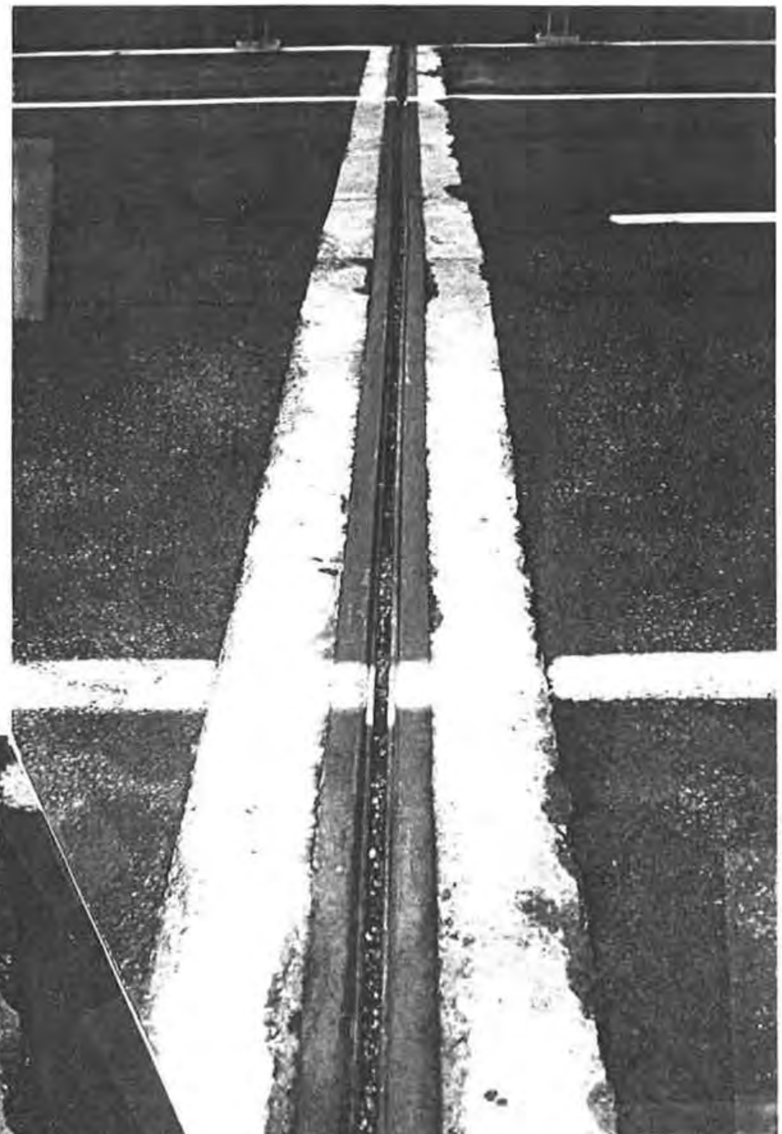


Figure 4.63. Spalling Deterioration of SCC Along Deck Joint

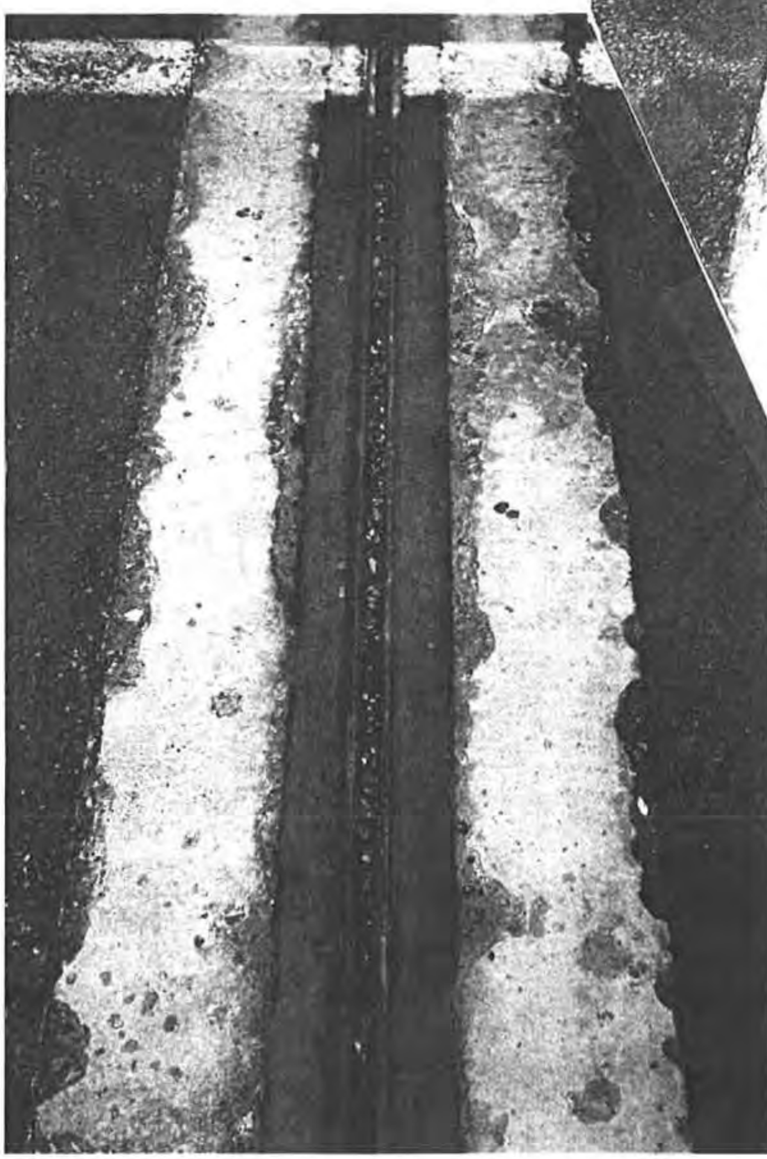


Figure 4.64. Close-Up of SCC Spalling at Joint

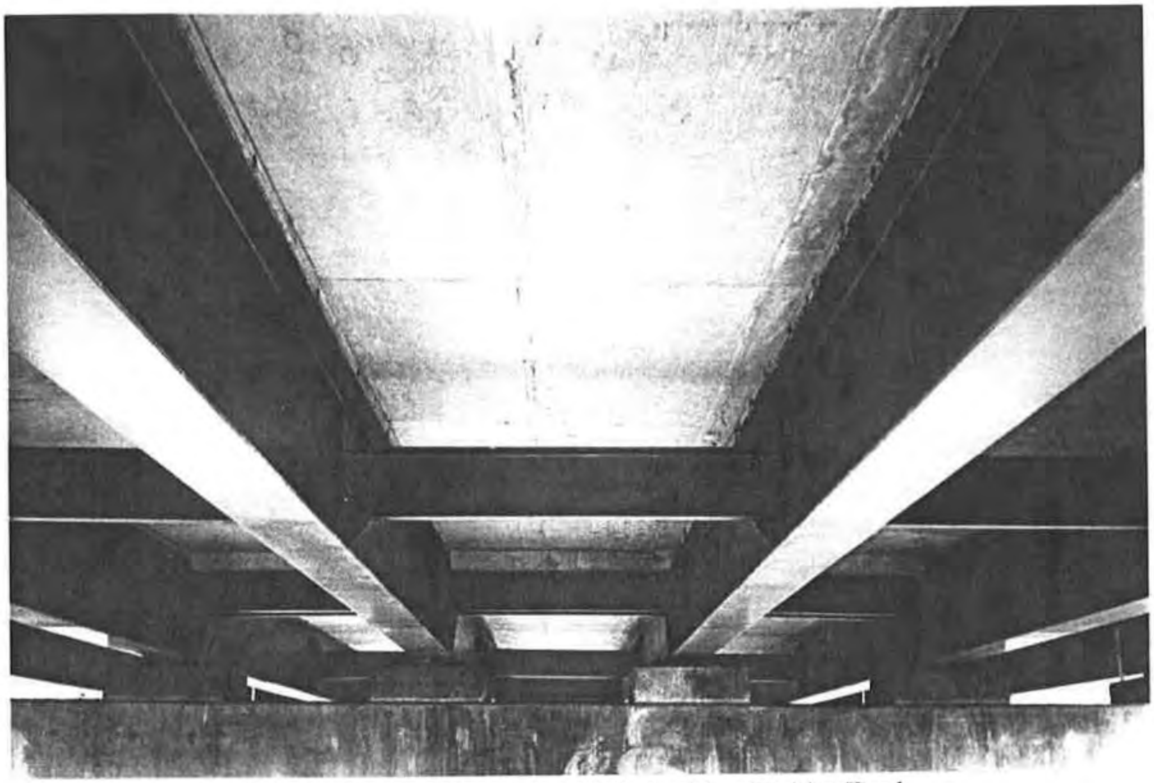


Figure 4.65. Mint Condition of Underside of Bridge/Deck

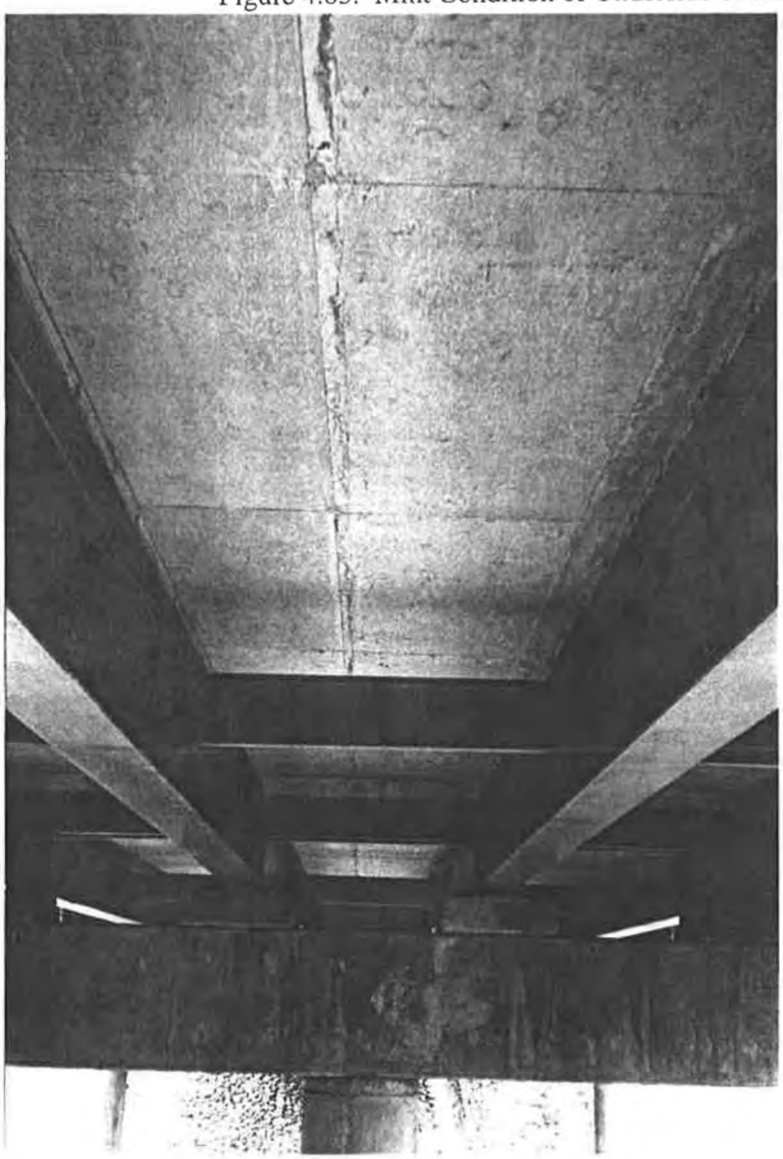


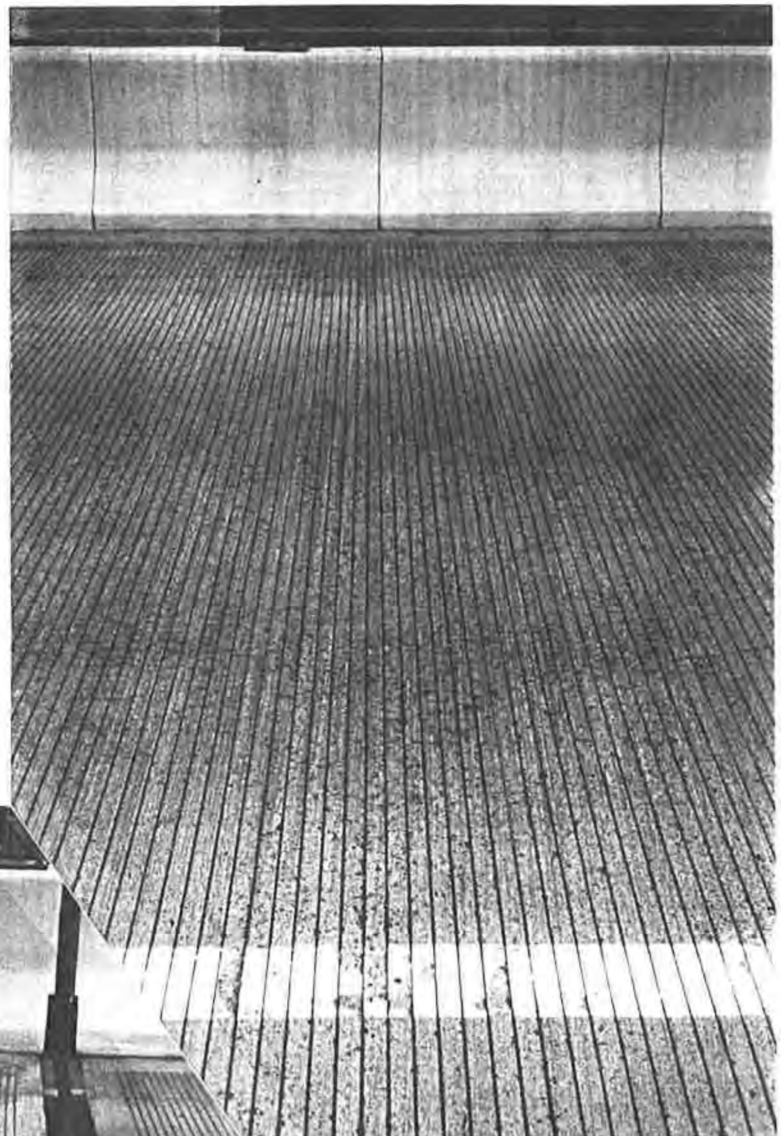
Figure 4.66. Close-Up of Crack
Free Underside of Deck

The photographs in Figures 4.67 - 4.69 show the relatively new Crossgates Mall Flyover. This bridge was constructed in 1993, however, it was not opened to traffic until recently and had never been exposed to deicing salts (as of the taking of these photographs). Additionally, the flyover carries minimal truck traffic. The bridge is about half straight (in the foreground of Figure 4.67) and half curved (in the background of Figure 4.67). An inspection of the top and bottom of the bridge deck found it to be essentially crack free (a couple of tight and short cracks were observed). Figures 4.68 and 4.69 show the excellent condition of the SCC deck and an expansion joint.

Other NOTA bridges with SCC deck were inspected and photographed. However, they are not presented here as the ones that have been discussed are representative of those that we inspected.

Figure 4.67. Crossgates Mall Flyover SCC Deck

Figure 4.68. Crack Free Deck



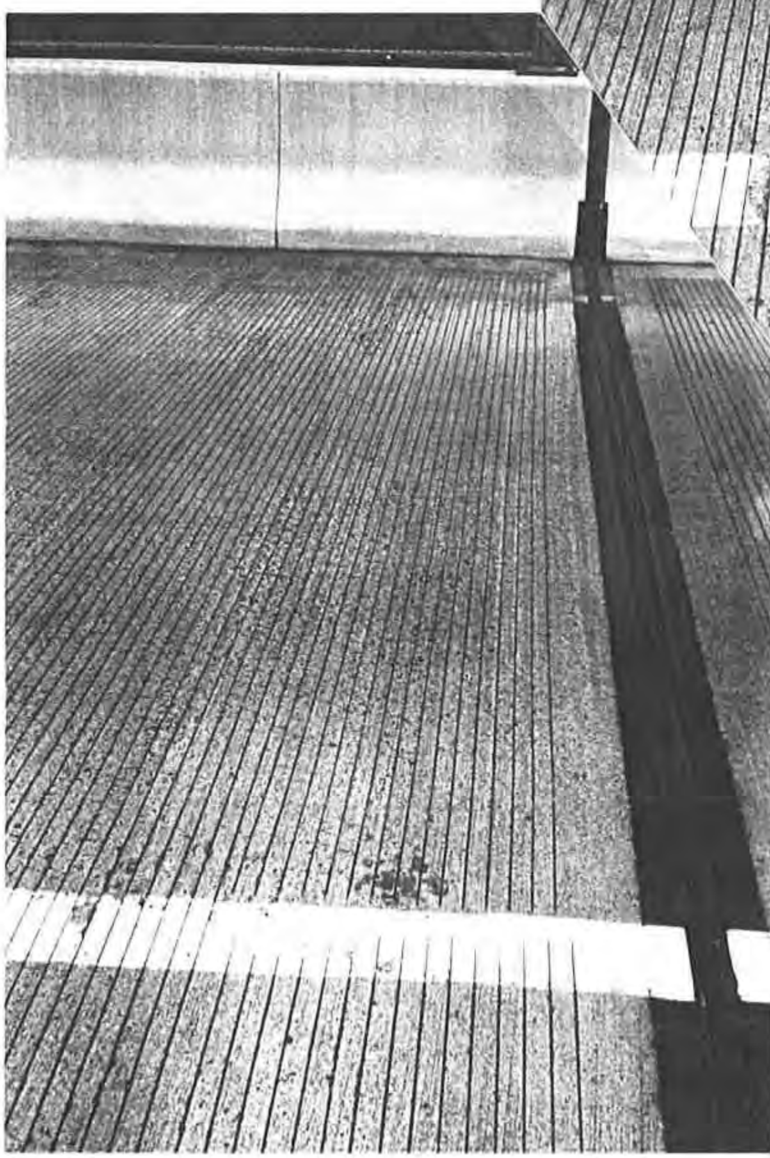


Figure 4.69. Excellent Deck and Joint Conditions

5. LABORATORY TESTING PROGRAM

5.1 General

As indicated in the Research Plan outlined in Chapter 3, the purposes of the laboratory testing programs were to

1) perform trial mix designs/testing to identify four candidate mixes for follow-up field evaluations on test bridges,

2) perform verification testing, property evaluation testing, and sensitivity testing of the four mix designs from (1) above.

This testing would allow us to compare the relative merits of the four mix designs for bridge deck applications, and later position us to recommend deck mix designs for field testing. The testing would also allow us to better understand and correlate later field test bridge results, and thus to make more informed decisions on the future use of one or more of the test concrete mixes for bridge decks.

5.2 Raw Materials Used

Mixes were formulated using Type I Portland cement, Type K expansive cement, microsilica admixture, air-entraining agent, and water-reducing agent. Cement used in the study was obtained from Blue Circle Cement Company based in Marietta, Georgia. Type I cement was Magnolia Brand Portland Cement, by Blue Circle and was manufactured to meet the requirements of ASTM C150 and Federal Specification SS-C-1960/3. Type K cement was Type K Expansive Cement manufactured by Blue Circle Cement Company. Microsilica used for the study was from Master Builders Technologies and was aluminum silicate.

Air-entraining agent was applied to each mix in order to provide entrained air in each concrete mix in an attempt to reach state specification for air content. The air-entraining agent used for this study was Microair manufactured by Master Builders Technologies and was in accordance with ASTM C260. Water-reducing agent was applied to mixtures in an attempt to achieve state specification of slump with use of set water-cement ratios and also provide improved workability to the mixes. Water-reducing agent for this project was Rheobuild 1000 manufactured by Master Builders Technologies and met requirements of ASTM C494 Type A and F.

Coarse and fine aggregates typical of the region were selected for the mixes and were provided by Blue Circle Cement Company in Auburn, AL. Aggregates were silicate based river gravel and sand quarried by Beaver Rock Company of Montgomery, AL. Coarse aggregates and fine aggregates were selected according to gradation, and tests were conducted to ensure that these aggregates met gradation specifications. Because of significant variations in the gradations (within the specification limits), the aggregates were blended manually to achieve consistent gradations in the laboratory study. Tests for specific gravities, unit weight, and sieve analysis were also conducted. Table 5.1 summarizes the materials and quantities used in the preliminary mix designs.

5.3 Raw Materials Testing

Aggregates - Coarse aggregate obtained from Blue Circle was ALDOT#57 river gravel. The aggregate gradation was somewhat in error, therefore, the aggregate was separated according to sieve sizes required by ALDOT specifications. Coarse aggregates were measured for moisture content or, in most cases, oven dried. After obtaining moisture content, the original sieving procedure to determine if gradation was correct and consequent sieving procedures to separate sieved particle sizes were conducted according to ASTM C136-84a and ASTM C117-84. The coarse aggregate for each mix was a proper combination of sieved particle sizes for ALDOT #57 coarse aggregate with a nominal maximum aggregate size of ¾".

Unit weight of the properly graded aggregate was determined using steps defined in ASTM C29-89. Specific gravities and absorption for the coarse aggregate were calculated according to ASTM C127-88. The values obtained from these tests were used in the mix design procedure for preliminary and final mixes.

Fine aggregate obtained from Blue Circle was ALDOT #100 concrete sand. The fine aggregate was found to be lacking sufficient fines with some error in gradation. As with the coarse

Table 5.1 Summary of Materials and Quantities Used in Preliminary Mixes

Mix Number	Cement Type	Coarse Agg.[1] (lbs/cy)	Fine Agg.[2] (lbs/cy)	w/c Ratio	Water Reducer[3] (ml)	Air Entrainer[4] (ml)
OOLL20	Type I	1777	1243	0.40	400	80
OOLL40	Type I	1777	1243	0.40	800	80
OOLL60	Type I	1777	1243	0.40	1200	80
OOLL100	Type I	1777	1243	0.40	2000	80
OOLHL	Type I	1777	1171	0.40	1200	200
OOLHH	Type I	1777	1171	0.40	1600	200
OMLLL	Type I w/MS	1777	1243	0.40	1600	120
OMLLH	Type I w/MS	1777	1243	0.40	2800	120
OMLHL	Type I w/MS	1777	1171	0.40	2800	200
OMLHH	Type I w/MS	1777	1171	0.40	4000	200
OOHLL	Type I	1777	1301	0.44	600	120
OOHLH	Type I	1777	1301	0.44	1200	120
OOHHL	Type I	1777	1229	0.44	600	200
OOHHH	Type I	1777	1229	0.44	1200	200
OMHLL	Type I w/MS	1777	1301	0.44	600	120
OMHLH	Type I w/MS	1777	1301	0.44	3000	120
OMHHL	Type I w/MS	1777	1229	0.44	2400	200
OMHHH	Type I w/MS	1777	1229	0.44	3000	200
KKLLL	Type K	1777	1243	0.40	2800	160
KKLLH	Type K	1777	1243	0.40	3200	160
KKLHL	Type K	1777	1171	0.40	2800	240
KKLHH	Type K	1777	1171	0.40	3400	240
KMLLL	Type K w/MS	1777	1243	0.40	3600	120
KMLLH	Type K w/MS	1777	1243	0.40	4200	120
KMLHL	Type K w/MS	1777	1171	0.40	3600	200
KMLHH	Type K w/MS	1777	1171	0.40	4200	200
KKHLL	Type K	1777	1301	0.44	1600	120
KKHLH	Type K	1777	1301	0.44	2400	120
KKHHL	Type K	1777	1229	0.44	1600	200
KKHHH	Type K	1777	1229	0.44	2000	200
KMHLL	Type K w/MS	1777	1301	0.44	3200	160
KMHLH	Type K w/MS	1777	1301	0.44	3600	160
KMHHL	Type K w/MS	1777	1229	0.44	2400	240
KMHHH	Type K w/MS	1777	1229	0.44	3000	240

[1]Coarse aggregate was ALDOT #57 River Gravel.
[2]Fine aggregate was ALDOT #100 Concrete Sand.
[3]Water reducing agent was Rheobuild 1000.
[4]Air entraining agent was Microair.

aggregate, the fine aggregate was oven dried and sieved according to ASTM C136-84a. The fine aggregate for each mix was a proper combination of sieved particle sizes. The fine aggregate was graded as ALDOT aggregate #100 concrete sand with a fineness modulus of 2.85.

Specific gravities and absorption for the fine aggregate were obtained using steps defined in ASTM C 128-88. Fineness modulus was determined as specified in ASTM C136-84a. The values obtained from these tests were implemented into the mix design procedure.

Due to a lack of fine material in both coarse and fine aggregates, dolcito was used to provide a definite amount of fine material in each aggregate batch. The dolcito was magnesian limestone quarried by Vulcan Materials, Tarrant, AL. The dolcito was 100% passing #200 sieve size.

Cements - The two cement types (Type I and Type K) and four cementitious material combinations were also subjected to testing. To compare expansive properties of the four different cementitious material combinations, restrained expansion mortar bars were tested for length change. Further description of this test is given in Section 5.7. Time of set tests were performed to determine how fast mortar consistency changed. See Section 5.7 for more details on time of set testing.

5.4 Batch Preparation Procedure

Proportions for the mix in this study were determined using the procedure described in ACI 211.1-91. In this specification, batch proportions are based upon mix volume of one cubic yard. Due to the limitation in capacity of lab equipment, mixes in this study were much smaller. Preliminary mixes were based on a volume of 1.35 ft^3 and final mixes were based on a volume of 1.69 ft^3. Mixes were designed to meet standards required for A-1c concrete as set forth in the Master Proportion Table of Section 501 of ALDOT Standard Specifications for Highway Construction 1992. The essence of this table is reproduced in Appendix C for convenience.

The design slump of the mixes was 1" to be altered with water-reducing agent. Maximum aggregate size was set at 3/4". Air content range was based directly on the numbers in the Master Proportion Table. The low target for air content was 4.5% and the high target was 6%. The table sets the range at 4-6%. A water/cement ratio of 0.40 (low) or 0.44 (high) was to be used for design of each mix with 0.44 being the maximum allowable water/cement ratio.

Certain values obtained from aggregate testing were important to the mix design procedure. For coarse aggregate, unit weight was found to be 107.02 lb/ ft^3, and specific gravity was 2.65. Absorption for coarse aggregate was 0.325%. For fine aggregate, absorption was 1.895% and fineness modulus was 2.85.

Microsilica was used in half of the mixes in this study. The amount of microsilica was based directly upon percentage of cement weight. For mixes with microsilica, 10% of the cement weight was substituted with microsilica. Specific gravity of cement was taken as 3.15 for cements with or without microsilica.

To best illustrate mix design procedures for the project an example of the procedure will be helpful. The following steps show the directions and tables utilized in ACI 211 to design each mix according to ALDOT specifications. For the example, the process used to obtain preliminary mix KMLLH will be used.

- From Table 6.3.3 of ACI 211, 280 lb water/yd^3 are required for 1" slump for air-entrained concrete with 3/4" maximum aggregate size.

- Using Table 6.3.6 of ACI 211, a ratio which was labeled as "X ratio" was obtained. The ratio is the volume of coarse aggregate divided by total volume of concrete. A value of 0.615 was obtained from the table by interpolation using a fineness modulus (FM) value of 2.85 and a maximum aggregate size of 3/4".

- The batch size was 1.35 ft^3, and the values obtained from ACI 211 are based on 1 yd^3. Therefore, to obtain batch sizes each weight value from ACI 211 was multiplied by a factor. This factor was $1.35\ ft^3 \div 27\ ft^3/yd^3 = 0.05\ yd^3$.

- Cement content $[w_{water} / (w/c)] = 280\ lb/yd^3 \div 0.40 = 700\ lb/yd^3$.

- Coarse Aggregate content = $(X\ ratio)(\gamma_{ca}) = (0.615\ yd^3) \times (27\ ft^3/yd^3) \times (107.02\ lb/ft^3) = 1777.07\ lb$.

- Air content = 4.5% : $0.045 \times 27\ ft^3/yd^3 = 1.22\ ft^3$.

- Water volume = $(280\ lb/yd^3) \times (yd^3/27\ ft^3) \times (ft^3/62.4\ lb) = 4.49\ ft^3$.

- Cement Volume = w_{cem} x $(1/\gamma_w)$ x SG_{cem} = (700 lb) x (62.4 lb) x (3.15)= 3.56 ft^3.

- Coarse Aggregate Vol. = $w_{ca} \div (SG_{ca}$ x $\gamma_w)$ = (1777.07 lb)/(2.65 x 62.4 lb/ft^3) = 10.75 ft^3.

- Fine Aggregate Vol. = 27 ft^3 - V_{ca} -V_{cem} -V_w -V_{air}. = (27-10.75-3.56-4.49-1.22) ft^3 = 6.99 ft^3.

- Fine Aggregate Weight = V_{fa} x γ_w x FM = (6.99 ft^3) x (62.4 lb/ft^3) x 2.85 = 1243.09 lb.

- All weights multiplied by the reduction factor for batch size of 0.05.

- Microsilica weight for batch = 10% of w_{ca} = 0.10 x 35 lb = 3.50 lb.

- Water weight must be adjusted to account for absorption of fine and coarse aggregates.

    1. Coarse Aggregate: 0.325% x 88.85 lb = 0.29 lb.
    2. Fine Aggregate: 1.895% x 62.15 lb = 1.18 lb
    3. Total water = 14 lb + 0.29 lb + 1.18 lb) = 15.47 lb.

- Final Batch Weights:

    w_{ca} = 88.85 lb.
    w_{fa} = 62.15 lb.
    w_{cem} = 31.50 lb
    w_{ms} = 3.50 lb.
    w_w = 15.47 lb.

Amounts of air entrainer and water reducer were determined from values used in trial mixes which were tested before beginning work on this testing procedure. Values for water reducer and air entrainer for preliminary and final mixes can be found in Table 6.1 and Table 6.5, respectively.

5.5 Preliminary Mix Design Evaluations

Thirty-two preliminary mixes were produced via variations of four variables: cementitious material type; water/cement ratio; air-entraining agent; water-reducing agent. Figures 3.1 and 5.1 depict the preliminary mix design program. Each of the test mixes reflected in Figures 3.1 and 5.1 were tested as indicated in Chapter 3, and later described in this chapter. Final mix designs for each cementitious material combination were selected through evaluation of the preliminary mixes.

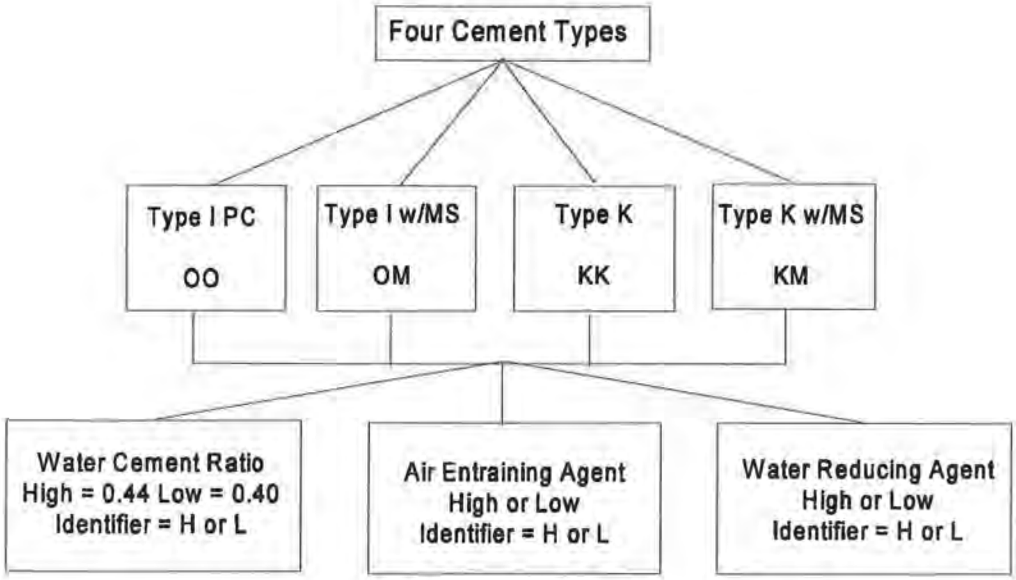


Figure 5.1. Preliminary Mix Design Program

As stated above, initially, thirty-two mixes were planned; however, thirty-four mixes were actually made because at the time the project started there was some uncertainty on effect of the water reducer.

One mix design variable was cement type, and four different cements were used in order to determine what effect each cement had on strength and expansion. The cements used were: Type I Portland cement; Type I with microsilica admixture; Type K expansive cement; Type K cement with microsilica. In mixes with the same water/cement ratio, the amount of cement used for each batch was identical.

A second variable in the preliminary mixing plan was water cement ratio. Water/cement ratio greatly affects the expansive nature of Type K cement. The two different values were studied in order to gauge the effect of water content on expansion as well as strength. A low value was chosen at 0.40 and a high value at 0.44. The maximum water/cement ratio allowed by ALDOT is 0.44.

The remaining two variables in the plan were two admixtures, i.e., an air-entraining agent and a water-reducing agent. An air-entraining agent was necessary to achieve the specified air content in each mix. A water-reducing agent was used in order to achieve a higher slump and, therefore, improved workability of the mix at time of delivery to the job site. A high value and low value for each admixture were chosen in order to see the effect of each admixture and use those results to pick an optimum value for each for the final mix procedure.

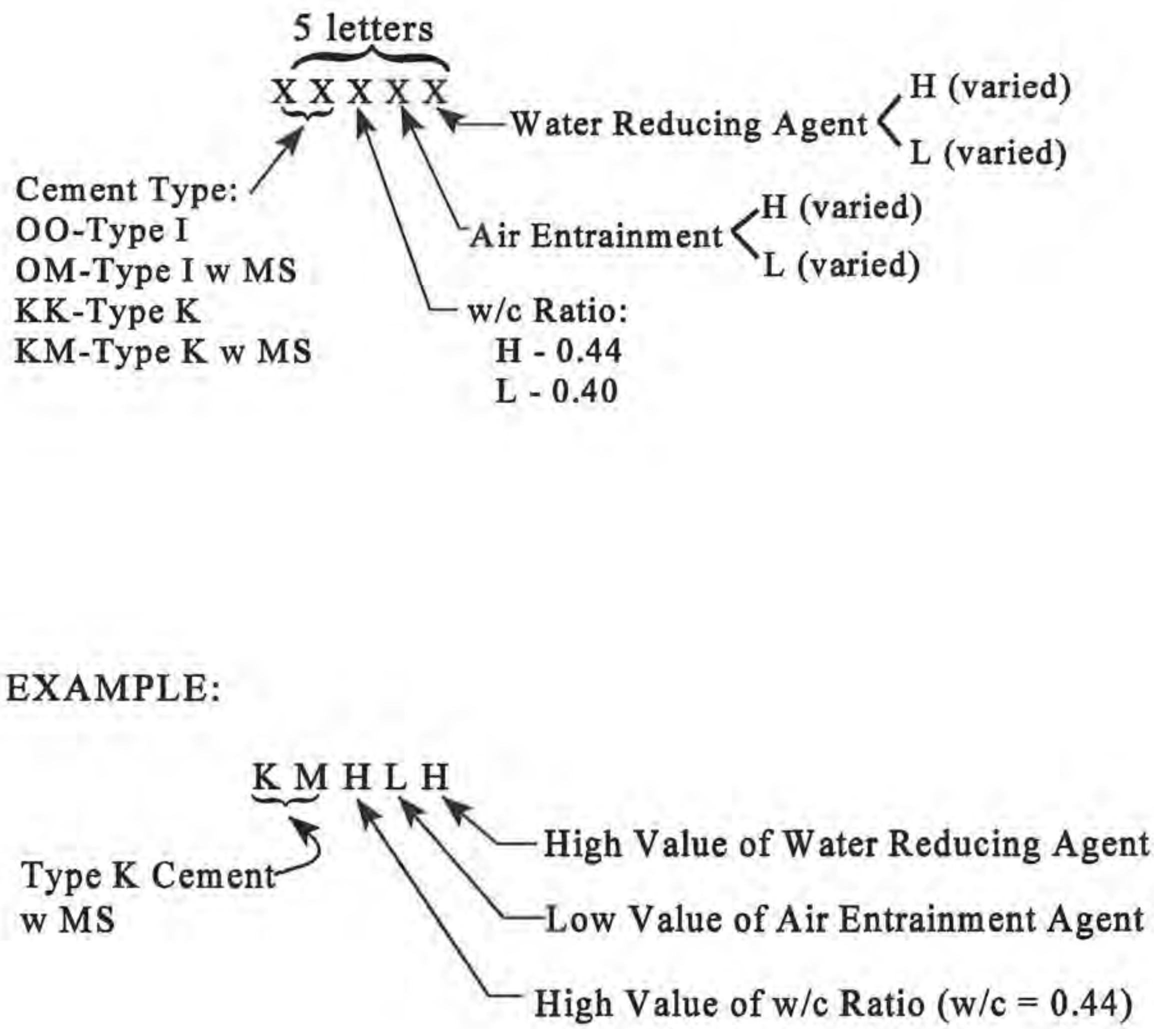
Each preliminary mix was assigned an identifier of five letters which corresponds to a change in the value of each variable. The nomenclature for the identifiers is as follows:

- First two letters represent the cement type: OO, OM, KK, KM
- Third letter represents value for water/cement ratio: H, L (High or Low)
- Fourth letter represents the value for air entraining agent: H,L (High or Low)
- Fifth letter represents the value for water reducing agent: H,L (High or Low).

The identification code is illustrated graphically in Figure 5.1a along with an example application.

Mixing Procedure - Each preliminary and final mix was prepared in the same manner. Twenty-four hours before mixing, the aggregate proportions were measured and part of the water was added to ensure saturated-surface-dry conditions. The procedure of mixing for each concrete batch followed the ASTM C192 - 902 Mixing Procedure which is basically as described in the following steps.

- Add batch ingredients into mixer.
- Start mixer and mix for 3 minutes.
- Stop mixer for 3 minutes.
- Take temperature at end of 3 minutes.
- Start mixer and mix for 2 minutes.
- Stop mixer for 15 minutes.
- At 10 minutes, slump is measured and temperature is recorded.



Note: Specific values of air entrainment and water reducers for each mix are identified in Table 5.1

Figure 5.1a. Identification Code for Preliminary Mix Designs

- Start mixer and mix for 2 minutes.
- Perform slump test(s).
- Record unit weight(s).
- Conduct air content test(s).
- Make cylinders.
- Make restrained expansion length bars (Type K mixes only.)

Testing - The tests and steps shown above were performed on fresh concrete. Other tests were conducted on cured concrete and cement mortar. The following list is a brief summary of the tests used in the preliminary and final mix design evaluations. A detailed description of the tests is included in a later section in this chapter.

- Slump Test - conducted on preliminary and final mixes.
- Air Content - conducted on preliminary and final mixes.
- Unit Weight - conducted on preliminary and final mixes.
- Compressive Strength - conducted with two cylinders from preliminary mixes conducted with four or five cylinders from final mixes.
- Splitting Tensile Strength - conducted with two cylinders from preliminary mixes. conducted with two or three cylinders from final mixes.
- Restrained Expansion of SCC - Two restrained expansion length bars made from each Type K concrete mix.
- Restrained Expansion of Cement Mortar - Two restrained expansion bars produced from each mortar mix of the four cementitious material combinations. Specimen aggregate was Ottawa sand and no admixtures were employed.
- Time of Set - The time of set using a Vicat needle apparatus was performed on each mortar mix for each type of cementitious material combination. Mortar aggregate was Ottawa sand and no admixtures were employed.

5.6 Final Mix Design Evaluations

From an examination of the results of the preliminary mixes, four final mixes were selected.

Each final mix used a different cement and was tested under different temperature and curing conditions as indicated in Figure 5.2.

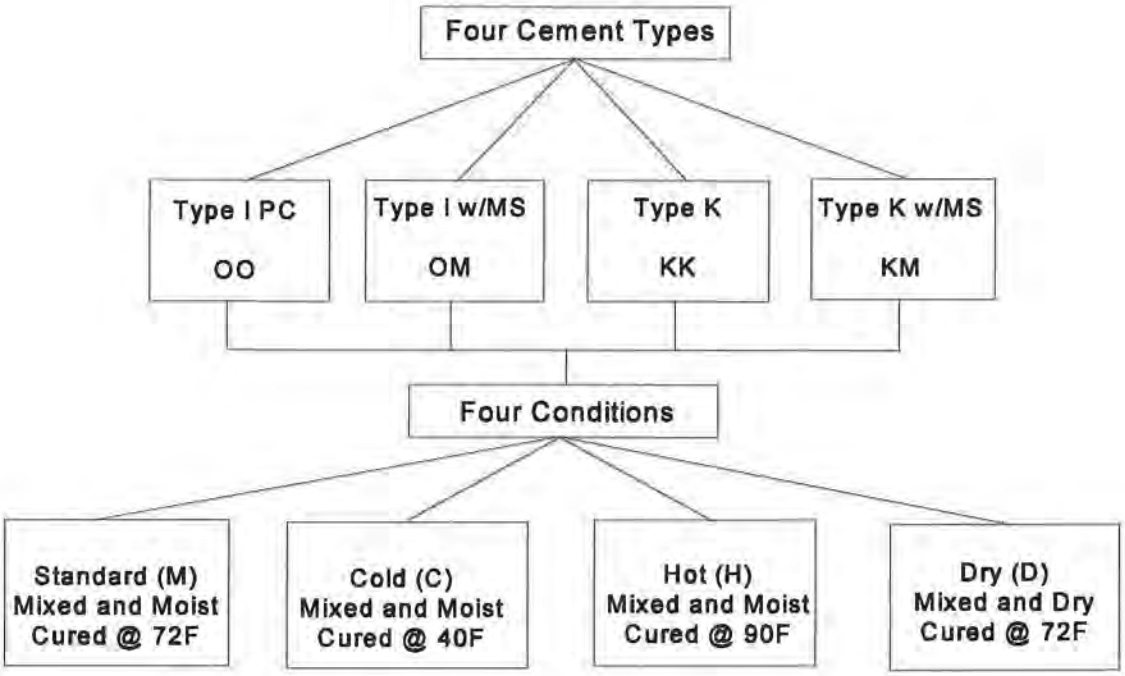


Figure 5.2. Final Mix Design Test Conditions

All four concrete mixes were subjected to the same environmental conditions and tested. There were four different conditions: mixing at 72°F and moist cure at 72°F; mixing at 40°F and moist cure at 40°F; mixing at 90°F and moist cure at 90°F; mixing at 72°F with dry cure at 72°F. The moist cure at 72°F served as a standard curing procedure. The 40°F and 90°F conditions simulated cold and hot curing which might occur in the field. The dry cure case was a look at a worst case scenario for curing.

Each final mix was assigned an identifier of three letters. The nomenclature for final mix identifiers is as follows:

- The first two letters represent the cement type: OO, OM, KK, KM.
- The letter after the dash represents the curing and mixing condition: M, C, H, D.

This identification code is graphically illustrated in Figure 5.2a along with an example application.

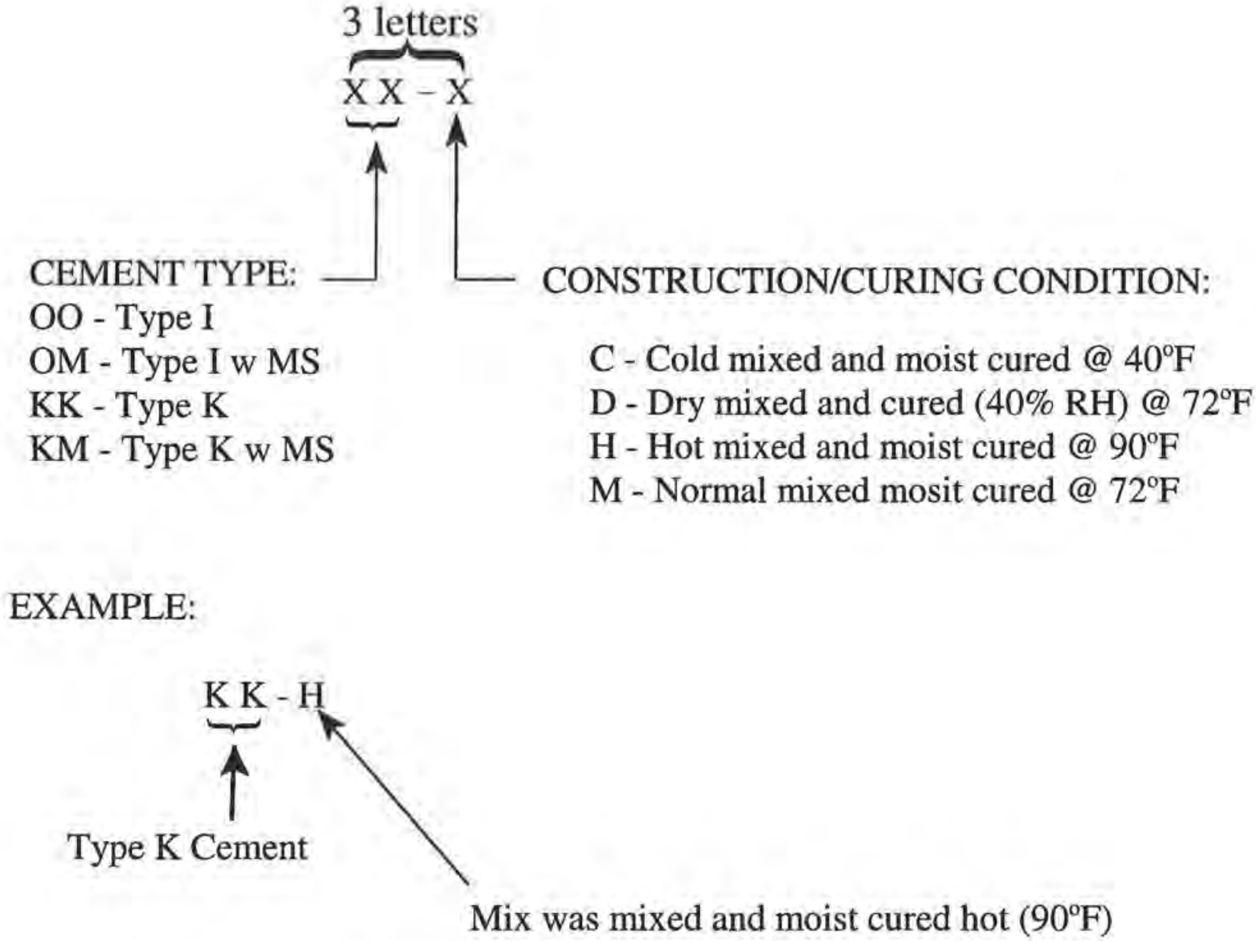


Figure 5.2a. Identification Code for Final Mix Design Test Conditions

Final mixes were prepared in the same manner as preliminary mixes. The same tests were performed on every mix. Restrained expansion mortar bars were made for each cement used and time of set tests were conducted on each mortar type. Results for final mixes can be found in Chapter 6.

5.7 Mix Design Testing

Testing for each of the preliminary and final mix designs was listed in Section 5.5. Descriptions and photographs of each test setup are given below.

Slump Test - The slump test was performed on each mix in order to assess the workability of each mix. The test was performed in accordance with ASTM C143-90. Equipment used for the test was a slump cone, ⅝" tamping rod, scoop, and a measuring tape. ALDOT specs require that the slump not exceed 3.5 inches. Typical equipment for the slump test is shown in Figure 5.3.

Unit Weight - Unit weight tests were conducted on each mix to determine weight of concrete mix per cubic foot. The test was conducted according to ASTM C138-81. Equipment utilized in the test was a balance, tamping rod, bucket from pressure air meter, flat strike rod, ⅝" tamping rod, scoop, and a mallet to provide vibration to close internal voids. The unit weight was calculated by dividing the weight of concrete and the bowl minus the weight of the bowl by the volume of the bowl which was 0.2496 ft^3. Figure 5.4 is a picture of equipment used in the unit weight test.

Air Content - Air content tests were performed on each mix to determine the volumetric percentage of entrained air in the freshly mixed concrete. The majority of the air content tests in this study including all of the final mixes were performed using the pressure method outline in ASTM C231-89a. Equipment for this test included a Hogentogler Press-UR-Meter, Type B air meter, ⅝" tamping rod, flat strike rod, scoop, and a mallet. The remaining air tests were conducted using the gravimetric method as outlined in ASTM 138-81. Equipment for this test was the same as the pressure method except for the air meter which was a Roll-A-Meter by Concrete Specialties Co. The Type B air meter used along with other equipment used in the air content test is displayed in Figure 5.5.

Compressive Strength Testing - From every mix in the study, 6" diameter x 12" tall cylinders were made for compression testing. The cylinders were constructed according to ASTM C192-90. Equipment used for making cylinders included cylinder molds, ⅝" tamping rod, flat strike rod, scoop, and mallet. Typical equipment for making cylinders is shown in Figure 5.6. Cylinders remained in the molds covered with plastic bags for twenty-four hours. At twenty-four hours, the cylinders were stripped and moist cured for twenty-eight days except for the dry cured mixes. At twenty-eight days the cylinders were tested for compressive strength according to ASTM C39-86.

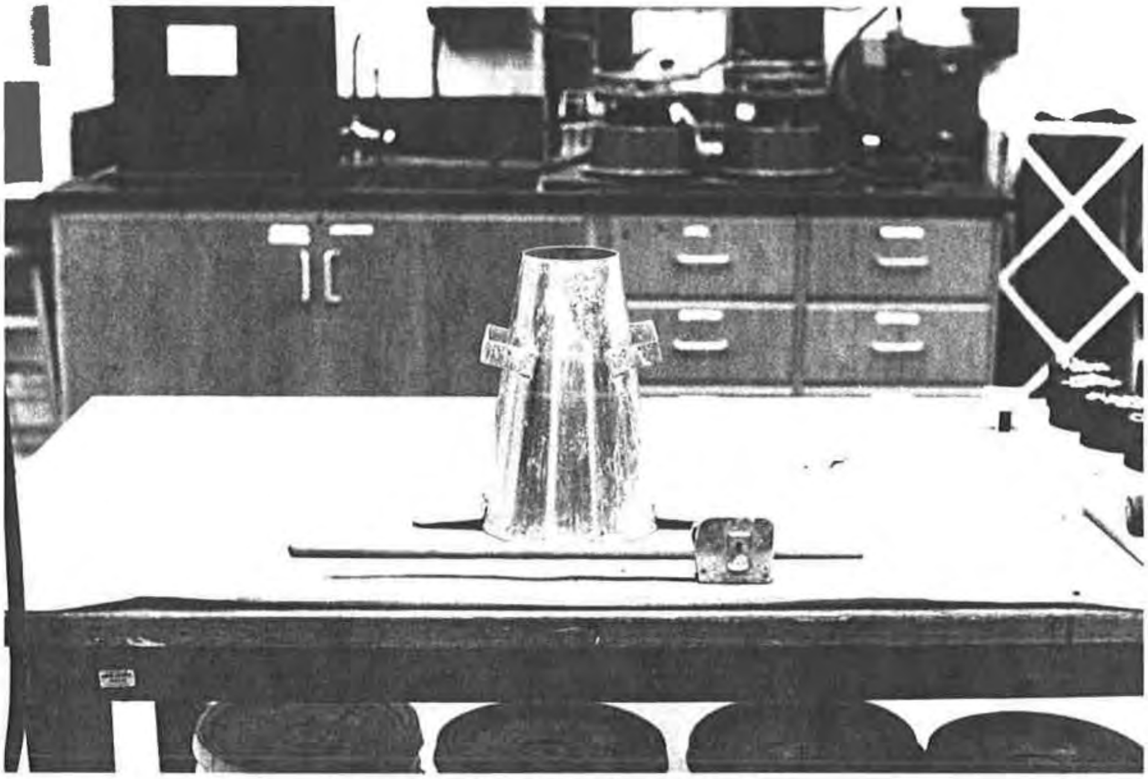


Figure 5.3. Slump Test Equipment

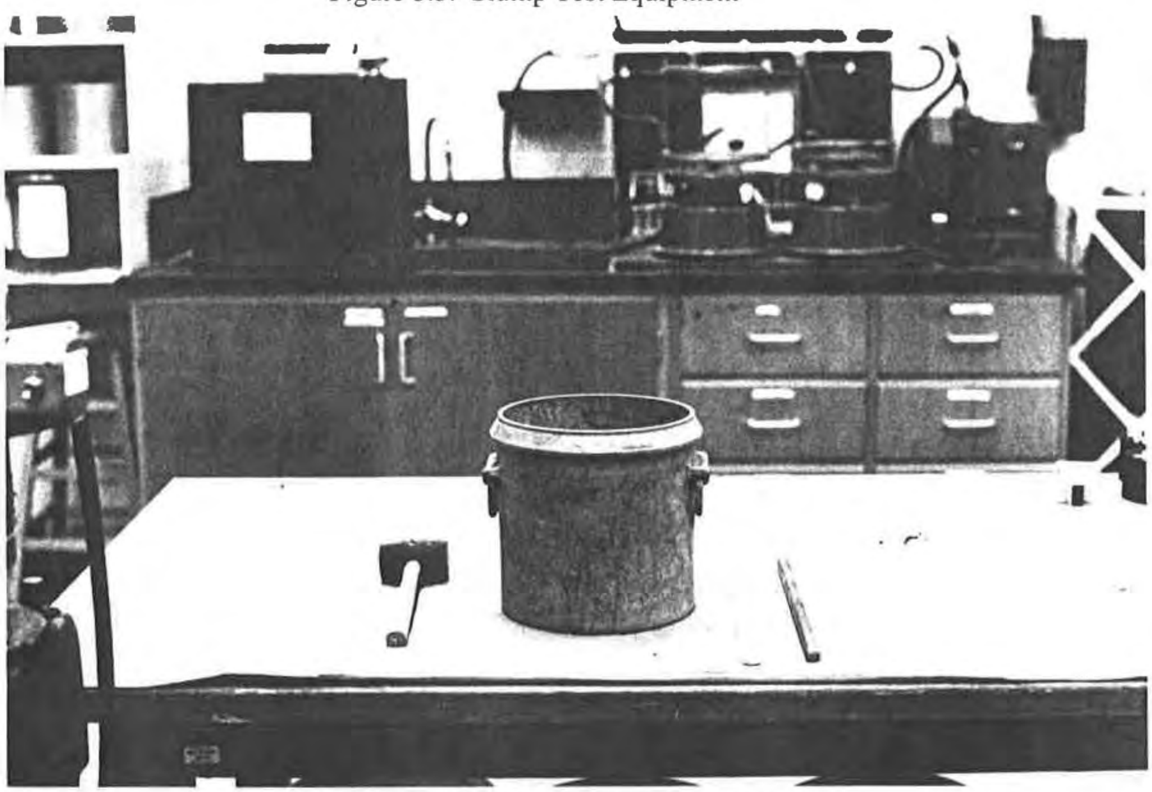


Figure 5.4. Unit Weigh Test Equipment

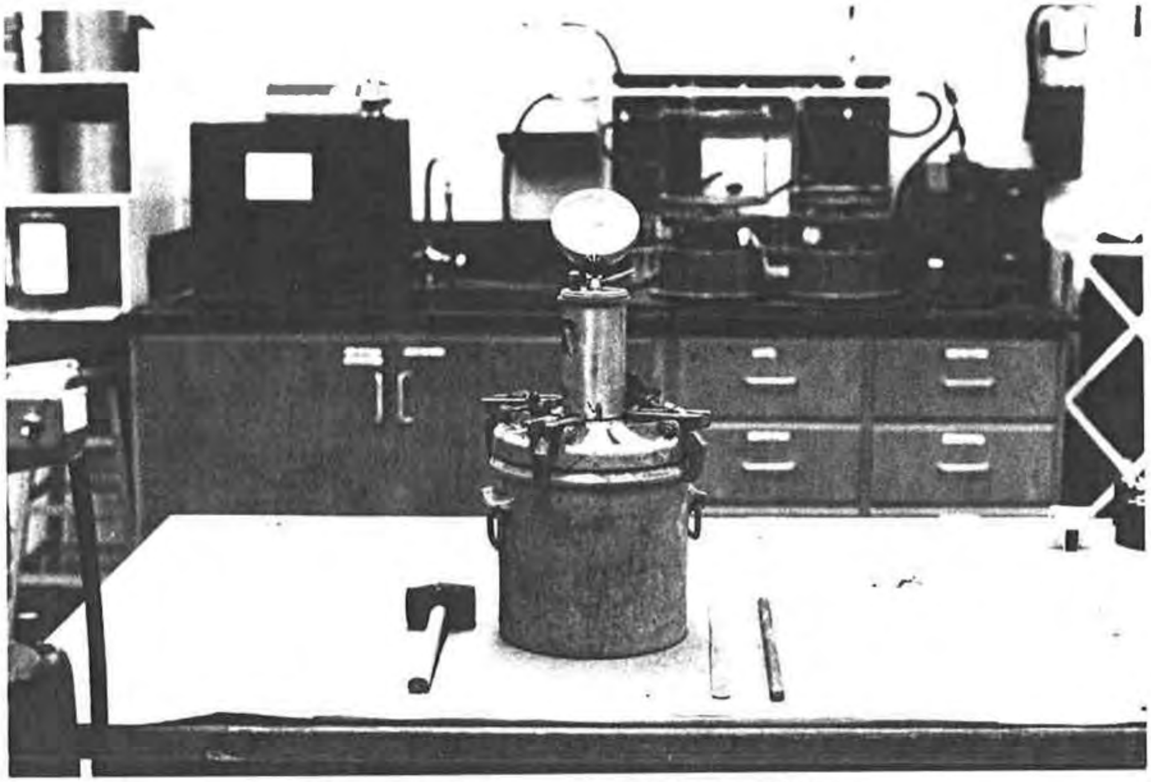


Figure 5.5. Equipment for Air Content Test

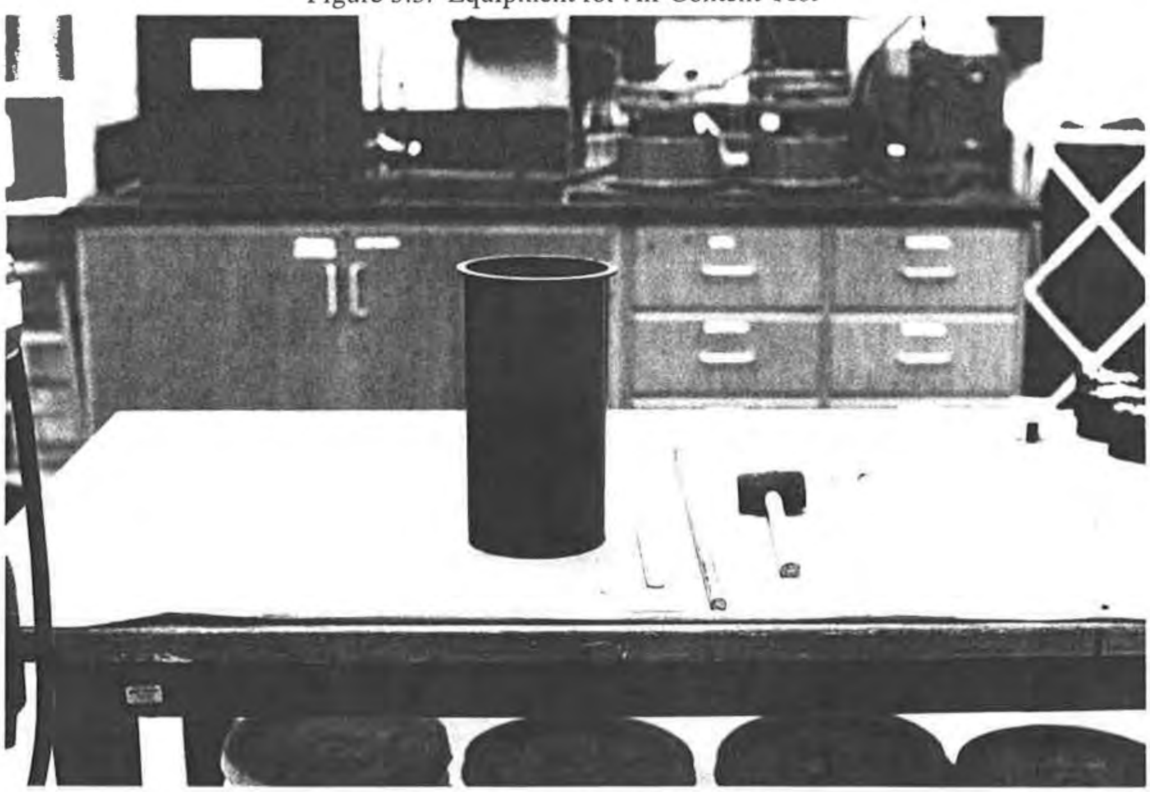


Figure 5.6. Equipment for Making Cylinders

The cylinders were loaded to failure using a Forney Model QC-400-02 with 400,000 pound capacity. The compressive strength was determined using $CS = F/\pi r^2$, where CS is compressive strength, F is load, and the denominator is area of the cylinder end. Values are recorded in the results section to the nearest 20 psi which was the precision of the loading machine. Figure 5.7 is a picture of a cylinder undergoing a compressive strength test.

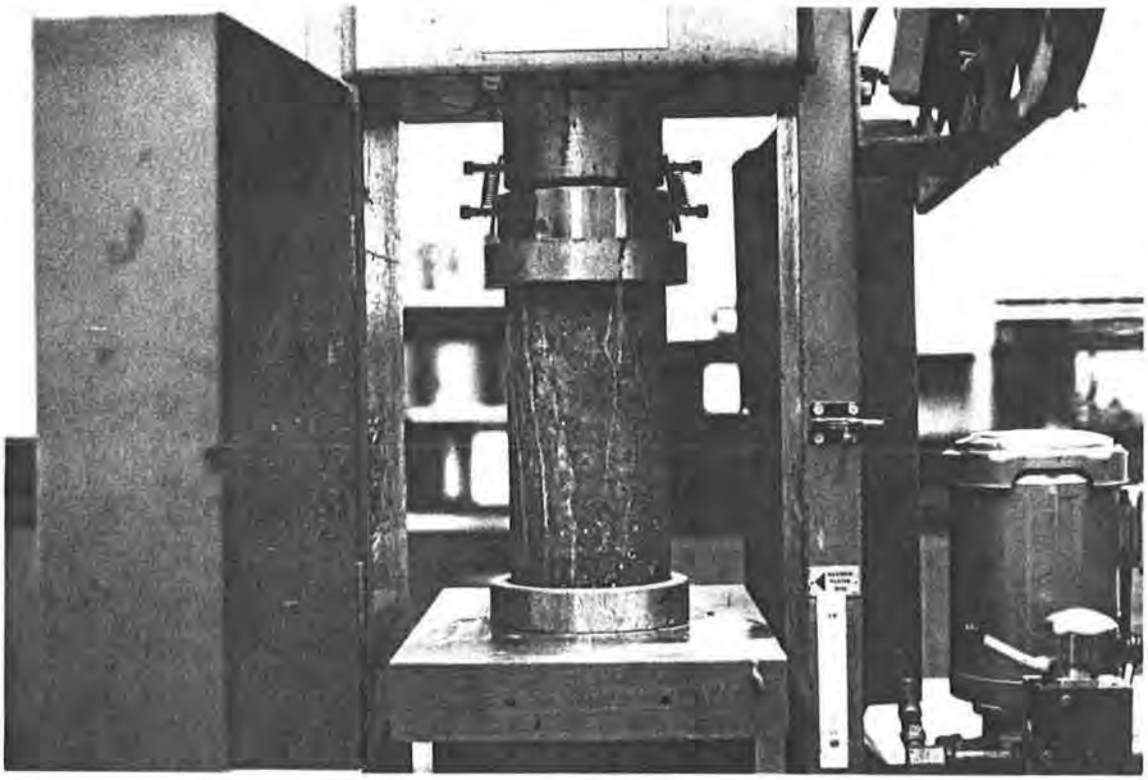


Figure 5.7. Cylinder Subjected to Compressive Load

Splitting Tensile Strength Testing - Cylinders from each mix were also made for splitting tensile strength testing. The tensile strength of concrete is an important value when studying shrinkage cracking because shrinkage cracks induce tensile stress in the concrete. The cylinders for splitting tensile testing were constructed in the same manner as compression cylinders and utilized the same equipment. The cylinders were allowed to cure for twenty-eight days before testing. The actual loading to failure was conducted using the procedure in ASTM C496-86. The same Forney Model QC-400-02 used for compressive strength tests was used in the splitting tensile test. Tensile strength was calculated using the formula, $T = 2F/(\pi ld)$, where F is load, l is the length of the cylinder, and r is the radius of the cylinder. The precision of the machine for splitting tensile strength was 10 psi. A splitting tensile specimen in loading is shown in Figure 5.8.

Restrained Expansion of Concrete - For each Type K cement concrete mix, two specimens for measurement of restrained expansion were cast. Restraining cages were constructed of 3" x 3" stainless steel plates and 3/16" steel threaded rod as specified in ASTM C878-87. Concrete was placed in stainless molds in order to adhere to the cages according to ASTM C878-87. Figure 5.9 is a picture of typical equipment used for this test. Molds were coated with a thin film of spray lubricant (WD-40) to ensure easy separation from the concrete specimens. Each length bar remained in the molds for six hours covered with plastic. At six hours, the length bars were stripped from the molds and placed in lime-saturated water for twenty-eight days. For final mixes, the bars were cured at the environmental condition of the mix. Length measurement for each bar was made every day including initial length of the cage before placement of the concrete. A reference bar of invar was also measured as a standard and an adjacent change for each length bar was recorded. Equipment for the test included 3" x 3" molds, a small tamping rod, mallet, and a length measuring device. The length comparator used was an H3250 by Humbolt Manufacturing Company. The length comparator is displayed in Figure 5.10. This test is a simulation of concrete in the field and can provide a predictable expansion for concrete made from expansive cements.

Restrained Expansion of Cement Mortar - This test was conducted in order to determine the magnitude of restrained expansion for each mortar. The test was performed in accordance with ASTM C806-87. Equipment used for the test included 2" x 2" molds, tamping rod, mallet, Humbolt H3250 length comparator, and a mixer. The bars were constructed of cages much like the length bars except that the end plates were 2" x 2" and the threaded rod was 1/4". The mortar was composed of the cement being tested and a prescribed amount of water and Ottawa sand. Type I Portland cement, Type I with microsilica, Type K expansive cement, and Type K with microsilica

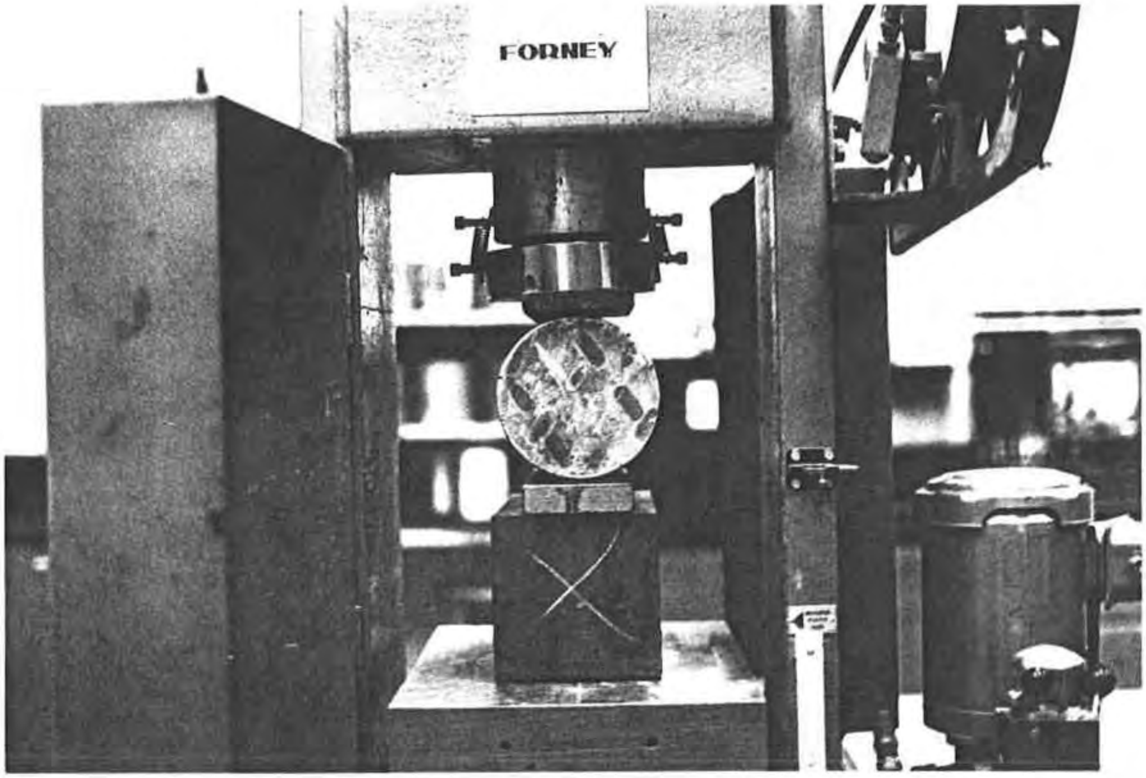


Figure 5.8. Cylinder Subjected to Splitting Tensile Test

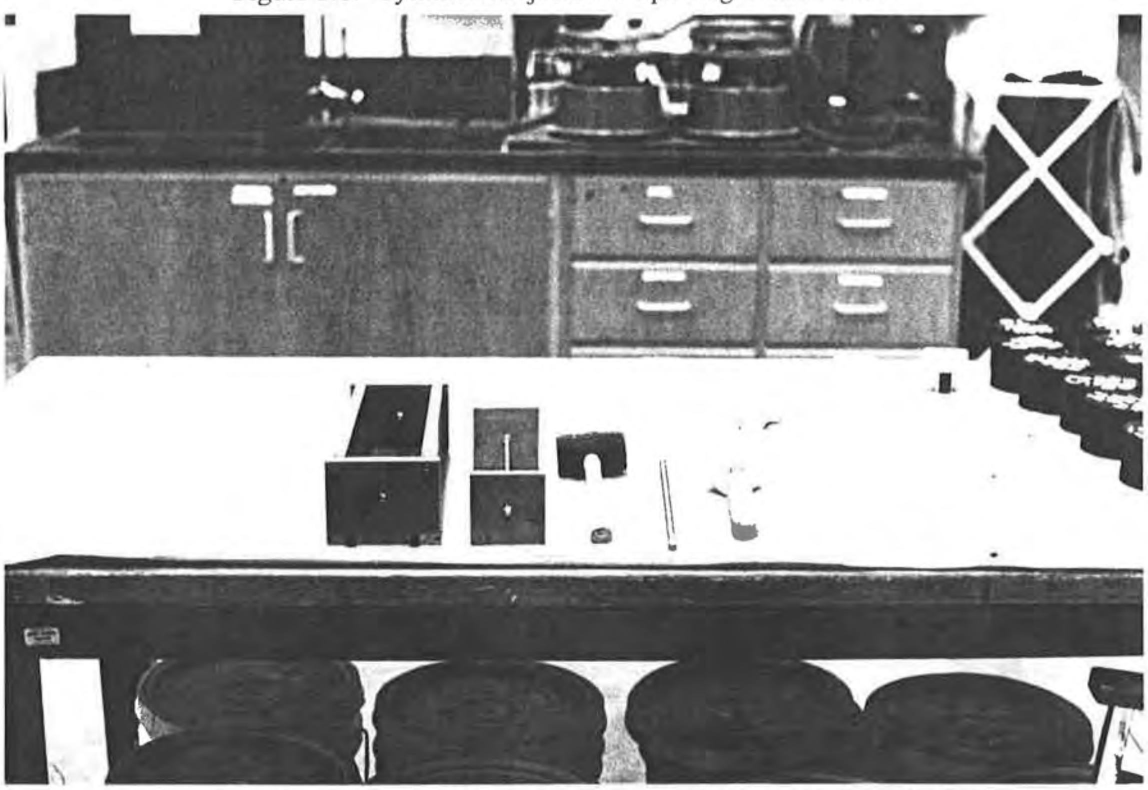


Figure 5.9. Equipment for Producing Restrained Expansion Length Bars

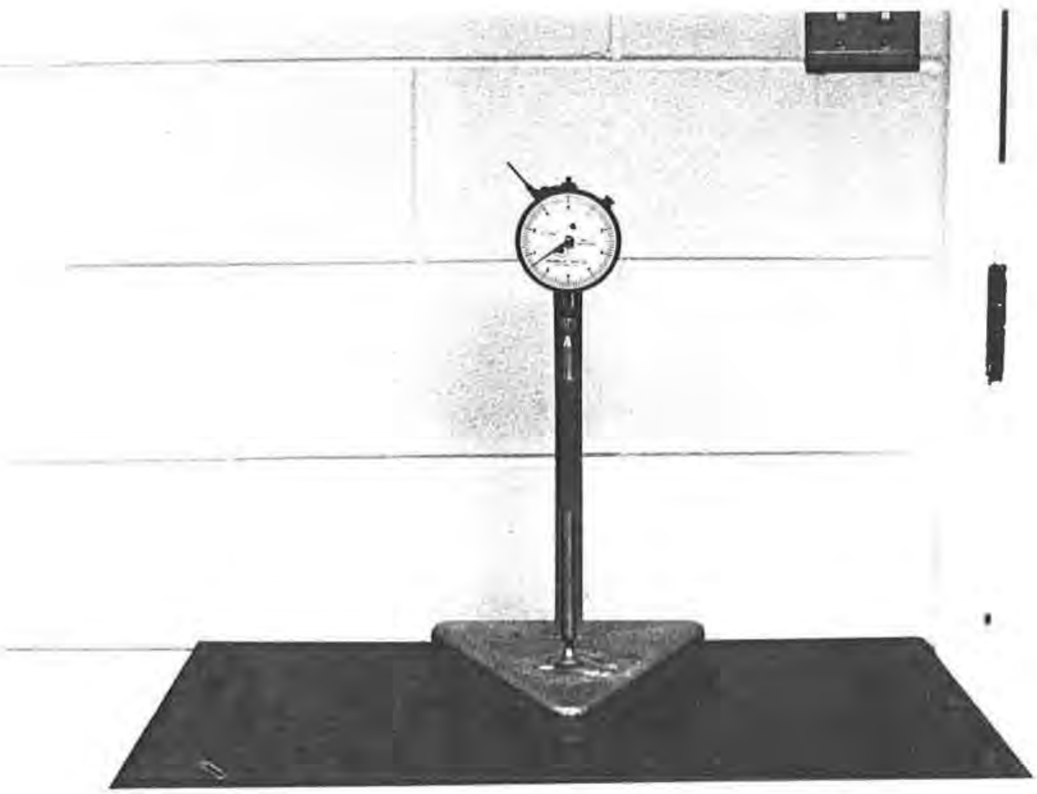


Figure 5.10. Length Comparator Used in Measurement of Restrained Expansion Length Bars and Mortar Bars

were subjected to mortar bar testing. The bars were cured in lime-saturated water at 72°F for twenty-eight days and measured every day. Molds and equipment used in the test are shown in Figure 5.11.

Time of Set - This test is a method of determining time of set of hydraulic cement mortar by means of a modern Vicat needle. Each of the four cement types was mixed with water and Ottawa sand to a desired consistency. The time required for a Vicat needle to penetrate 10mm was recorded. These steps followed in the time of set test are outlined in ASTM C 807-89. Equipment used for the test included the Vicat needle plunger apparatus, brass molds, mixer, balance, specified rubber tamping rod, trowel and/or spatula, and glass graduates. Equipment for the time of set test is shown in Figure 5.12.

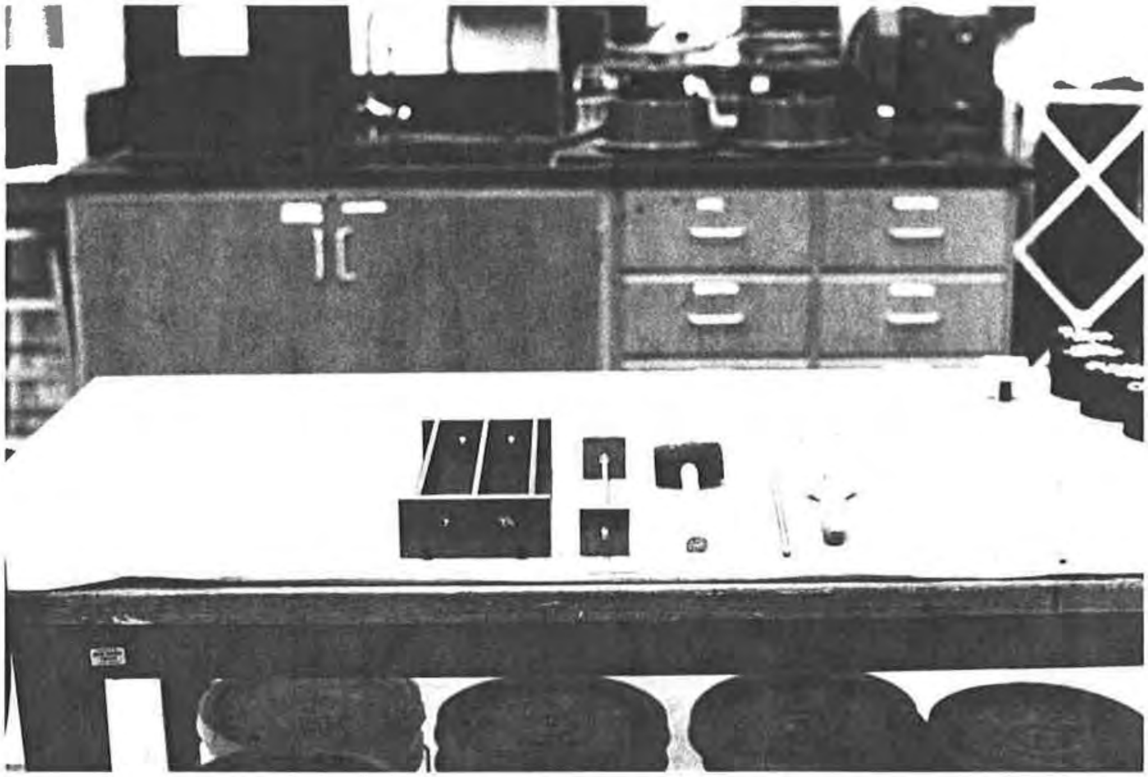


Figure 5.11. Equipment for Producing Restrained Expansion Mortar Bars

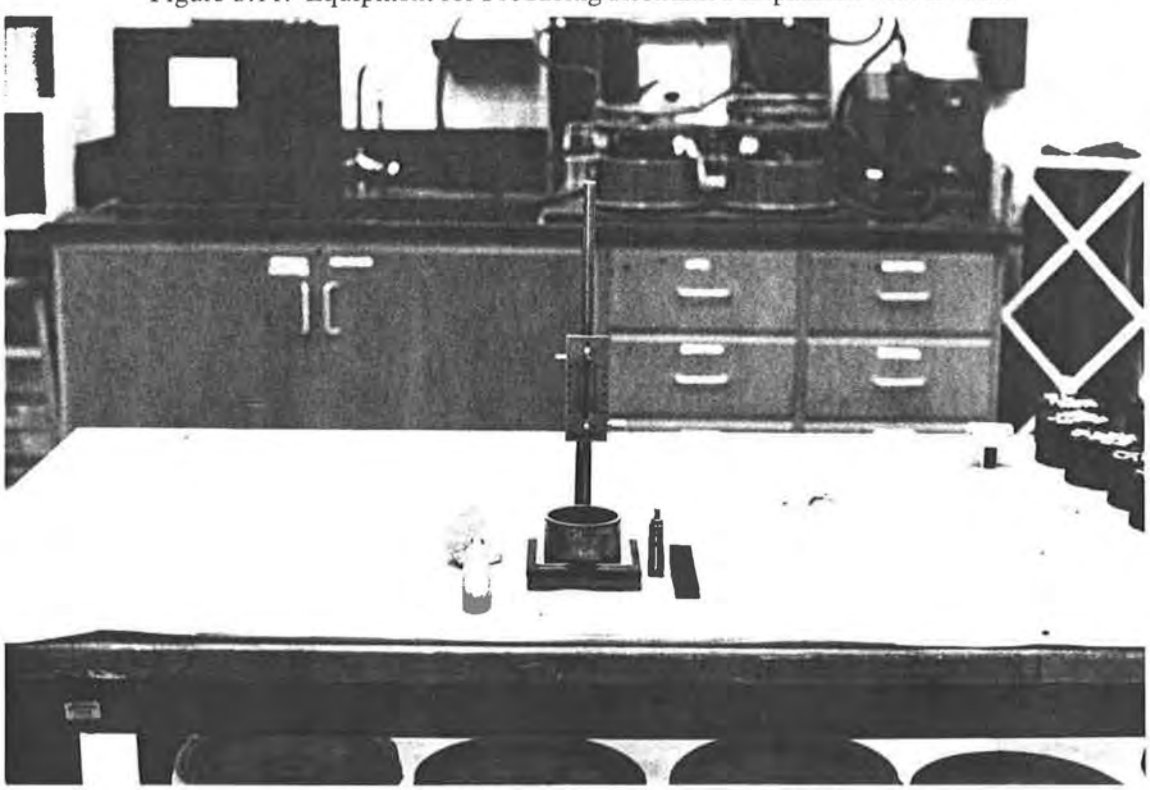


Figure 5.12. Equipment for Time of Set Test

6. PRESENTATION OF LABORATORY RESULTS

6.1 General

Laboratory testing was initially performed on preliminary concrete mix designs of

- Type I Portland Cement (designated 00)
- Type I Portland Cement with Microsilica (designated OM)
- Type K Cement (designated KK)
- Type K Cement with Microsilica (designated KM)

to identify a good final mix design for each of these four (4) mix types. The data/results from this testing are presented in Sections 6.2 and 6.3.

Upon completion of the preliminary mix design testing, a final mix design for each of the four (4) cementitious material types above was identified for further verification, property evaluation, and sensitivity testing. The data/results of these final mix design testings are presented in Sections 6.4 - 6.7.

6.2 Preliminary Mix Data/Results

After completion of the preliminary mixing plan, thirty-four mixes had been produced. Information from each test was recorded and is presented in the figures and tables of this section. Information and data about restrained expansion length bars for preliminary mixes can be found in Section 6.3. A summary of data for preliminary mixes can be found in Table 6.1. The table lists each preliminary mix number and its values for water/cement ratio, rounded values for compressive and splitting tensile strengths, water-reducing agent, air-entraining agent, slumps, air content, unit weight, and 7-day expansion of Type K length bars.

In the preliminary mixing plan water-reducing agent and air-entraining agent were two of the variables studied. Figures 6.la and 6.lb are graphical representations of the amount of water reducer used in each preliminary mix. Because of the trial-and-error nature of the preliminary mixes, water reducer values varied from mix to mix. In an attempt to meet the ALDOT slump requirements, more water reducer was needed for the Type K mixes. The quantity of air entraining agent for each mix can be seen in Figures 6.2a and 6.2b. Air content measured for each mix is shown in Figures 6.3a and 6.3b. Of the thirty-four preliminary mixes, seventeen mixes did not achieve required air content and four other mixes had an air content of less than 4.5%. For values of high water/cement ratio, air content appeared to increase with an increase in water-reducing agent. Increase of air entraining agent had a greater effect on Type I mixes than Type K mixes. Increase of water/cement ratio in general caused a general increase in air content, especially in Type K mixes.

Slump is an important parameter for accessing the workability of a concrete mix. Slump was recorded for each preliminary mix and is shown in Figures 6.4a and 6.4b. Figure 6.4a displays values for slump recorded for preliminary mixes with a water/cement ratio of 0.40. Slumps for mixes with a water/cement ratio of 0.44 are displayed in Figure 6.4b. Though there was much scatter in the data, Figures 6.4a and 6.4b show greater slump losses with the Type K mixes as expected. Also, data for preliminary mixes shows a general trend of increasing slump loss with increase in water-reducing agent.

Table 6.2 contains a list of the preliminary mixes and the corresponding compressive and splitting tensile strengths for each mix. Two cylinders from each mix were tested for compressive strength and two cylinders were tested for splitting tensile strength. The values of compressive strength were obtained by averaging the values from the two cylinders, and the final value was rounded to the nearest 10 psi. Compressive strengths for mixes with a water/cement ratio of 0.40 can be observed in Figure 6.5a. Figure 6.5b shows compressive strengths for mixes with the high water/cement ratio. As expected, compressive strength decreased with an increase in water/cement ratio. Likewise, an increase in water-reducing agent decreased compressive strength. Microsilica admixture yielded an increase in compressive strength in both Type K and Type I mixes. All of the preliminary mixes were in excess of the required bridge deck strength of 4000 psi. Compressive strength for Type K mixes was significantly higher than Type I mixes.

The final value for splitting tensile strength was calculated by averaging the value of the two cylinders tested for splitting tensile strength, and the value was rounded to the nearest 5 psi. Splitting tensile strength for mixes with low value of water/cement ratio are shown in Figure 6.6a. Values of splitting tensile strength for mixes with the high value of water/cement ratio are represented in Figure 6.6b. Type K mixes were found to have a significantly higher splitting tensile strength than Type I mixes. As with compressive strength, an increase in water/cement ratio led to a decrease in splitting tensile strength. This increase in water/cement ratio did not affect the splitting tensile strength of the Type K mixes as much as it did the tensile strength of the Type I mixes. The value of the splitting tensile strength was in the vicinity of one-twelfth the compressive strength.

Table 6.1 Preliminary Mix Data

Mix Number	w/c Ratio	WR (ml)	Air AEA (ml)	Air (%)	Air Time[1] (min)	Slump 20 Min. (in)	Slump Final (in)	Slump Time[1] (min)	Slump Loss (in)	Unit Wt (pcf)	Expan. (%)	Comp[2] Str. (psi)	Tens[3] Str. (psi)
OOLL20	0.40	20						32				5010	455
OOLL40	0.40	40						28				5120	430
OOLL60	0.40	60						36				4820	395
OOLL100	0.40	100	4			6 1/4	3 1/2	38	2 3/4	146.5		4730	365
OOLHL	0.40	60	10	5	36	2 1/2	2 1/4	34	1/4	145.1		4440	380
OOLHH	0.40	80	10	4.9	35	4 1/2	(not measured)			144.7		4290	410
OMLLL	0.40	80	6	3.8	62	1/2	1/4	34	1/4	148.0		5750	420
OMLLH	0.40	140	6	2.8	63	2	1	31	1	148.1		5670	400
OMLHL	0.40	140	10	3.4	54	2 1/4	1 1/2	34	3/4	146.2		5140	355
OMLHH	0.40	200	10	3.6	50	5	3	34	2	146.0		5410	380
KKLLL	0.40	140	8	3	40	1	1/2	40	1/2	149.1	0.021	7130	570
KKLLH	0.40	160	8	4.2		6	2 1/4	37	3 3/4	146.8	0.029	6870	545
KKLHL	0.40	140	12	3.2	37	4 1/4	2 1/2	30	1 3/4	149.2	0.048	7300	510
KKLHH	0.40	170	12	6.5	39	9 1/2	4 3/4	50	4 3/4	142.4	0.030	6250	515
KMLLL	0.40	180	6	1.9	32	1 1/2	3/4	27	3/4	148.5	0.030	7420	535
KMLLH	0.40	210	6		34	3 1/2	2 1/4	25	1 1/4	148.2	0.039	7310	545
KMLHL	0.40	180	10	1.5	44	1	3/4	34	1/4	140.9	0.034	6990	525
KMLHH	0.40	210	10	2.2	50	2	3/4	35	1 1/4	147.2	0.024	6760	535
OOHLL	0.44	30	6	2.6	50	1 3/4	1 1/2	41	1/4	147.3		4530	420
OOHLH	0.44	60	6	3.1	46	2 1/2	2	39	1/2	146.8		4700	445
OOHHL	0.44	30	10	3.7	45	1 1/2	1 1/2	41	0	146.7		4420	375
OOHHH	0.44	60	10	6.1	47	4 1/4	3	41	1 1/4	144.4		4000	330
OMHLL	0.44	30	6	3.6	58	1 1/4	1 1/4	44	0	147.3		4950	400
OMHLH	0.44	150	6	3	52	4 1/2	2 1/4	39	2 1/4	145.8		4740	350
OMHHL	0.44	120	10	6	57	4 1/2	2 1/2	41	2	142.4		4240	315
OMHHH	0.44	150	10	5.6	58	6 3/4	3 1/2	44	3 1/4	141.9		4330	360
KKHLL	0.44	80	6	3.7	36	3 1/2	1 1/2	40	2	144.5	0.056	6450	535
KKHLH	0.44	120	6	5.6	33	8	5 1/2	28	2 1/2	144.3	0.051	5890	500
KKHHL	0.44	80	10	4	35	2 1/2	3/4	41	1 3/4	147.3	0.029	6000	555
KKHHH	0.44	100	10	4.6	37	4 3/4	1 1/2	35	3 1/4	145.8	0.035	5910	495
KMHLL	0.44	160	8	3.4		1 3/4	1/2		1 1/4	146.8	0.021	6950	500
KMHLH	0.44	180	8	4.5	37	5 1/4	2 3/4	39	2 1/2	144.7	0.018	6650	550
KMHHL	0.44	120	12	3.6	33	1	0[4]		1	146.7	0.043	6630	520
KMHHH	0.44	150	12	4.1	32	3 1/4	1 1/4	34	2	146.1	0.038	6420	540

1 Elapsed Time from initiation of mixing
2 Rounded to nearest 10 psi
3 Rounded to nearest 5 psi
4 This value was not measured. It estimated to be zero.

Cement Type: XXXXX
OO-Type I
OM-Type I with MS
KK-Type K
KM-Type K with MS

Water Reducing Agent, High (H) or Low (L)
Air Entraining Agent, High (H) or Low (L)
Water/Cement Ratio: 0.44 (H) or 0.40 (L)

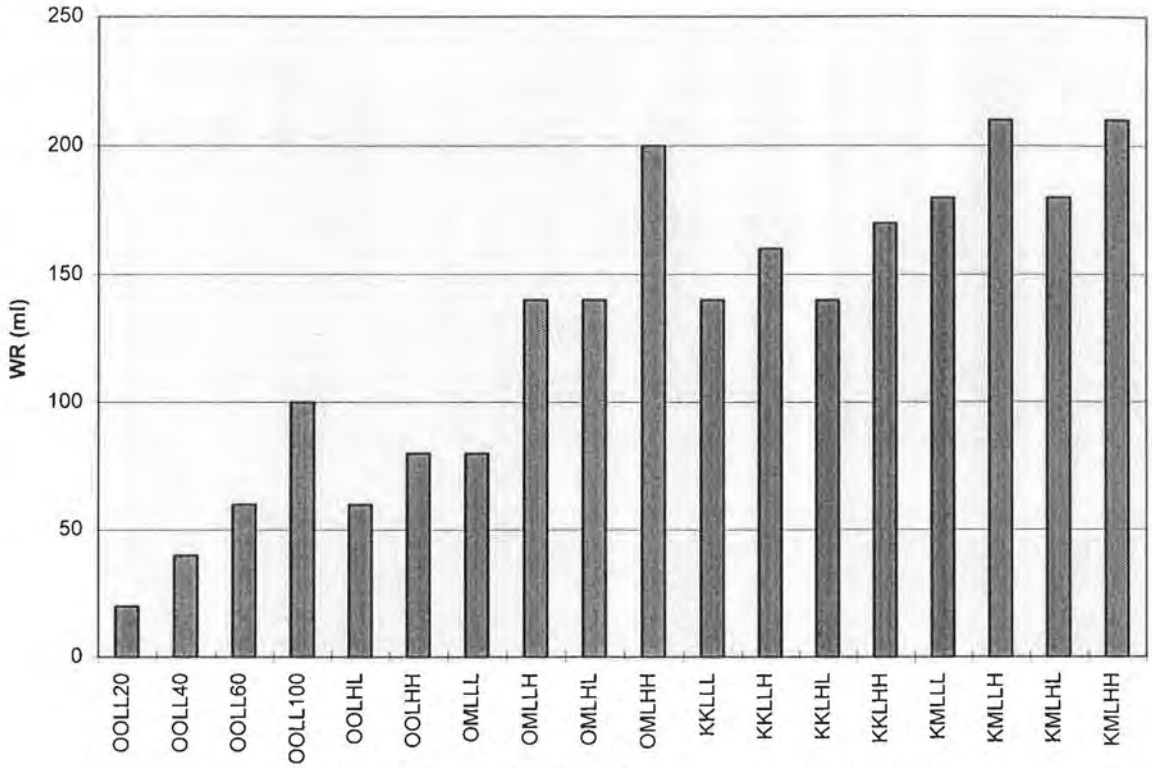


Fig 6.1a Preliminary Mix Water Reducing Agents (ml) [w/c=0.40]

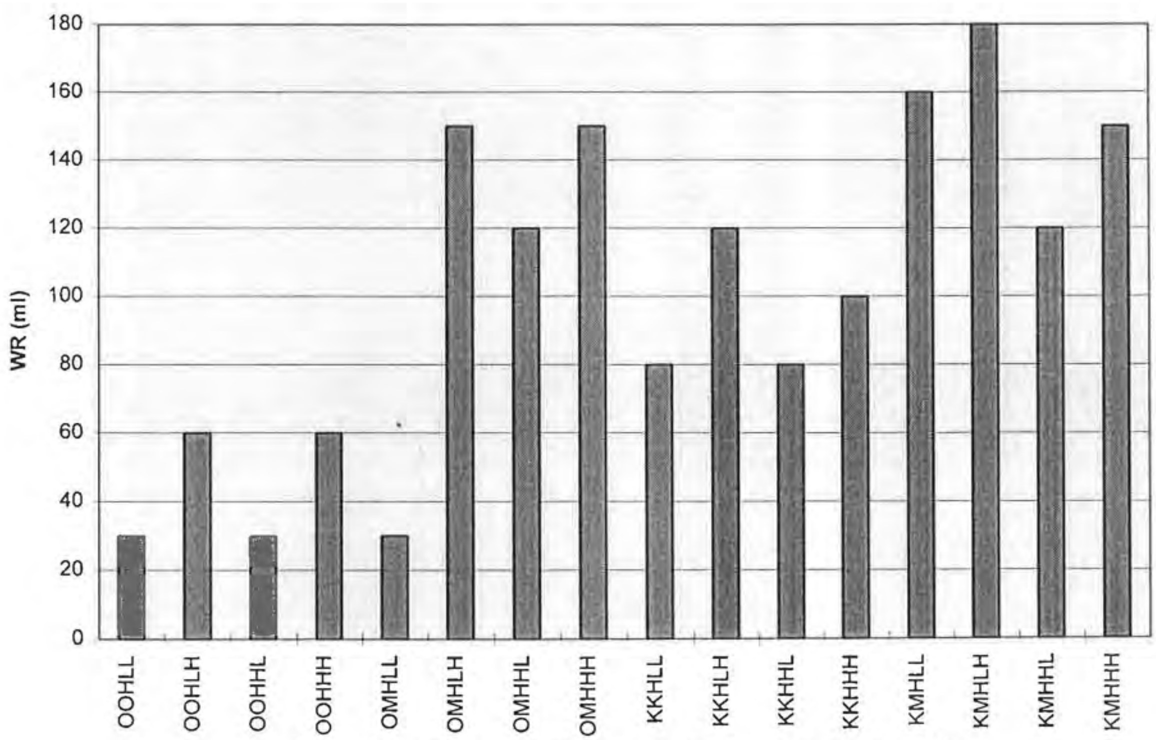


Fig 6.1b Preliminary Mix Water Reducing Agents (ml) [w/c=0.44]

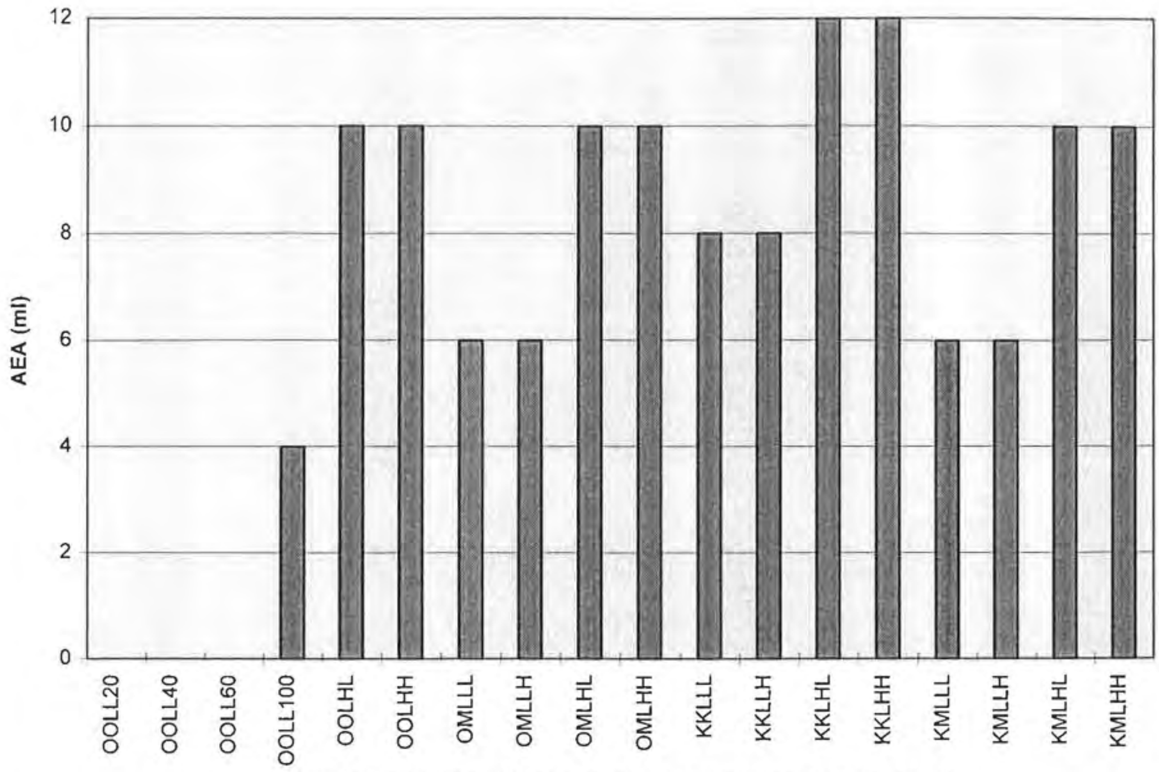


Fig 6.2a Preliminary Mix Air Entraining Admixtures (ml) [w/c=0.40]

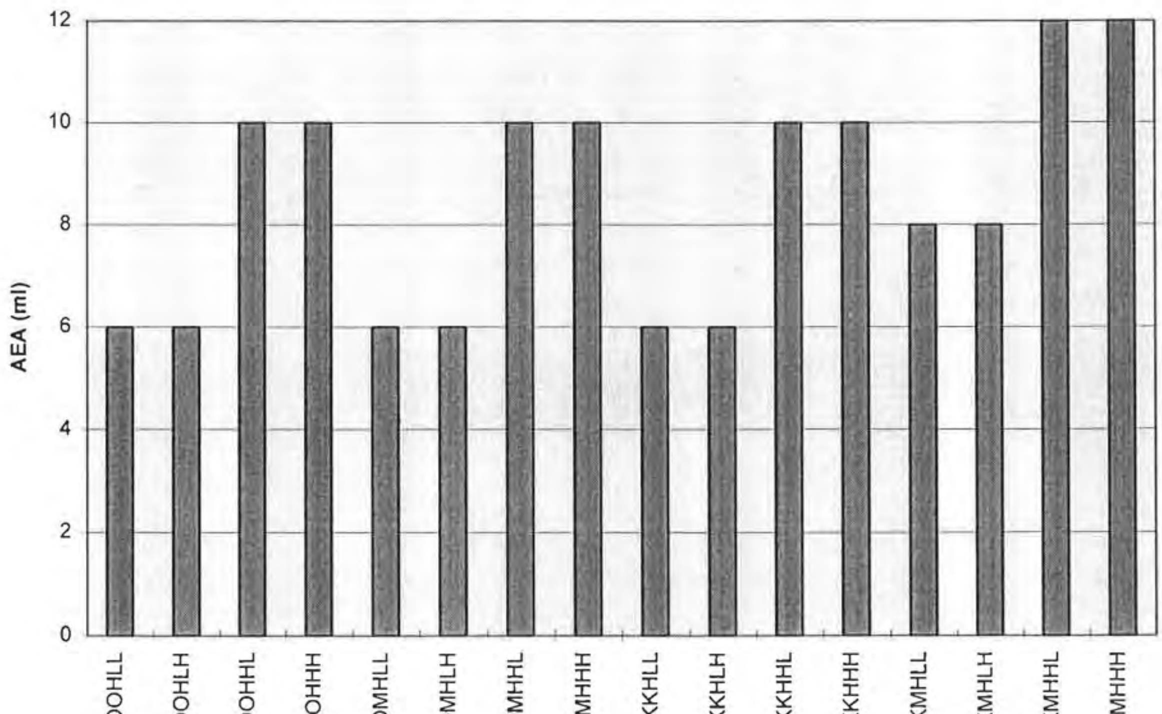


Fig 6.2b Preliminary Mix Air Entraining Admixtures (ml) [w/c=0.44]

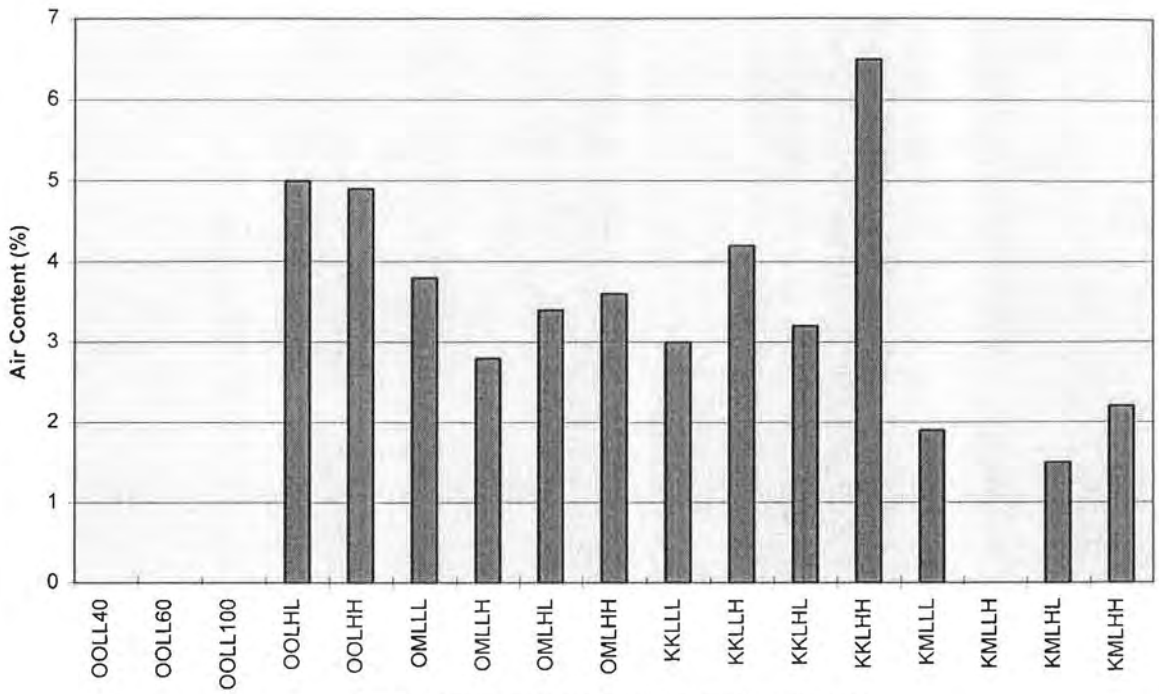


Fig 6.3a Preliminary Mix Air Content (%) [w/c=0.40]

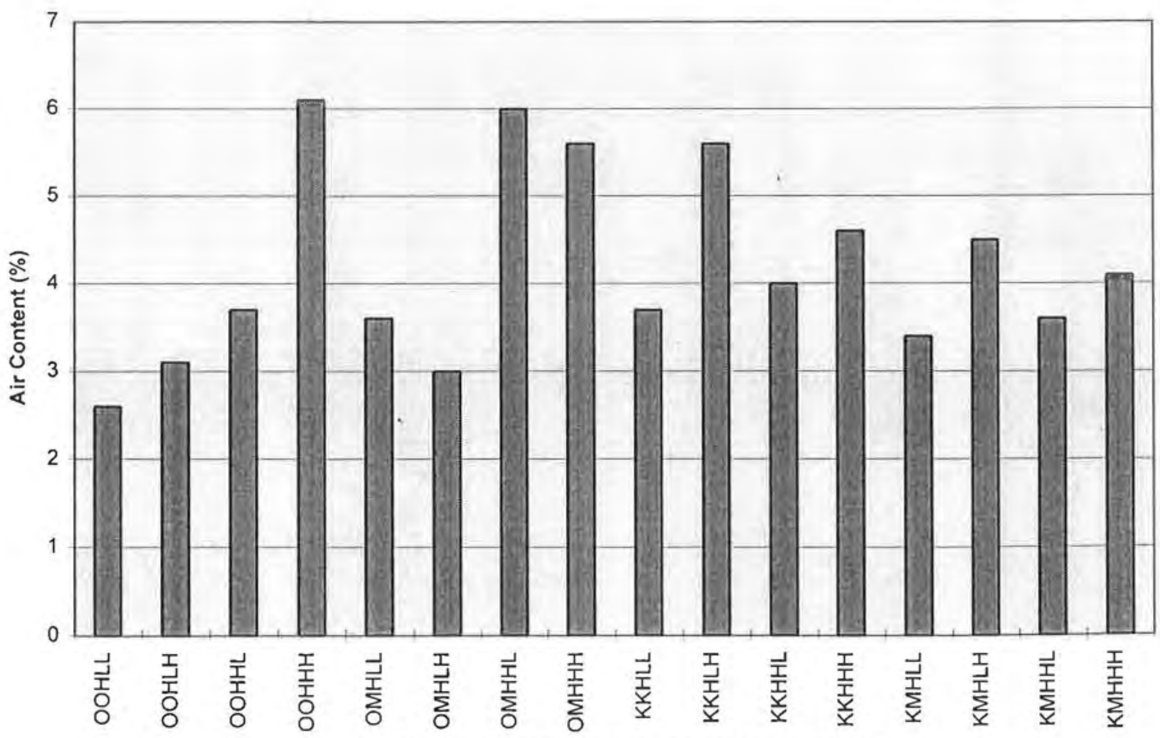


Fig 6.3b Preliminary Mix Air Contents (%) [w/c=0.44]

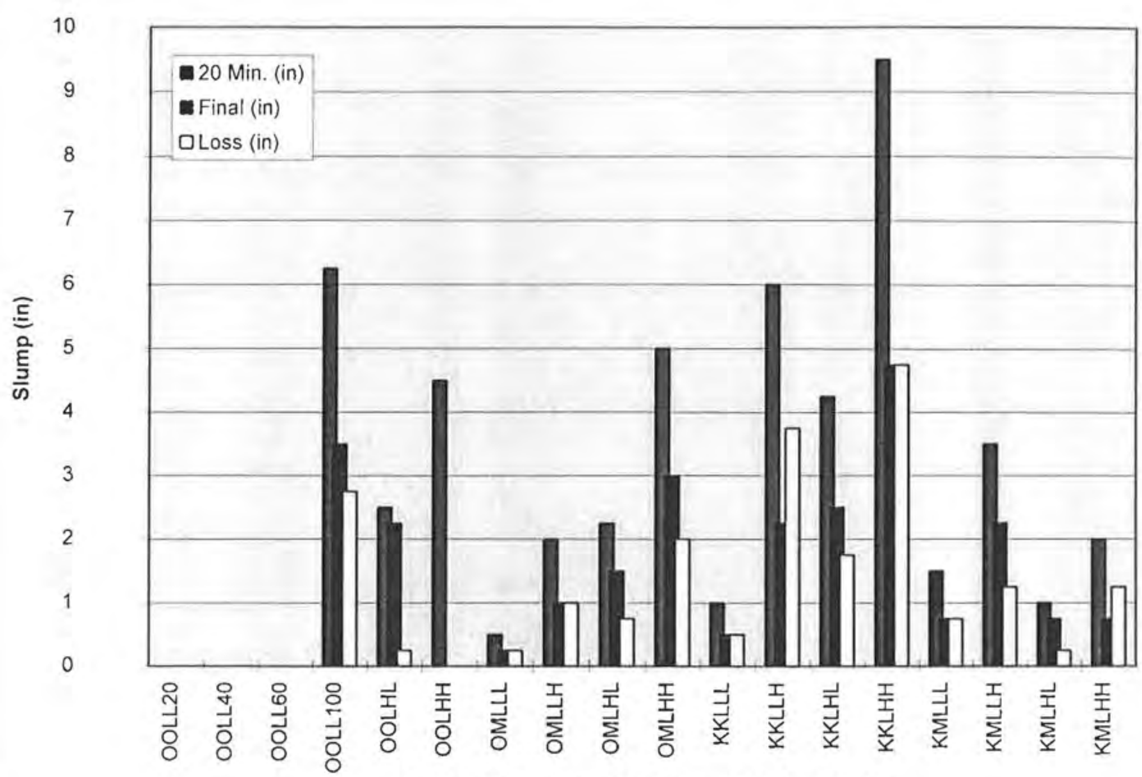


Fig 6.4a Preliminary Mix Slump Data [w/c=0.40]

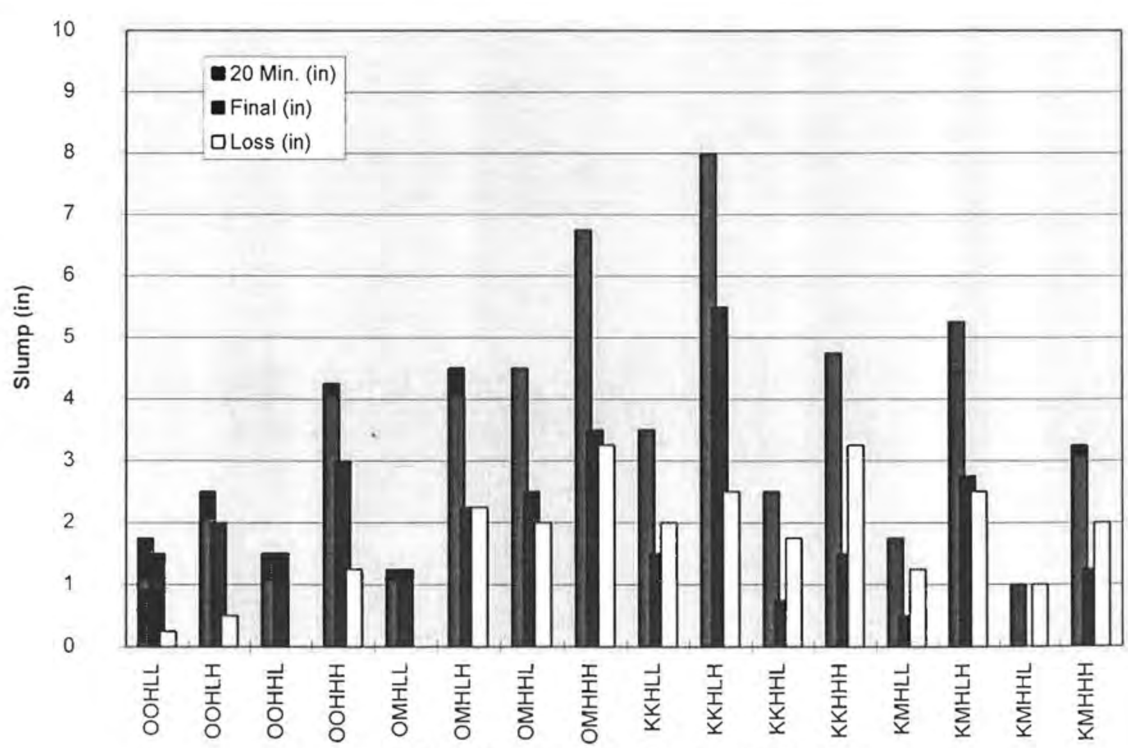


Fig 6.4b Preliminary Mix Slump Data [w/c=0.44]

Table 6.2 Preliminary Mix Strength Data[1]

Mix Number	Compressive Test-1	Compressive Test-2	Tensile Test-1	Tensile Test-2	Rounded Compressive[2]	Rounded Tensile[3]
OOLL20	4722	5305	440	471	**5010**	**455**
OOLL40	5146	5075	410	449	**5110**	**430**
OOLL60	4792	4845	364	428	**4820**	**395**
OOLL100	4421	5040	309	424	**4730**	**365**
OOLHL	4545	4333	354	405	**4440**	**380**
OOLHH	4333	4244	407	411	**4290**	**410**
OMLLL	5668	5836	401	442	**5750**	**420**
OMLLH	5509	5836	398	400	**5670**	**400**
OMLHL	5128	5155	398	316	**5140**	**355**
OMLHH	5323	5500	354	402	**5410**	**380**
KKLLL	6472	7781	586	550	**7130**	**570**
KKLLH	6950	6791	555	533	**6870**	**545**
KKLHL	7233	7357	496	528	**7300**	**510**
KKLHH	6225	6278	518	512	**6290**	**515**
KMLLL	7516	7321	516	552	**7420**	**535**
KMLLH	7357	7268	536	554	**7310**	**545**
KMLHL	6755	7215	549	502	**6990**	**525**
KMLHH	6702	6808	571	500	**6760**	**535**
OOHLL	4439	4615	402	442	**4530**	**420**
OOHLH	4775	4615	447	442	**4700**	**445**
OOHHL	4421	4421	354	398	**4420**	**375**
OOHHH	4085	3908	309	354	**4000**	**330**
OMHLL[4]	4775	5128	371	424	**4950**	**400**
OMHLH	4810	4669	336	363	**4740**	**350**
OMHHL	4226	4262	318	314	**4240**	**315**
OMHHH	4386	4280	367	354	**4330**	**360**
KKHLL	6366	6525	558	516	**6450**	**535**
KKHLH	5924	5853	491	509	**5890**	**500**
KKHHL	5853	6136	554	559	**6000**	**555**
KKHHH	5942	5871	492	499	**5910**	**495**
KMHLL	6578	7321	497	505	**6950**	**500**
KMHLH	6649	6649	574	528	**6650**	**550**
KMHHL	6619	6631	514	529	**6630**	**520**
KMHHH	6543	6295	541	541	**6420**	**540**

1 Preliminary Mixes were moist cured at 72 degrees F, for 28 days
2 Compressive strengths rounded to nearest 10 psi
3 Tensile strengths rounded to nearest 5 psi
4 Broken 7 days early (21 days)

Cement Type: XXXXX
OO-Type I
OM-Type I with MS
KK-Type K
KM-Type K with MS

Water Reducing Agent, High (H) or Low (L)
Air Entraining Agent, High (H) or Low (L)
Water/Cement Ratio: 0.44 (H) or 0.40 (L)

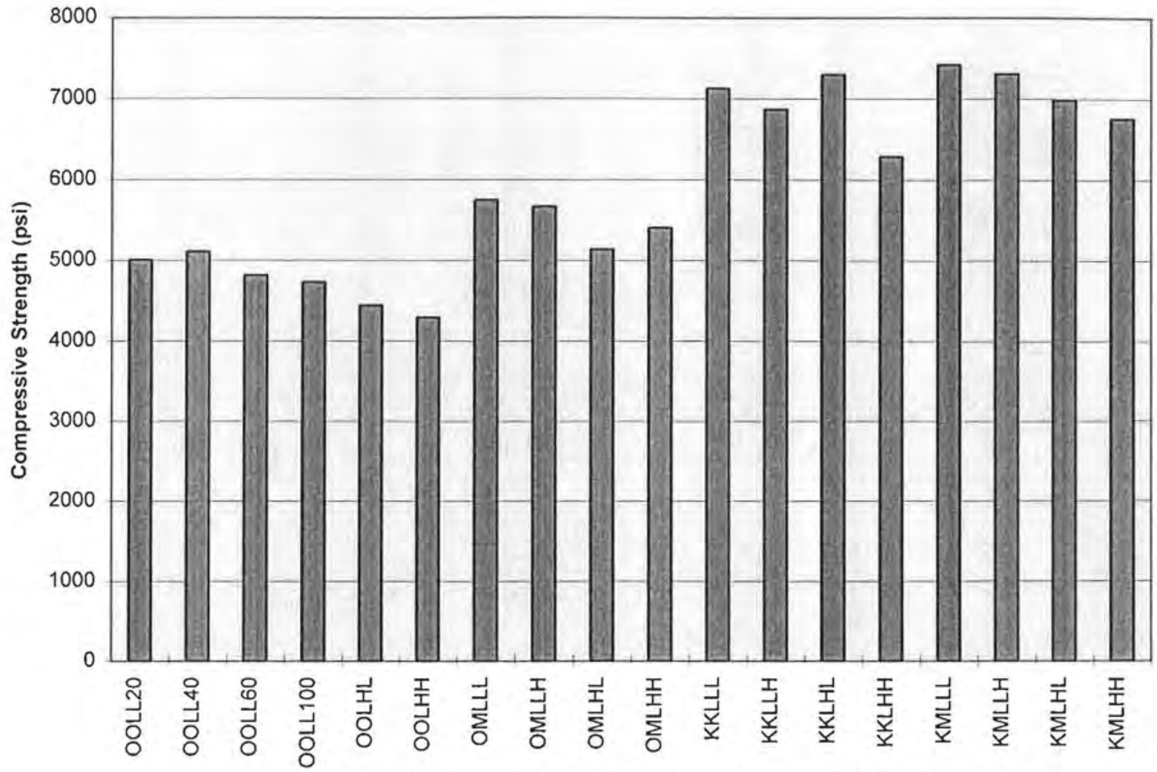


Fig 6.5a Preliminary Mix Compressive Strengths [w/c=0.40]

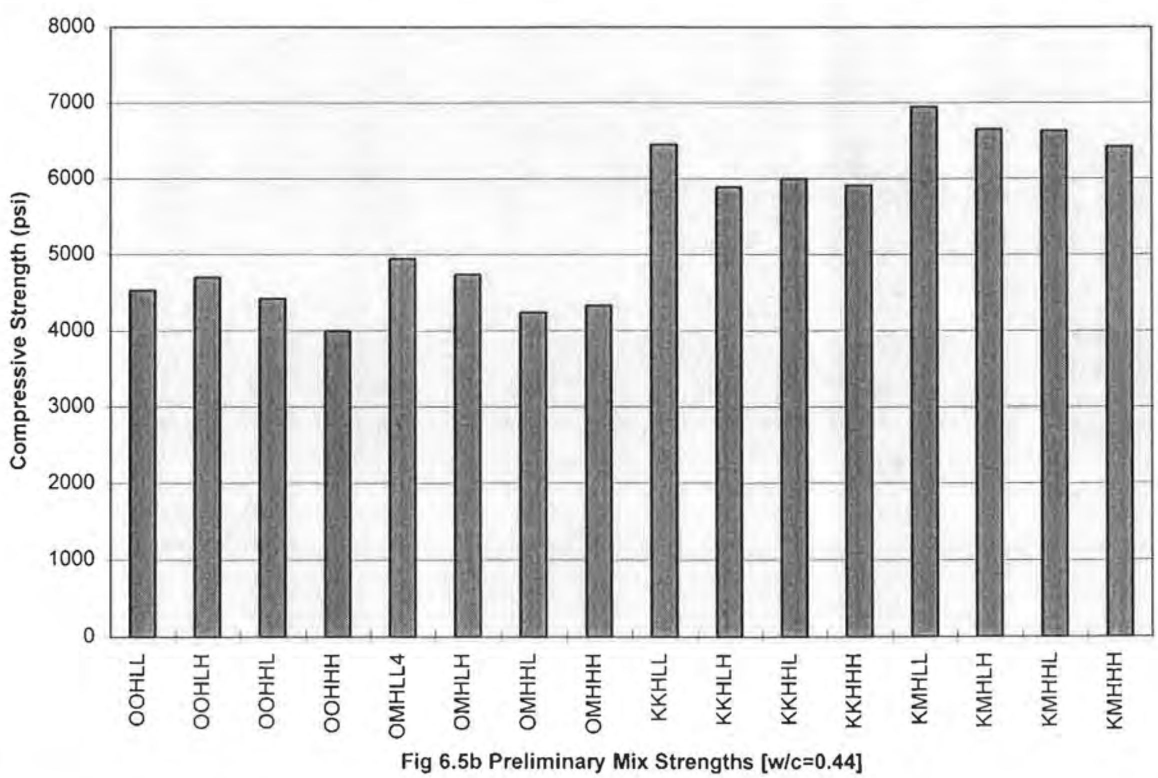


Fig 6.5b Preliminary Mix Strengths [w/c=0.44]

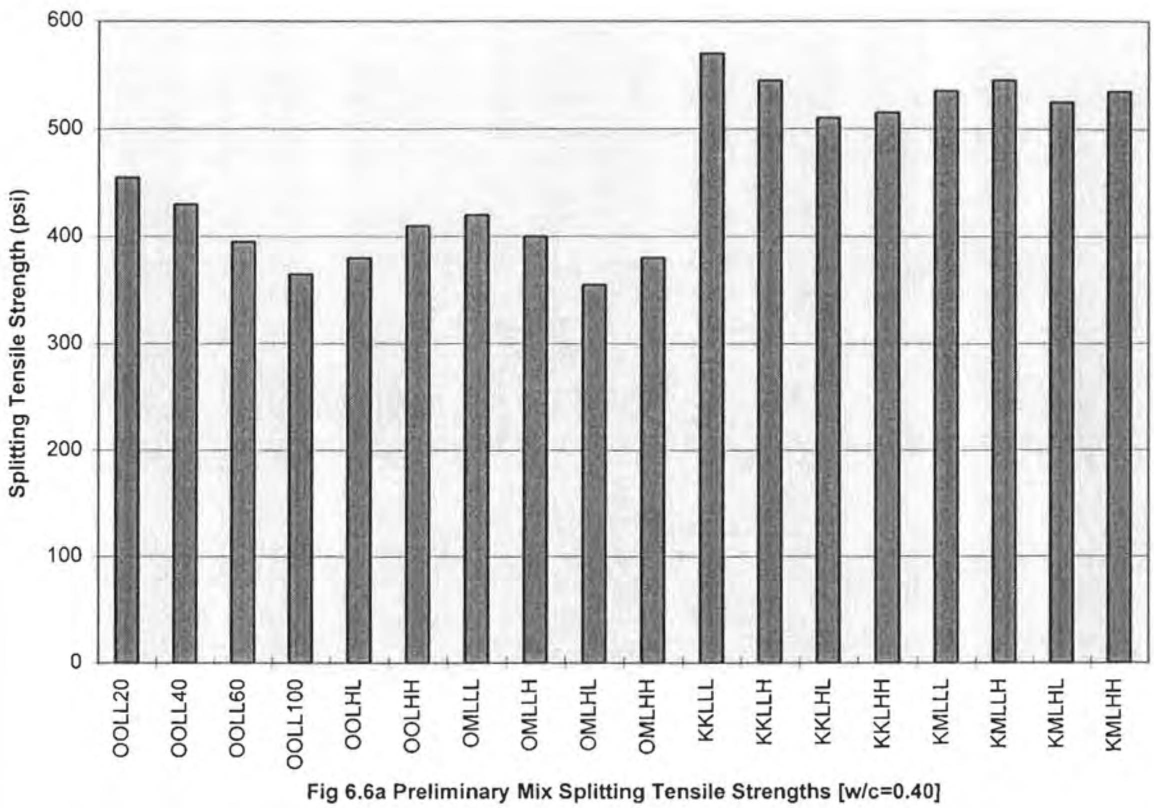


Fig 6.6a Preliminary Mix Splitting Tensile Strengths [w/c=0.40]

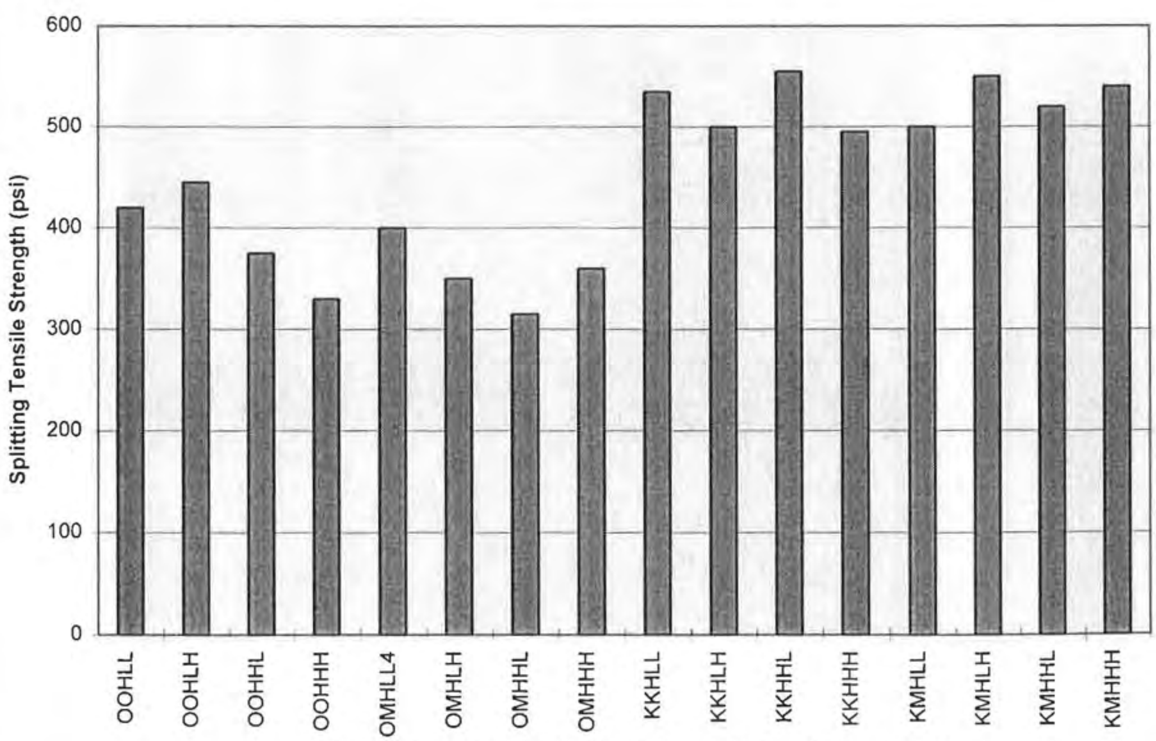


Fig 6.6b Preliminary Mix Splitting Tensile Strengths [w/c=0.44]

6.3 Restrained Expansions for Preliminary Mixes

Data obtained from restrained expansion length bars of preliminary mixes (for Type K mixes only) was very important in choosing mixes for the final mix procedures. The amount of expansion is critical in reducing shrinkage after moist curing, and reduced drying shrinkage cracking is the primary reason for considering Type K mixes. The length bar data is displayed in Table 6.3. Figure 6.7 is a graphical representation of the expansion of each mix over the 28-day moist cure test period. The values graphed are an average of the expansion of the two bars made from each preliminary mix. In the course of collecting the length bar data, a day or two was sometimes missed. In these cases, the length data was linearly interpolated between the two adjacent measured values. Also, each test specimen was corrected for its length relative to the comparator bar, i.e., the data was adjusted to give an initial length change of zero for each test specimen.

The restrained expansion bars were 10" in length and expansion ranged from 0.029% to 0.053%. From examination of Table 6.3 and Figure 6.7, an increase in water/cement ratio appears to increase expansion. In the preliminary mixes, microsilica mixes without a high water/cement ratio or high amounts of water-reducing agent had lower values of expansion.

Since most of the bars reached maximum expansion by day seven and remained at a fairly constant value after seven days, a statistical analysis of the 7-28 day expansions was performed. The results of this analysis are shown in Table 6.4, which includes a reordering of the preliminary mix designs in the lower portion of the table based on average expansion values of the mixes.

Table 6.3 Length Bar Analysis-Preliminary Mixes

Day	KKLLL	KKLLH	KKLHL	KKLHH	KKHLL	KKHLH	KKHHL	KKHHH	KMLLL	KMLLH	KMLHL	KMLHH	KMHLL	KMHLH	KMHHL	KMHHH
0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1	0.0018	0.0020	0.0015	0.0018	0.0031	0.0031	0.0016	0.0020	0.0020	0.0022	0.0018	0.0019	0.0007	0.0007	0.0021	0.0022
2	0.0023	0.0025	0.0024	0.0026	0.0048	0.0046	0.0020	0.0025	0.0027	0.0031	0.0025	0.0024	0.0015	0.0015	0.0027	0.0029
3	0.0027	0.0028	0.0040	0.0030	0.0048	0.0048	0.0024	0.0030	0.0027	0.0033	0.0029	0.0031	0.0022	0.0020	0.0032	0.0034
4	0.0029	0.0030	0.0046	0.0035	0.0049	0.0048	0.0027	0.0034	0.0029	0.0036	0.0031	0.0030	0.0025	0.0022	0.0035	0.0036
5	0.0028	0.0029	0.0050	0.0039	0.0051	0.0048	0.0028	0.0035	0.0029	0.0035	0.0035	0.0033	0.0027	0.0024	0.0037	0.0037
6	0.0029	0.0029	0.0053	0.0040	0.0052	0.0048	0.0030	0.0038	0.0031	0.0038	0.0033	0.0031	0.0025	0.0022	0.0038	0.0037
7	0.0028	0.0029	0.0053	0.0040	0.0050	0.0047	0.0031	0.0037	0.0030	0.0036	0.0031	0.0029	0.0027	0.0024	0.0037	0.0037
8	0.0030	0.0032	0.0056	0.0041	0.0052	0.0049	0.0033	0.0039	0.0028	0.0034	0.0028	0.0027	0.0026	0.0023	0.0037	0.0037
9	0.0029	0.0031	0.0054	0.0039	0.0052	0.0048	0.0034	0.0040	0.0026	0.0032	0.0026	0.0025	0.0027	0.0023	0.0038	0.0038
10	0.0030	0.0032	0.0055	0.0040	0.0055	0.0052	0.0035	0.0041	0.0024	0.0029	0.0027	0.0027	0.0028	0.0022	0.0038	0.0038
11	0.0028	0.0030	0.0056	0.0038	0.0059	0.0055	0.0032	0.0038	0.0024	0.0031	0.0027	0.0026	0.0026	0.0022	0.0036	0.0036
12	0.0027	0.0030	0.0056	0.0040	0.0063	0.0058	0.0034	0.0039	0.0024	0.0030	0.0029	0.0026	0.0025	0.0021	0.0038	0.0037
13	0.0026	0.0028	0.0054	0.0038	0.0060	0.0057	0.0033	0.0038	0.0027	0.0034	0.0031	0.0024	0.0024	0.0020	0.0037	0.0037
14	0.0027	0.0023	0.0052	0.0035	0.0062	0.0057	0.0032	0.0039	0.0031	0.0038	0.0028	0.0022	0.0023	0.0021	0.0037	0.0037
15	0.0026	0.0023	0.0051	0.0033	0.0062	0.0058	0.0034	0.0039	0.0030	0.0038	0.0028	0.0022	0.0023	0.0020	0.0039	0.0038
16	0.0025	0.0023	0.0050	0.0031	0.0061	0.0059	0.0032	0.0038	0.0029	0.0037	0.0029	0.0021	0.0022	0.0019	0.0039	0.0038
17	0.0023	0.0023	0.0052	0.0031	0.0064	0.0058	0.0031	0.0037	0.0027	0.0036	0.0029	0.0021	0.0022	0.0019	0.0039	0.0038
18	0.0022	0.0023	0.0051	0.0032	0.0061	0.0057	0.0029	0.0036	0.0026	0.0035	0.0028	0.0020	0.0022	0.0019	0.0038	0.0038
19	0.0022	0.0024	0.0051	0.0031	0.0059	0.0053	0.0030	0.0037	0.0028	0.0034	0.0030	0.0021	0.0022	0.0019	0.0038	0.0038
20	0.0022	0.0024	0.0049	0.0030	0.0058	0.0053	0.0030	0.0036	0.0030	0.0037	0.0029	0.0020	0.0021	0.0018	0.0036	0.0035
21	0.0022	0.0027	0.0050	0.0028	0.0057	0.0051	0.0029	0.0036	0.0029	0.0035	0.0033	0.0023	0.0022	0.0019	0.0038	0.0043
22	0.0020	0.0028	0.0049	0.0028	0.0056	0.0049	0.0029	0.0035	0.0031	0.0039	0.0035	0.0024	0.0021	0.0018	0.0037	0.0037
23	0.0021	0.0028	0.0048	0.0028	0.0056	0.0052	0.0029	0.0035	0.0031	0.0041	0.0036	0.0025	0.0021	0.0018	0.0038	0.0038
24	0.0020	0.0028	0.0048	0.0028	0.0055	0.0052	0.0029	0.0035	0.0032	0.0042	0.0033	0.0023	0.0022	0.0019	0.0039	0.0039
25	0.0019	0.0028	0.0047	0.0026	0.0055	0.0051	0.0028	0.0035	0.0030	0.0038	0.0035	0.0025	0.0022	0.0019	0.0040	0.0039
26	0.0019	0.0028	0.0049	0.0028	0.0055	0.0051	0.0029	0.0036	0.0031	0.0040	0.0033	0.0023	0.0022	0.0019	0.0038	0.0037
27	0.0019	0.0026	0.0048	0.0028	0.0054	0.0050	0.0029	0.0036	0.0031	0.0039	0.0033	0.0022	0.0021	0.0019	0.0038	0.0037
28	0.0021	0.0029	0.0048	0.0030	0.0056	0.0051	0.0029	0.0035	0.0030	0.0039	0.0034	0.0024	0.0021	0.0018	0.0043	0.0038

Cement Type: XXXXX
OO-Type I
OM-Type I with MS
KK-Type K
KM-Type K with MS
Water Reducing Agent, High (H) or Low (L)
Air Entraining Agent, High (H) or Low (L)
Water/Cement Ratio: 0.44 (H) or 0.40 (L)

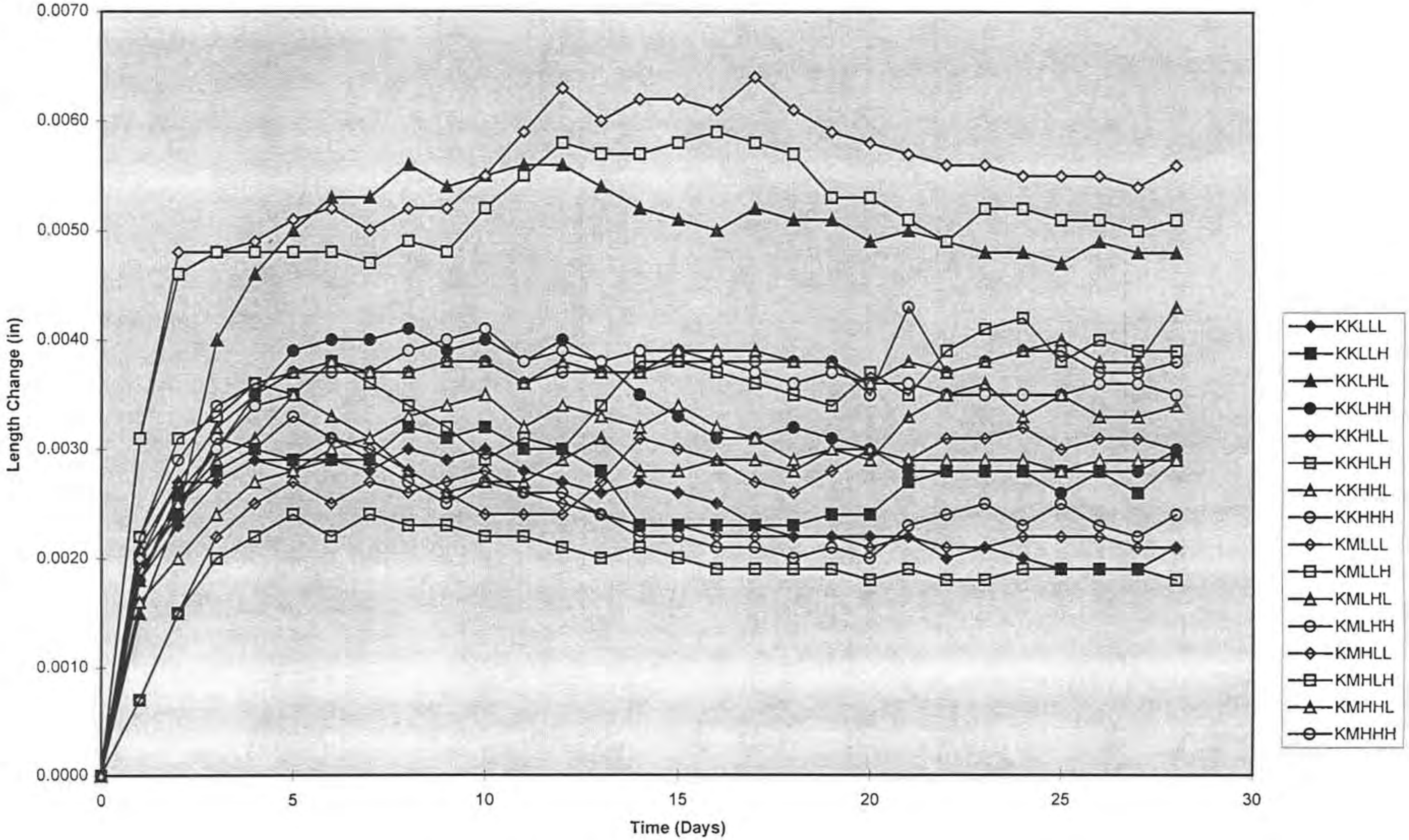


Fig 6.7. Length Bar Analysis - Preliminary Mixes

Table 6.4 Length Bar Analysis-Preliminary Mixes

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
	KKLLL	**KKLLH**	**KKLHL**	**KKLHH**	**KKHLL**	**KKHLH**	**KKHHL**	**KKHHH**	**KMLLL**	**KMLLH**	**KMLHL**	**KMLHH**	**KMHLL**	**KMHLH**	**KMHHL**	**KMHHH**
Sum	0.0526	0.0597	0.1127	0.0723	0.1262	0.1168	0.0681	0.0817	0.0629	0.0794	0.0672	0.0520	0.0510	0.0439	0.0838	0.0830
Average	0.0024	0.0027	0.0051	0.0033	0.0057	0.0053	0.0031	0.0037	0.0029	0.0036	0.0031	0.0024	0.0023	0.0020	0.0038	0.0038
Mode	0.0022	0.0028	0.0048	0.0028	0.0055	0.0051	0.0029	0.0036	0.0030	0.0034	0.0028	0.0025	0.0022	0.0019	0.0038	0.0038
Std. Dev.	0.0004	0.0003	0.0003	0.0005	0.0004	0.0004	0.0002	0.0002	0.0003	0.0004	0.0003	0.0002	0.0002	0.0002	0.0001	0.0001
Maximum	0.0030	0.0032	0.0056	0.0041	0.0064	0.0059	0.0035	0.0041	0.0032	0.0042	0.0036	0.0029	0.0028	0.0024	0.0043	0.0043
Minimum	0.0019	0.0023	0.0047	0.0026	0.0050	0.0047	0.0028	0.0035	0.0024	0.0029	0.0026	0.0020	0.0021	0.0018	0.0036	0.0035
Range	0.0011	0.0009	0.0009	0.0015	0.0014	0.0012	0.0007	0.0006	0.0008	0.0013	0.0010	0.0009	0.0007	0.0006	0.0007	0.0008

Expansion Arranged By Average

Greatest	----------	----------	----------	----------	----------	----------	----------	----------	----------	----------	----------	----------	----------	----------	----------	Smallest
	5	6	3	15	16	8	10	4	7	11	9	2	1	12	13	14
	KKHLL	**KKHLH**	**KKLHL**	**KMHHL**	**KMHHH**	**KKHHH**	**KMLLH**	**KKLHH**	**KKHHL**	**KMLHL**	**KMLLL**	**KKLLH**	**KKLLL**	**KMLHH**	**KMHLL**	**KMHLH**
Sum	0.1262	0.1168	0.1127	0.0838	0.0830	0.0817	0.0794	0.0723	0.0681	0.0672	0.0629	0.0597	0.0526	0.0520	0.0510	0.0439
Average	0.0057	0.0053	0.0051	0.0038	0.0038	0.0037	0.0036	0.0033	0.0031	0.0031	0.0029	0.0027	0.0024	0.0024	0.0023	0.0020
Mode	0.0055	0.0051	0.0048	0.0038	0.0038	0.0036	0.0034	0.0028	0.0029	0.0028	0.0030	0.0028	0.0022	0.0025	0.0022	0.0019
Std. Dev.	0.0004	0.0004	0.0003	0.0001	0.0001	0.0002	0.0004	0.0005	0.0002	0.0003	0.0003	0.0003	0.0004	0.0002	0.0002	0.0002
Maximum	0.0064	0.0059	0.0056	0.0043	0.0043	0.0041	0.0042	0.0041	0.0035	0.0036	0.0032	0.0032	0.0030	0.0029	0.0028	0.0024
Minimum	0.0050	0.0047	0.0047	0.0036	0.0035	0.0035	0.0029	0.0026	0.0028	0.0026	0.0024	0.0023	0.0019	0.0020	0.0021	0.0018
Range	0.0014	0.0012	0.0009	0.0007	0.0008	0.0006	0.0013	0.0015	0.0007	0.0010	0.0008	0.0009	0.0011	0.0009	0.0007	0.0006

Cement Type: X X X X X
OO-Type I
OM-Type I with MS
KK-Type K
KM-Type K with MS

Water Reducing Agent, High (H) or Low (L)
Air Entraining Agent, High (H) or Low (L)
Water/Cement Ratio: 0.44 (H) or 0.40 (L)

6.4 Final Mix Data/Results

After completion of the preliminary mixing study, the preliminary mixes were analyzed and values for admixtures and water/cement ratio were selected for the final mixes. The final mixes were then subjected to the same testing procedure as the preliminary mixes. However, in the final mix design testing, specimens were constructed, cured, and tested under different temperature and humidity conditions to simulate different bridge site weather conditions during the year. Four conditions were used in the laboratory testing and were,

C - Specimens were constructed with cold materials (40°F) and cured at 40°F for 28 days at 100% humidity.

H- Specimens were constructed with hot materials (90°F) and cured at 90°F for 28 days at 100 % humidity.

M - Specimens were constructed with moderate temperature materials (72°F) and cured at 72°F for 28 days at 100% humidity.

D - Specimens were constructed with moderate temperature materials (72°F) and cured at 72°F for 28 days at 40% humidity.

A summary of final mix data is shown in Table 6.5. Listed are values of compressive strength, splitting tensile strength, water-reducing agent, air-entraining agent, slump, air content, unit weight, and 7-day expansion of Type K length bars. All of the final mixes had a water/cement ratio of 0.44. Data for length bars for final mixes can be found in Section 6.5.

Admixture amounts varied for each final mix. Volume of water-reducing agent in milliliters used in each final mix is displayed in Figure 6.8. Figure 6.9 shows the milliliters of air-entraining agent used in each final mix. The amounts of air-entraining agent and water-reducing agent were more consistent in the final mixes because optimum values for cement types were estimated from the preliminary mixing plan.

Air content for the different mixes is found in Figure 6.10. For final mixes, the temperature conditions had little or no effect on air content. Lower values of air content were found for microsilica mixes. The Type K mixes without microsilica, KK, had higher air content than other mixes.

Slump data for the final mixes is presented in graphical form in Figures 6.11 and 6.12. The graphs display slump at 20 minutes and the final slump recorded in the mix procedure. A third bar in Figure 6.11 shows the loss in slump and is the difference in the twenty minute slump and the final slump. Type K mixes showed significantly higher slump loss than Type I mixes. In addition, the hot condition created a very high slump loss in the Type K mixes.

Table 6.6 is a list of the final mixes and their average compressive and splitting tensile strengths. Graphical comparison of compressive strength of final mixes is found in Figure 6.13. The light bar indicates the average compressive strength for two cylinders which were cured at 72°F in the moisture cabinet. For the hot and cold temperature mixes, the remaining cylinders for compression testing, two for Type K mixes and three for Type I mixes, were cured in a submerged state at the temperature of the stated test environmental condition, i.e., 90°F and 40°F respectively. This submerged curing does not apply to the dry cured mixes or the moist mixes which were cured in the moisture cabinet at 100% humidity. Compressive testing revealed that if properly cured, all of the mixes met strength requirements (4000 psi). The condition of dry curing decreased compressive strength around 1000 psi. As for temperature effects, cold temperatures tended to increase the compressive strength for Type K mixes and the hot condition caused a slight decrease in strength.

Two cylinders from each final mix were tested for splitting tensile strength except for the Type I moist cured mixes from which three cylinders were tested. Splitting tensile cylinders were cured at the specified environmental condition. For moist mixes, the cylinders were cured in the moisture cabinet and, for hot and cold mixes, cylinders were cured at the specified temperature completely submerged. Values for splitting tensile strength are shown in Figure 6.14. Curing

Table 6.5 Final Mix Data

Mix No.	w/c Ratio	WR (ml)	Air AEA (ml)	Air (%)	Air Time[1] (min)	Slump 20 Min. (in)	Slump Final (in)	Slump Time[1] (min)	Slump Loss (in)	Unit Wt (pcf)	Exp[4] (%)	Comp[2] Moist (psi)	Comp[2] @Cond (psi)	Tens[3] Cond (psi)
OO-C	0.44	55	12	5.3	32	3 3/4	3 1/4	32	1/2	144.8		5660	5860	475
OM-C	0.44	160	15	4.4	33	2 1/2	2	28	1/2	144.5		6150	5610	435
KK-C	0.44	170	12	4.8	36	5	2 1/2	36	2 1/2	144.8	0.037	7450	7080	490
KM-C	0.44	305	14	3.5	47	3	2	38	1	146.3	0.029	8370	7220	475
OO-D	0.44	50	12	4.5	34	2 1/2	2 1/4	34	1/4	146.2		5240	3570	445
OM-D	0.44	160	17	4	36	1 1/2	1 1/2	28	0	145.6		5710	4170	480
KK-D	0.44	200	12	6.6	34	7 1/2	4	34	3 1/2	143.2	-0.018	6510	5190	545
KM-D	0.44	270	17	3.8	32	2 1/2	1 1/4	31	1 1/4	146.6	-0.032	7090	5630	615
OO-H	0.44	50	12	5.2	40	3 1/2	2 1/2	34	1	144.8		4740	4770	430
OM-H	0.44	140	17	4.3	34	1 1/4	1	34	1/4	145.7		5570	5390	410
KK-H	0.44	200	12	6.2	39	6 1/2	2	39	4 1/2	145.2	0.041	6280	6300	565
KM-H	0.44	270	17	4.6	34	5	1 1/2	34	3 1/2	145.2	0.041	6700	7190	560
OO-M	0.44	80	10	4.9	32	5	4 1/2	25	1/2	144.8		4450	4580	380
OM-M	0.44	140	15	3.8	33	2 1/2	1	35	1 1/2	145.5		5020	5090	380
KK-M	0.44	200	13	7.2	30	8	4 3/4	38	3 1/4	142.3	0.041	6620	6300	485
KM-M	0.44	240	12	2.5	38	3	1 1/4	32	1 3/4	146.9	0.045	6800	6960	505

1 Elapsed Time from initiation of mixing
2 Rounded to nearest 10 psi
3 Rounded to nearest 5 psi
4 Negative numbers indicate shrinkage

Mix/Construction/Curing Code:

XX-X

Cement Type:
OO-Type I
OM-Type I with MS
KK-Type K
KM-Type K with MS

Construction/Curing Condition:
C-Cold: Mixed and Moist Cured, both at 40 degrees F
D-Dry: Mixed and Cured (40%RH), both at 72 degrees F
H-Hot: Mixed and Moist Cured, both at 90 degrees F
M-Moist: Mixed and Moist Cured, both at 72 degrees F

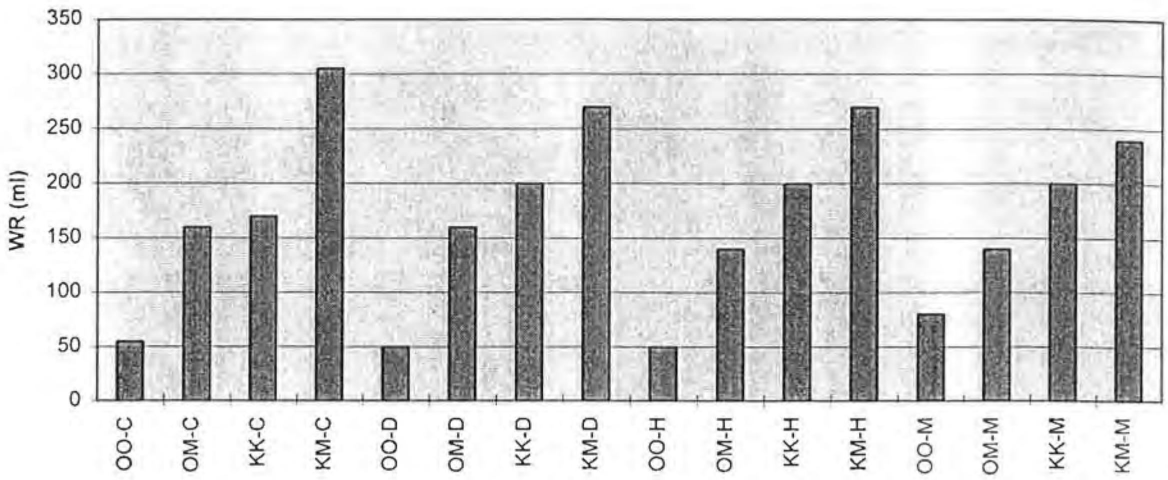


Fig 6.8. Final Mix Water Reducing Agents (ml)

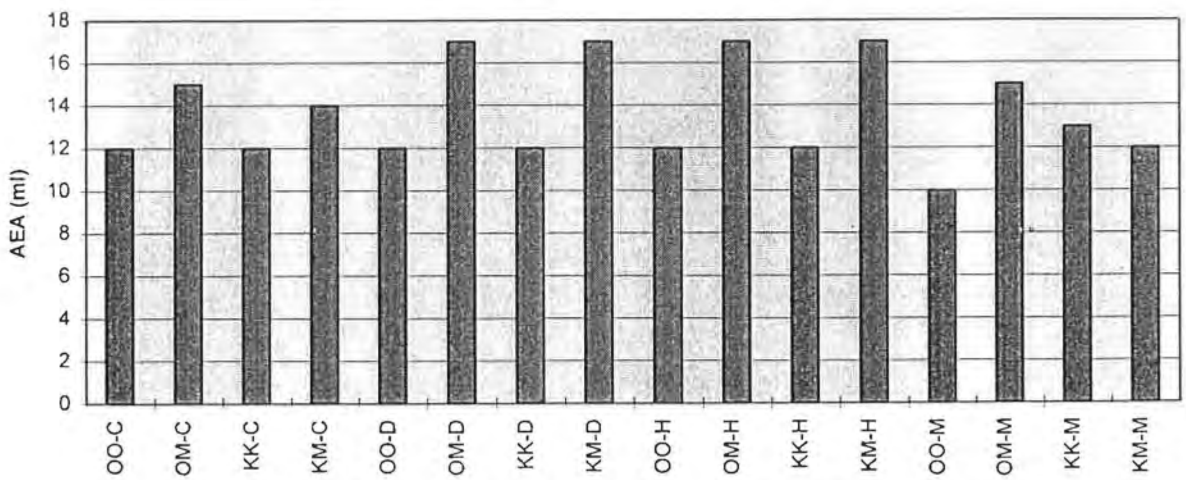


Fig 6.9. Final Mix Air Entraining Admixtures (ml)

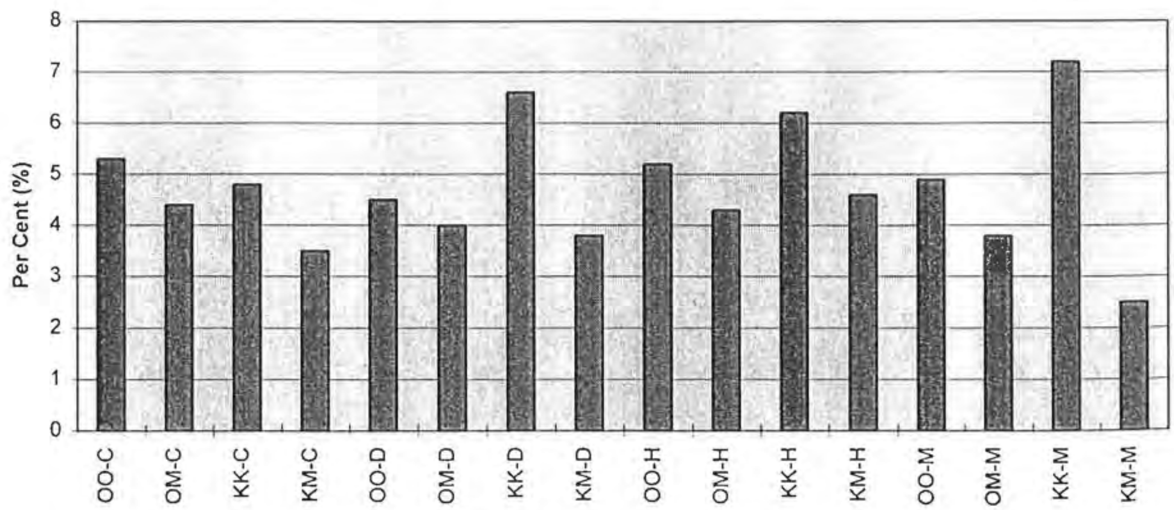


Fig 6.10. Final Mix Air Contents (%)

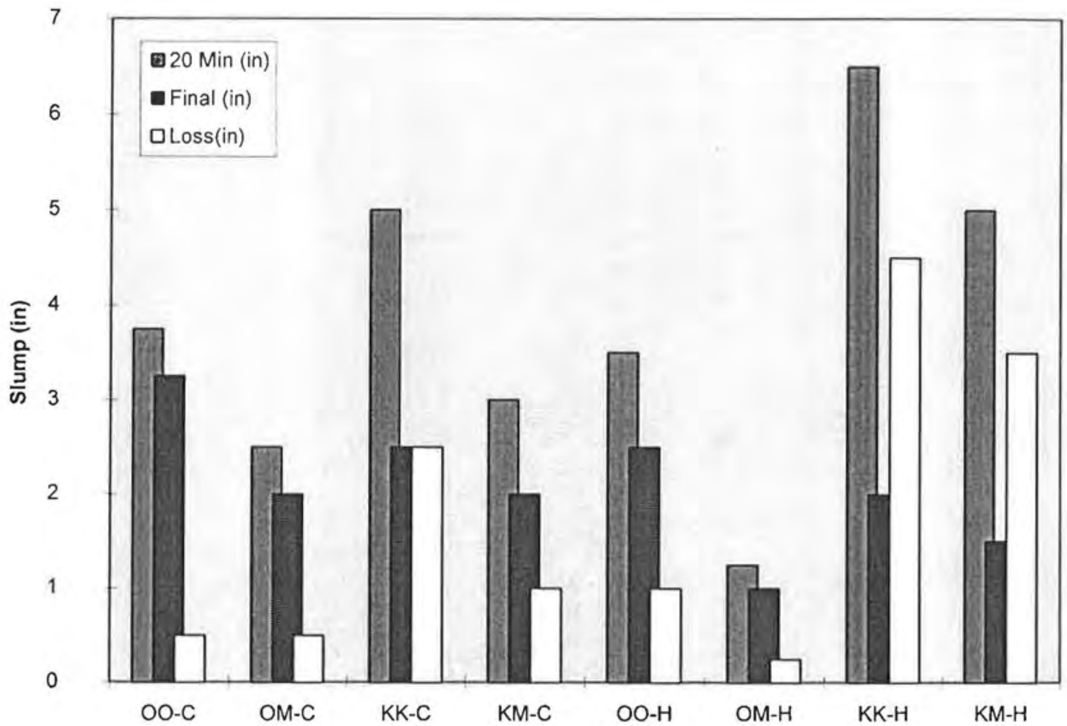


Fig 6.11a Final Mix Slump Data (Cold & Hot Mixes)

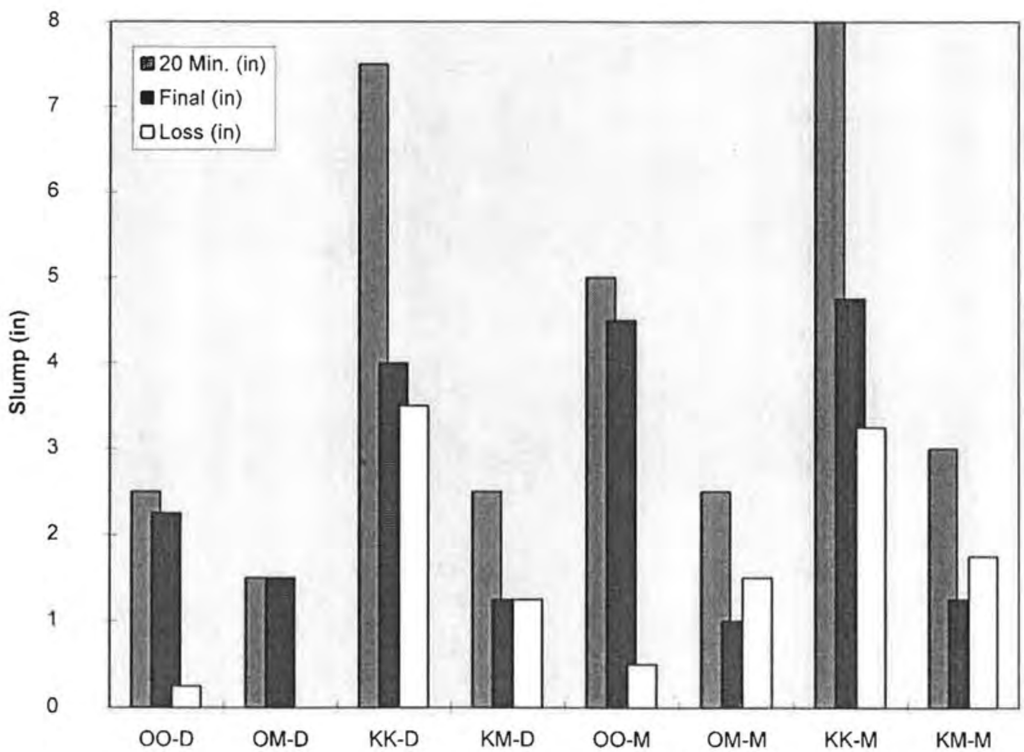


Fig 6.11b Final Slump Data (Dry & Moist Mixes)

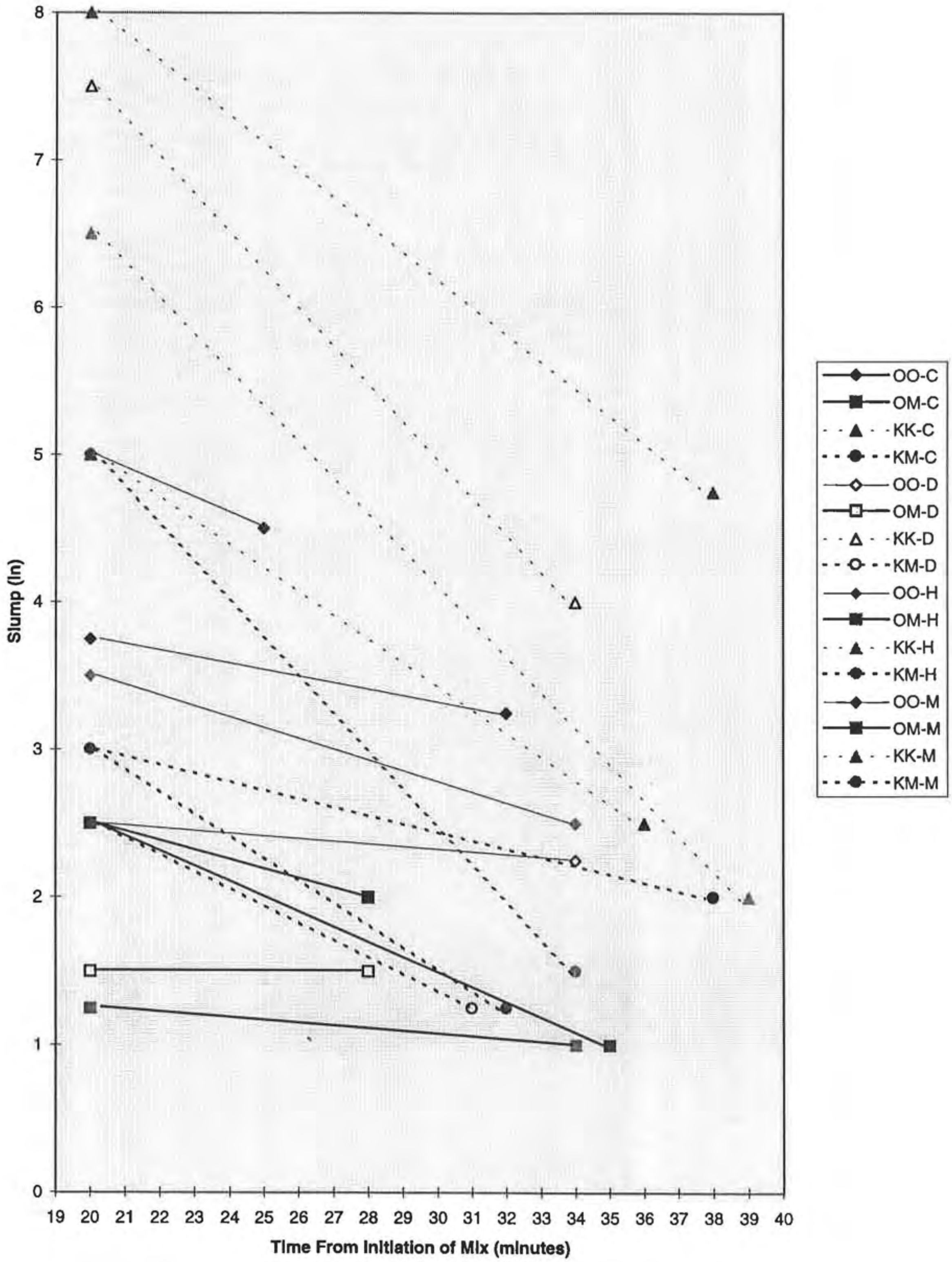


Fig 6.12 Final Mix Slump Loss with Time

Table 6.6 Final Mix Strength Data[1]

Mix Number	72 Degree Moist Cure			Varied Construction/Curing Condition							
	Comp 1	Comp 2	Average	Comp1	Comp2	Comp3	Tens1	Tens2	Tens3	Comp	Tensile
OO-C	5588	5730	5660	5482	5995	6101	493	458		5860	475
OM-C	6472	5836	6150	5641	5411	5765	448	426		5610	435
KK-C	7498	7392	7450	6808	7357		516	462		7080	490
KM-C	8400	8347	8370	7074	7357		471	474		7220	475
OO-D	5199	5270	5240	3714	3537	3466	449	445		3570	445
OM-D	5623	5800	5710	4280	4173	4067	487	471		4170	480
KK-D	6614	6402	6510	4951	5429		536	556		5190	545
KM-D	7109	7074	7090	5677	5588		602	624		5630	615
OO-H	4792	4686	4740	4898	4633	4792	443	420		4770	430
OM-H	5712	5429	5570	5323	5500	5358	412	405		5390	410
KK-H	6172	6384	6280	6402	6189		558	574		6300	565
KM-H	6791	6614	6700	7180	7197		565	554		7190	560
OO-M	4439	4456	4450	4545	4615		313	397	436	4580	380
OM-M	4951	5093	5020	5146	5040		357	371	411	5090	380
KK-M	6437	6808	6620	6225	6366		412	553		6300	485
KM-M	6808	6791	6800	6914	7003		474	534		6960	505

1 Final Mixes were cured at varying conditions, for 28 days
2 Compressive strengths rounded to nearest 10 psi
3 Tensile strengths rounded to nearest 5 psi

Cement Type: X X - X
OO-Type I
OM-Type I with MS
KK-Type K
KM-Type K with MS

Construction/Curing Condition
C-Cold: Mixed and moist cured, both at 40 degrees F
D-Dry: Mixed and cured (40% RH) at 72 degrees F
H-Hot: Mixed and moist cured, both at 90 degrees F
M-Moist: Mixed and moist cured at 72 degrees F

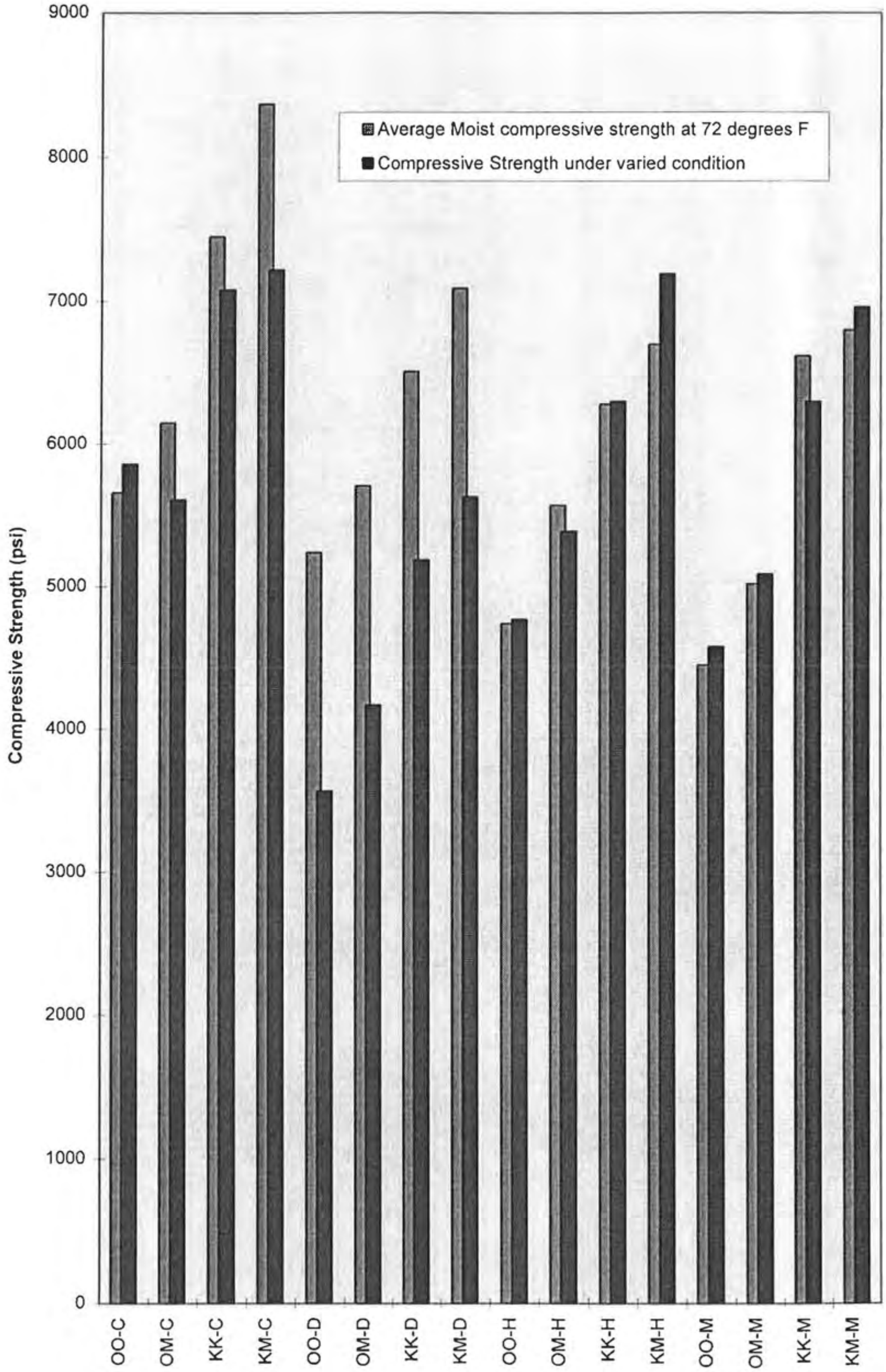


Fig 6.13. Final Mix Compressive Strengths (psi)

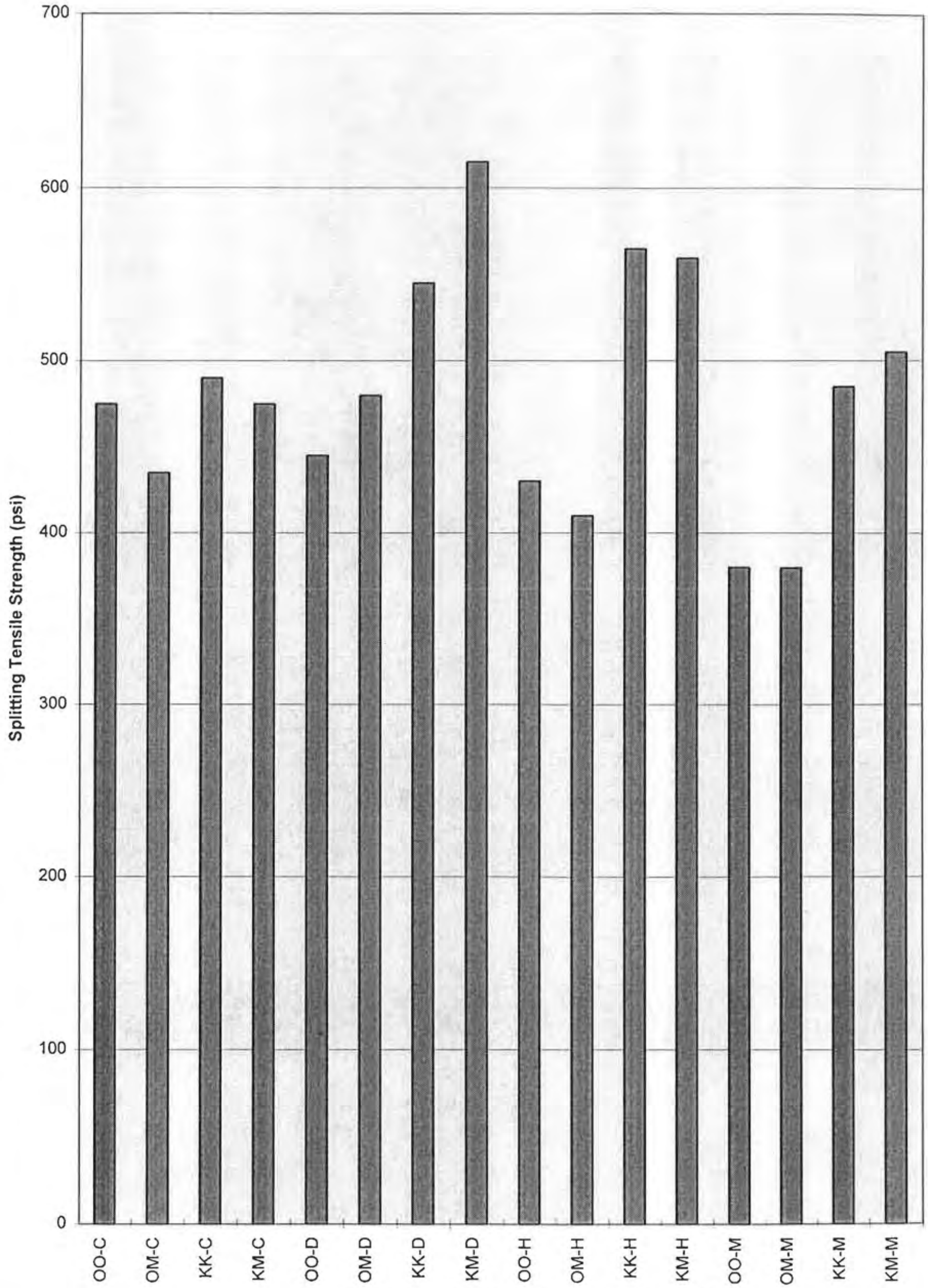


Fig 6.14 Final Mix Splitting Tensile Strengths (psi)

conditions did not appear to decrease splitting tensile strength. Surprisingly, Type K mixes showed an increase in tensile strength when dry cured or cured at the hot condition.

6.5 Restrained Expansions Data for Final Mixes

Analysis of restrained expansion length bar data for the final mixes was made in the same manner as that for preliminary mixes. Data for length bars is an average of the two bars made from each mix, and is given in Table 6.7, and presented graphically in Figure 6.15. Linear interpolation between adjacent measured values, and corrections for bars lengths to provide zero initial length changes were performed in the same manner as described for the preliminary mix bar testing.

Expansion data for final mixes was very interesting. Environmental conditions had a direct impact upon expansion. The hot condition did not reduce expansion values, but the cold did. The cold condition greatly increased the time of set for the Type K length bars. The bars had to be stripped at close to 18 hours instead of the 6-hour mark specified in the test. As can be seen in Figure 6.15, the slope of the expansion plots for the cold bars is much lower over the initial 7-days than the bars at the hot and normal conditions. Microsilica admixture did not have a great affect on expansion in the final mixes. Expansion values for the microsilica mixes were in the same range as for mixes without. Note in Table 6.7 (negative values) and Figure 6.15 how the dry curing condition was detrimental to expansion. The bars continued to shrink for the 28-day reading period. This data emphasizes the importance of proper curing procedures for Type K cement concrete.

A statistical analysis was conducted for the values of 7-28 day expansions, and results from that analysis are given in Table 6.8. The lower portion of this table provides a reordering of the mix designs based on average expansion values of the mixes.

Table 6.7 Length Bar Analysis-Final Mixes

Day	KK-C	KK-D	KK-H	KK-M	KM-C	KM-D	KM-H	KM-M
0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1	0.0008	-0.0003	0.0037	0.0024	0.0005	-0.0008	0.0037	0.0026
2	0.0016	-0.0003	0.0042	0.0032	0.0013	-0.0009	0.0041	0.0036
3	0.0024	-0.0007	0.0040	0.0036	0.0021	-0.0011	0.0040	0.0040
4	0.0033	-0.0009	0.0040	0.0038	0.0029	-0.0017	0.0040	0.0042
5	0.0036	-0.0012	0.0039	0.0040	0.0030	-0.0021	0.0039	0.0044
6	0.0037	-0.0015	0.0040	0.0043	0.0031	-0.0026	0.0039	0.0046
7	0.0037	-0.0018	0.0041	0.0041	0.0029	-0.0032	0.0041	0.0045
8	0.0040	-0.0019	0.0042	0.0040	0.0034	-0.0032	0.0041	0.0045
9	0.0039	-0.0018	0.0042	0.0043	0.0033	-0.0032	0.0041	0.0046
10	0.0038	-0.0020	0.0041	0.0044	0.0032	-0.0035	0.0041	0.0049
11	0.0039	-0.0020	0.0038	0.0042	0.0032	-0.0034	0.0038	0.0047
12	0.0040	-0.0023	0.0040	0.0040	0.0032	-0.0037	0.0040	0.0045
13	0.0041	-0.0026	0.0042	0.0040	0.0033	-0.0039	0.0039	0.0045
14	0.0040	-0.0027	0.0042	0.0040	0.0033	-0.0040	0.0041	0.0044
15	0.0039	-0.0027	0.0042	0.0041	0.0032	-0.0041	0.0041	0.0043
16	0.0039	-0.0029	0.0042	0.0041	0.0032	-0.0043	0.0041	0.0043
17	0.0039	-0.0030	0.0039	0.0041	0.0031	-0.0043	0.0038	0.0043
18	0.0039	-0.0031	0.0040	0.0039	0.0030	-0.0044	0.0038	0.0042
19	0.0040	-0.0031	0.0043	0.0042	0.0032	-0.0044	0.0041	0.0042
20	0.0040	-0.0032	0.0041	0.0044	0.0032	-0.0044	0.0039	0.0043
21	0.0040	-0.0033	0.0041	0.0043	0.0032	-0.0046	0.0040	0.0047
22	0.0040	-0.0034	0.0041	0.0038	0.0032	-0.0047	0.0040	0.0042
23	0.0039	-0.0035	0.0042	0.0038	0.0030	-0.0048	0.0041	0.0041
24	0.0039	-0.0033	0.0041	0.0038	0.0029	-0.0046	0.0039	0.0041
25	0.0038	-0.0040	0.0039	0.0039	0.0028	-0.0053	0.0037	0.0042
26	0.0039	-0.0041	0.0039	0.0039	0.0029	-0.0055	0.0037	0.0042
27	0.0037	-0.0043	0.0039	0.0040	0.0028	-0.0056	0.0037	0.0043
28	0.0036	-0.0045	0.0035	0.0041	0.0027	-0.0058	0.0035	0.0043

Mix/Construction/Curing Code

XX - X

Cement Type:	Construction/Curing Condition:
OO-Type I	C-Cold: Mixed and moist cured, both at 40 degrees F
OM-Type I with MS	D-Dry: Mixed and cured (40% RH) at 72 degrees F
KK-Type K	H-Hot: Mixed and moist cured, both at 40 degrees F
KM-Type K with MS	M-Moist: Mixed and moist cured, both at 72 degrees F

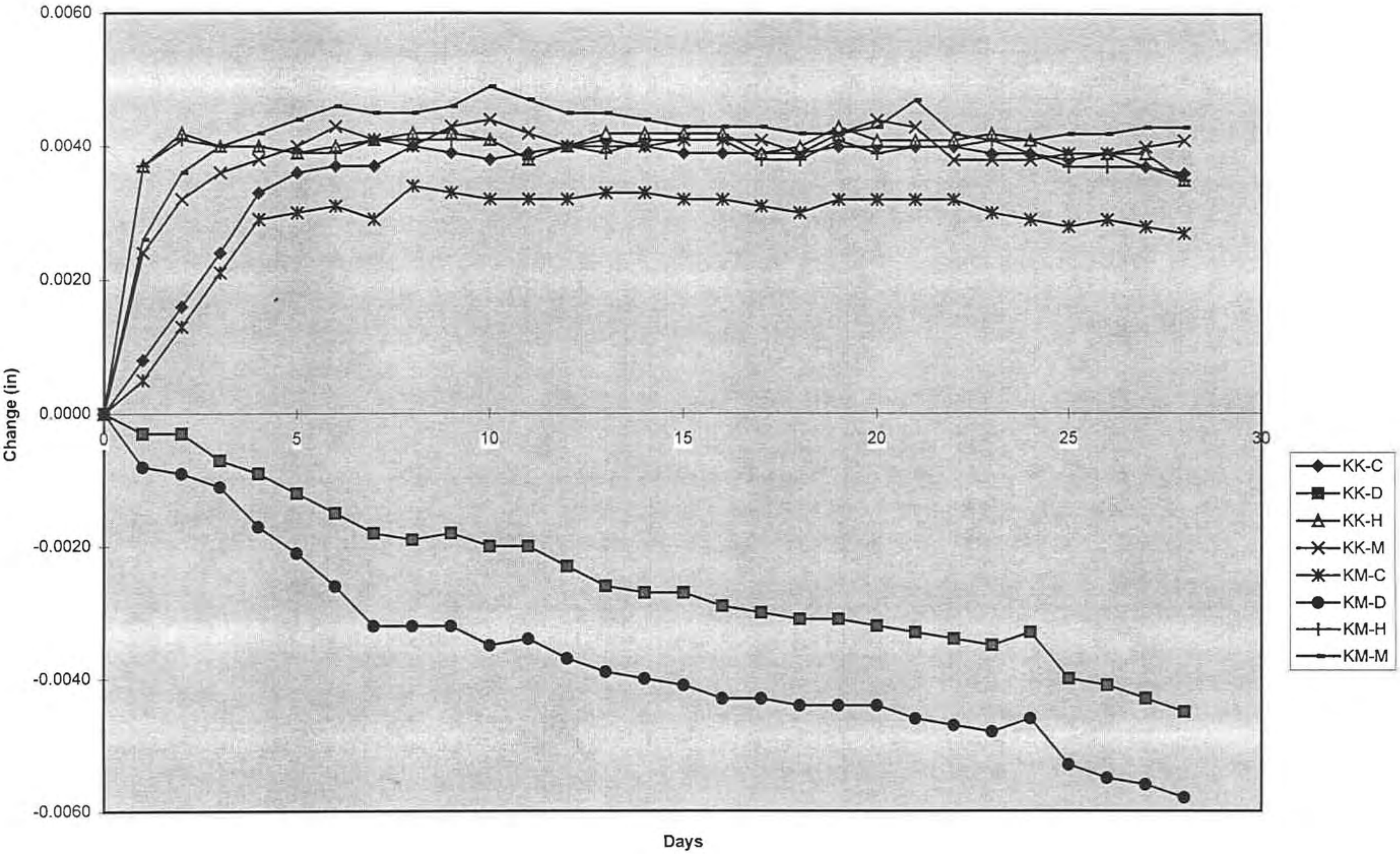


Fig 6.15. Length Bar Analysis - Final Mixes

Table 6.8 Length Bar Analysis-Final Mixes

	1	2	3	4	5	6	7	8
	KK-C	**KK-D**	**KK-H**	**KK-M**	**KM-C**	**KM-D**	**KM-H**	**KM-M**
Sum	0.0858	-0.0655	0.0892	0.0894	0.0682	-0.0949	0.0866	0.0963
Average	0.0039	-0.0030	0.0041	0.0041	0.0031	-0.0043	0.0039	0.0044
Mode	0.0039	-0.0018	0.0042	0.0041	0.0032	-0.0032	0.0041	0.0043
Std. Dev.	0.0001	0.0008	0.0002	0.0002	0.0002	0.0008	0.0002	0.0002
Maximum	0.0041	-0.0018	0.0043	0.0044	0.0034	-0.0032	0.0041	0.0049
Minimum	0.0036	-0.0045	0.0035	0.0038	0.0027	-0.0058	0.0035	0.0041
Range	0.0005	0.0027	0.0008	0.0006	0.0007	0.0026	0.0006	0.0008

Expansion Arranged by Average Reading

Greatest	--------------	--------------	--------------	--------------	--------------	--------------	--------------	Smallest
	8	**4**	**3**	**7**	**1**	**5**	**2**	**6**
	KM-M	**KK-M**	**KK-H**	**KM-H**	**KK-C**	**KM-C**	**KK-D**	**KM-D**
Sum	0.0963	0.0894	0.0892	0.0866	0.0858	0.0682	-0.0655	-0.0949
Average	0.0044	0.0041	0.0041	0.0039	0.0039	0.0031	-0.0030	-0.0043
Mode	0.0043	0.0041	0.0042	0.0041	0.0039	0.0032	-0.0018	-0.0032
Std. Dev.	0.0002	0.0002	0.0002	0.0002	0.0001	0.0002	0.0008	0.0008
Maximum	0.0049	0.0044	0.0043	0.0041	0.0041	0.0034	-0.0018	-0.0032
Minimum	0.0041	0.0038	0.0035	0.0035	0.0036	0.0027	-0.0045	-0.0058
Range	0.0008	0.0006	0.0008	0.0006	0.0005	0.0007	0.0027	0.0026

Mix/Construction/Curing Code

XX-X

Cement Type:	Construction/Curing Condition:
OO-Type I	C-Cold: Mixed and moist cured, both at 40 degrees F
OM-Type I with MS	D-Dry: Mixed and cured (40% RH) at 72 degrees F
KK-Type K	H-Hot: Mixed and moist cured, both at 40 degrees F
KM-Type K with MS	M-Moist: Mixed and moist cured, both at 72 degrees F

6.6 Restrained Expansions for Mortar Bars

Data for restrained expansion mortar bars was obtained in the same manner as data for the concrete length bars. The data presented in Table 6.9 and Figure 6.16 for each of the 2 replica specimens. These data were filled in by linear interpolation (for a few missing data points) and zeroed in the same manner as described earlier for the concrete length bars. Average expansions of the two bars for each type of mortar are given in the last four columns of Table 6.9.

Good results were obtained from the mortar bar tests. Type K cement mortar showed much greater expansion than Type I mortar. Microsilica, as found in the concrete bars, did not seem to harm expansion. In fact, as can be seen in Figure 6.16, the average expansion for the Type K bars with microsilica was greater.

Statistical analysis of the 7-28 day expansions was performed on the mortar bars in the same manner as for the concrete bars, and the results are given in Table 6.10.

Table 6.9 Mortar Length Bar Analysis

Day	MOO	MOO-2	MOM	MOM-2	MKK	MKK-2	MKM	MKM-2	Avg. MOO	Avg. MOM	Avg. MKK	Avg. MKM
0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1	0.0005	0.0006	0.0003	0.0008	0.0016	0.0020	0.0018	0.0019	0.0006	0.0006	0.0018	0.0019
2	0.0003	0.0009	0.0002	0.0010	0.0020	0.0033	0.0023	0.0030	0.0006	0.0006	0.0027	0.0027
3	0.0005	0.0007	0.0003	0.0011	0.0022	0.0033	0.0027	0.0037	0.0006	0.0007	0.0028	0.0032
4	0.0005	0.0006	0.0004	0.0010	0.0025	0.0033	0.0028	0.0036	0.0006	0.0007	0.0029	0.0032
5	0.0005	0.0006	0.0004	0.0009	0.0025	0.0033	0.0028	0.0036	0.0006	0.0007	0.0029	0.0032
6	0.0005	0.0006	0.0004	0.0009	0.0025	0.0032	0.0027	0.0036	0.0006	0.0007	0.0029	0.0032
7	0.0008	0.0007	0.0006	0.0008	0.0025	0.0031	0.0031	0.0034	0.0008	0.0007	0.0028	0.0033
8	0.0009	0.0007	0.0007	0.0008	0.0027	0.0031	0.0032	0.0034	0.0008	0.0008	0.0029	0.0033
9	0.0008	0.0008	0.0006	0.0009	0.0027	0.0032	0.0031	0.0036	0.0008	0.0008	0.0030	0.0034
10	0.0008	0.0008	0.0006	0.0010	0.0026	0.0032	0.0031	0.0036	0.0008	0.0008	0.0029	0.0034
11	0.0007	0.0009	0.0005	0.0011	0.0026	0.0032	0.0030	0.0036	0.0008	0.0008	0.0029	0.0033
12	0.0006	0.0009	0.0004	0.0011	0.0025	0.0033	0.0029	0.0037	0.0008	0.0008	0.0029	0.0033
13	0.0006	0.0009	0.0004	0.0012	0.0024	0.0033	0.0029	0.0037	0.0008	0.0008	0.0029	0.0033
14	0.0008	0.0009	0.0005	0.0011	0.0024	0.0033	0.0030	0.0037	0.0009	0.0008	0.0029	0.0034
15	0.0008	0.0008	0.0006	0.0010	0.0025	0.0033	0.0030	0.0037	0.0008	0.0008	0.0029	0.0034
16	0.0009	0.0008	0.0007	0.0010	0.0025	0.0033	0.0031	0.0037	0.0009	0.0009	0.0029	0.0034
17	0.0009	0.0008	0.0007	0.0011	0.0026	0.0033	0.0032	0.0037	0.0009	0.0009	0.0030	0.0035
18	0.0010	0.0008	0.0008	0.0011	0.0026	0.0033	0.0033	0.0037	0.0009	0.0010	0.0030	0.0035
19	0.0009	0.0008	0.0007	0.0011	0.0026	0.0033	0.0032	0.0037	0.0009	0.0009	0.0030	0.0035
20	0.0008	0.0008	0.0006	0.0011	0.0025	0.0033	0.0031	0.0037	0.0008	0.0009	0.0029	0.0034
21	0.0008	0.0008	0.0006	0.0011	0.0025	0.0033	0.0031	0.0037	0.0008	0.0009	0.0029	0.0034
22	0.0008	0.0008	0.0006	0.0011	0.0025	0.0033	0.0031	0.0036	0.0008	0.0009	0.0029	0.0034
23	0.0008	0.0008	0.0007	0.0010	0.0025	0.0033	0.0031	0.0035	0.0008	0.0009	0.0029	0.0033
24	0.0008	0.0008	0.0007	0.0011	0.0025	0.0034	0.0031	0.0037	0.0008	0.0009	0.0030	0.0034
25	0.0008	0.0008	0.0007	0.0011	0.0025	0.0033	0.0031	0.0037	0.0008	0.0009	0.0029	0.0034
26	0.0008	0.0008	0.0007	0.0011	0.0025	0.0033	0.0031	0.0037	0.0008	0.0009	0.0029	0.0034
27	0.0008	0.0008	0.0007	0.0011	0.0025	0.0033	0.0031	0.0037	0.0008	0.0009	0.0029	0.0034
28	0.0008	0.0008	0.0006	0.0010	0.0025	0.0033	0.0031	0.0036	0.0008	0.0008	0.0029	0.0034

Mix Code:

M X X - 2

Indicates Mortar Bar

Cement Type:
- OO-Type I
- OM-Type I with MS
- KK-Type K
- KM-Type K with MS

Test Set number:
- Blank indicates Test Set #1
- "-2" Indicates Test Set #2

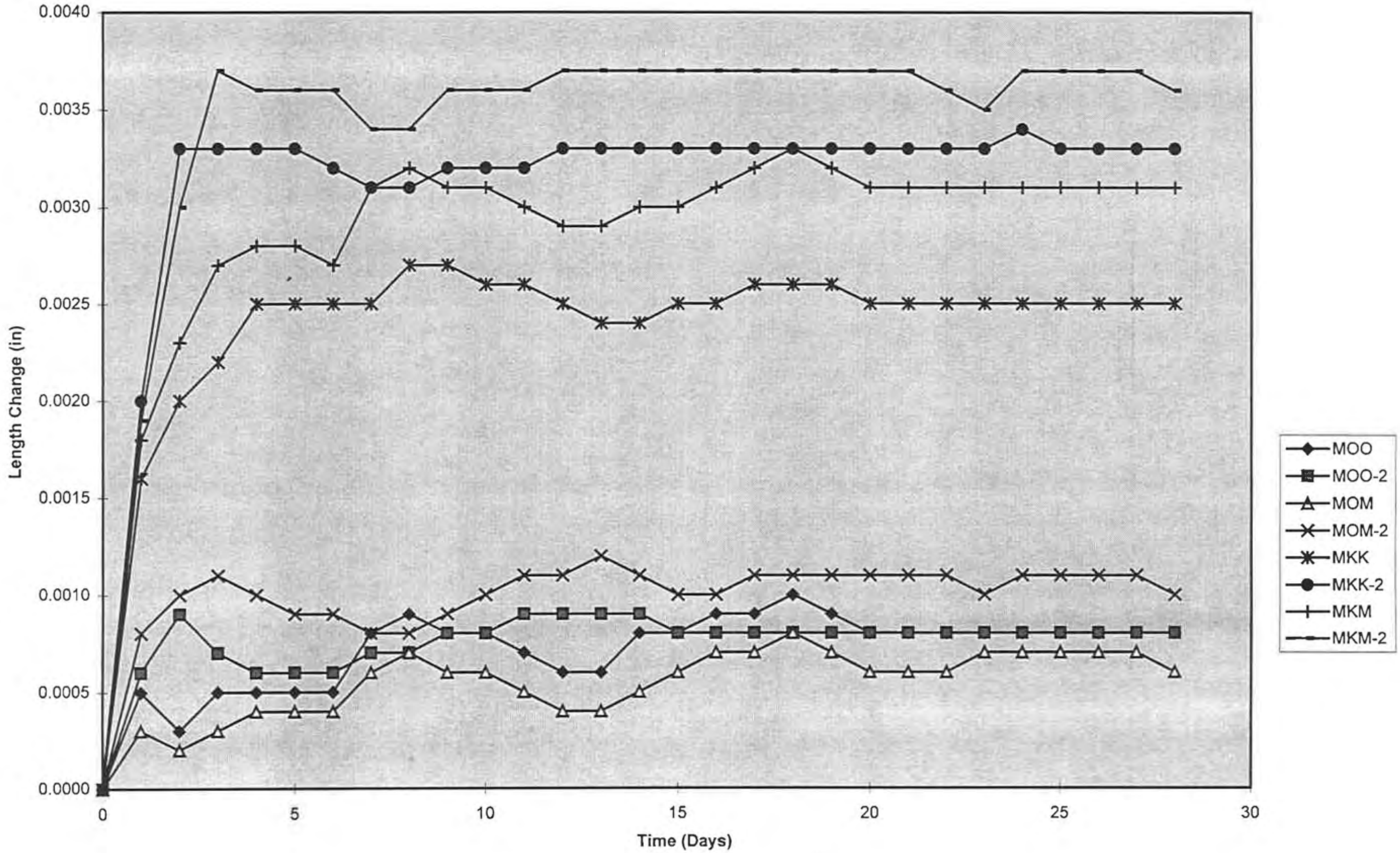


Fig 6.16. Mortar Length Bar Analysis

Table 6.10 Mortar Length Bar Statistics

	1	2	3	4	5	6	7	8
	MOO	**MOO-2**	**MOM**	**MOM-2**	**MKK**	**MKK-2**	**MKM**	**MKM-2**
Sum	0.0177	0.0178	0.0137	0.0230	0.0557	0.0720	0.0680	0.0801
Average	0.0008	0.0008	0.0006	0.0010	0.0025	0.0033	0.0031	0.0036
Mode	0.0008	0.0008	0.0007	0.0011	0.0025	0.0033	0.0031	0.0037
Std. Dev.	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001
Maximum	0.0010	0.0009	0.0008	0.0012	0.0027	0.0034	0.0033	0.0037
Minimum	0.0006	0.0007	0.0004	0.0008	0.0024	0.0031	0.0029	0.0034
Range	0.0004	0.0002	0.0004	0.0004	0.0003	0.0003	0.0004	0.0003

Expansion Arranged by Average Value

Greatest	----------	----------	----------	----------	----------	----------	----------	Smallest
	8	6	7	5	4	2	1	3
	MKM-2	**MKK-2**	**MKM**	**MKK**	**MOM-2**	**MOO-2**	**MOO**	**MOM**
Sum	0.0801	0.0720	0.0680	0.0557	0.0230	0.0178	0.0177	0.0137
Average	0.0036	0.0033	0.0031	0.0025	0.0010	0.0008	0.0008	0.0006
Mode	0.0037	0.0033	0.0031	0.0025	0.0011	0.0008	0.0008	0.0007
Std. Dev.	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001
Maximum	0.0037	0.0034	0.0033	0.0027	0.0012	0.0009	0.0010	0.0008
Minimum	0.0034	0.0031	0.0029	0.0024	0.0008	0.0007	0.0006	0.0004
Range	0.0003	0.0003	0.0004	0.0003	0.0004	0.0002	0.0004	0.0004

Mix Code:

M X X - 2

Indicates Mortar Bar

Cement Type:
OO-Type I
OM-Type I with MS
KK-Type K
KM-Type K with MS

Test Set number:
Blank indicates Test Set #1
"-2" Indicates Test Set #2

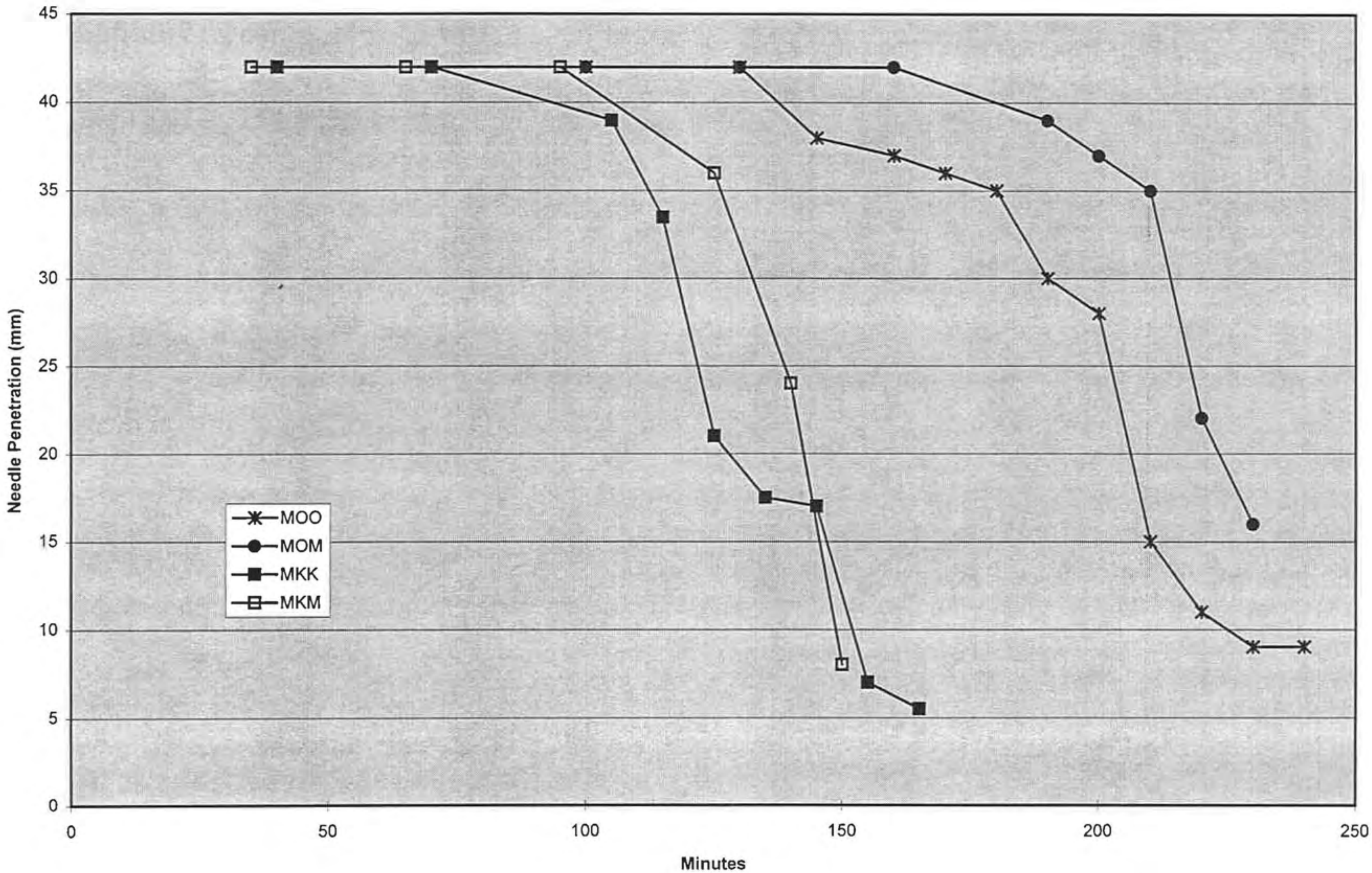
6.7 Time of Set Data/Results

Results for the time of set tests were recorded for each mortar type. Table 6.11 and Figure 6.17 present the Vicat Time of Set data for each of the four mixes tested. It should be noted that the early needle penetrations of 42 mm is the maximum possible "bottoming-out" value, and elapsed time at 10 mm penetration is defined as the Time of Set.

Table 6.12 shows each mortar with the amount of sand and water used for proper consistency and the time corresponding to a Vicat needle penetration of 10 mm, i.e., the Time of Set. The Time of Set for each mix is also presented in bar chart form in Figure 6.18. From observation of these data, the Type K mortars hardened significantly faster than the Type I mortars. Time of set for Type K mortars was nearly 100 minutes less than Type I mortars.

Table 6.11 Vicat Time of Set[1]				
	Needle Penetration[2]			
Time (min)	MOO	MOM	MKK	MKM
0				
30				
35				42
40	42	42	42	
45				
50				
55				
60				
65				42
70	42	42	42	
75				
80				
85				
90				
95				42
100	42	42		
105			39	
110				
115			33.5	
120				
125			21	36
130	42	42		
135			17.5	
140				24
145	38		19	
150				8
155			7	
160	37	42		
165			5.5	
170	36			
175				
180	35			
185				
190	30	39		
195				
200	28	37		
205				
210	15	35		
215				
220	11	22		
225				
230	9	16		
235				
240		9		

[1] ASTM C807-89
[2] Max Penetration is 42mm



6.17. Vicat Time of Set Data

Table 6.12. Vicat Data Summary

	Time	Sand	Cement	Microsilica	Water
Mix	min	g	g	g	ml
OO	225	1720.0	750.0		375.0
OM	238	1545.0	675.0	75.0	375.0
KK	143	1700.0	750.0		375.0
KM	148	1321.8	675.0	75.0	375.0

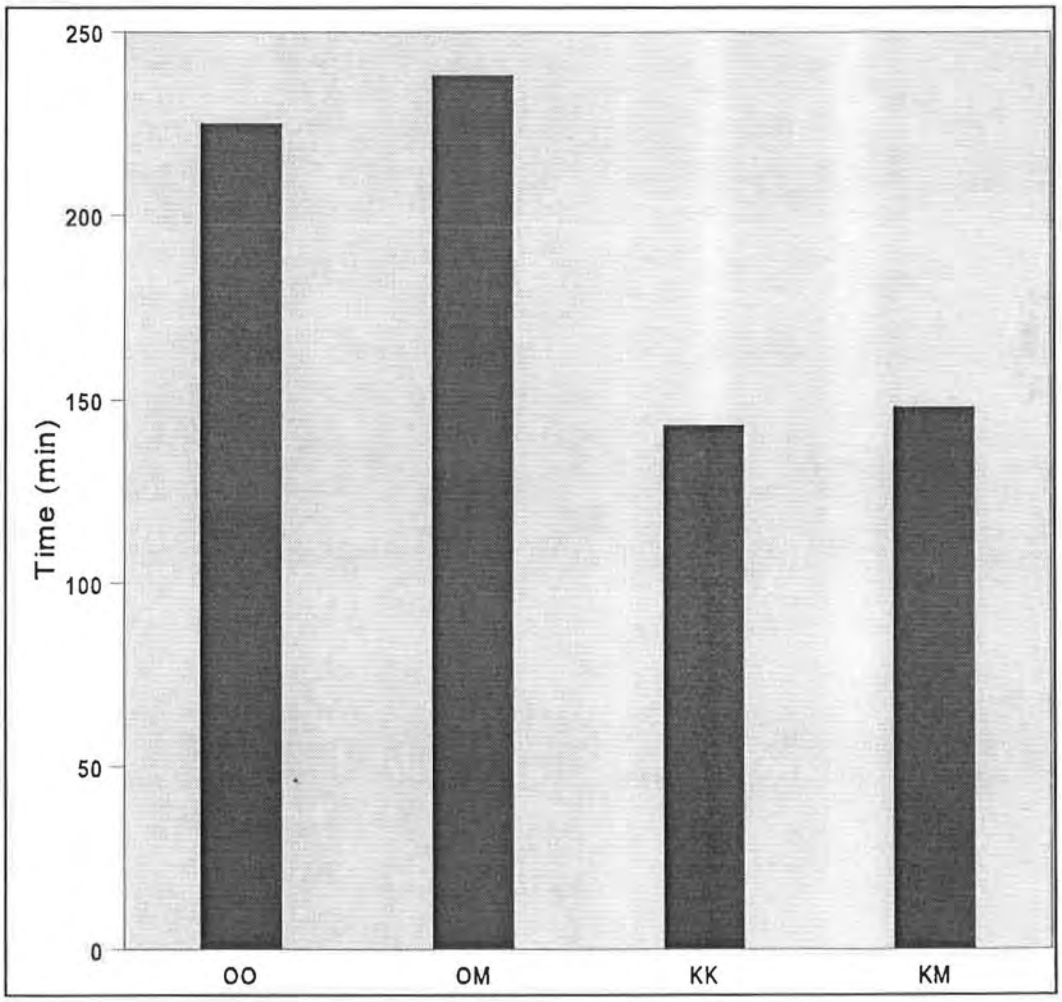


Fig 6.18. Time of Set Summary

7. RESULTS TOWARD EMPLACEMENT OF FIELD TEST DECKS

7.1 General

The last objective of this work was to try to plan and coordinate the emplacement of some SCC decks in a test/verification/demonstration scenario on some bridges in the ALDOT system. Toward this end, the PIs met with the Research and Development, Bridge, Maintenance, Construction, and Materials and Tests Bureau Chiefs in Montgomery in June, 1995 to discuss the ALDOT's continued interest in the field testing and limiting constraints and other details. The group indicated an interest in going forward to the field testing phase, and Mr. Stan Cauthen suggested the use of the Keller Bridge in Decatur, which was being rehabilitated at the time, as a good test site. Additionally, the group expressed an interest in testing a granulated blast furnace slag concrete mix as well as looking at other curing conditions.

After our initial meeting, Dr. Ramey coordinated the details of the Keller Bridge test site with Mr. Cauthen, and prepared a research proposal for the Phase II - Field Testing Program. This proposal was submitted to the ALDOT in August, 1995. Subsequently, the ALDOT rejected this proposal, and indicated that they desired to expand the scope of the work to include testing of deck design and construction changes are well as deck material changes.

In January, 1996 the PIs submitted a new Phase II - Field Testing Program proposal which was much broader in scope than the earlier proposal. In May 1996, the ALDOT decided that field testing phase was not necessary or cost effective, and thus they rejected the Phase II proposal.

In July, 1996 the PIs submitted a new Phase II - Field Testing Program proposal to the Auburn University Highway Research Center (HRC). This proposal was much smaller in scope and cost than the earlier field testing proposals. This proposal was rejected in September 1996 due to lack of interest/support by the HRC Research Advisory Board.

The ALDOT remains interested in comparing the performance of a SCC bridge deck with one of their standard PC decks. However, they feel that the concrete industry, and more specifically the manufactures of Type K cement, should bear the burden of demonstrating that Type K cement and SCC is an effective and viable solution to Alabama's early deck cracking problems. In the future, the PIs plan on working with representatives of the concrete industry, a Type K manufacturer, a bridge contractor, and the ALDOT to try to have a demonstration test bridge deck of SCC emplaced along with one of ALDOT's standard PC concrete mix for control.

For completeness and for potential future use, the body and essence of the PI's three Phase II - Field Testing Program proposals are included in the following three sections.

7.2 First Proposed Field Testing Program

The research objectives and work plan for the first field testing program proposed to the ALDOT in August, 1995 are given below.

Objectives

The long term objective of this proposed project is to assess the practicality and viability of using Type K shrinkage-compensating concrete and other special concrete mixes to increase the service life/durability of bridge decks in Alabama. To achieve this long term objective, specific subobjectives of the project are as follows:

1. Use the Keller bridge deck rehabilitation project in Decatur, Alabama in part as a field test program to assess the abilities of SCC and other special concrete mixes and curing procedures in mitigating shrinkage and other early nonstructural cracking, and later in mitigating traffic induced cracking of bridge decks.

2. Record cost parameters and constructability/construction issues for the various deck concrete mixes during construction of the bridge decks.

3. Monitor early (1-day) through long-term (1-year) deck longitudinal and transverse dimensional changes and deck top surface cracking along with bottom surface cracking at select locations.

4. Based on results from (2) and (3) above make recommendations to ALDOT regarding the use of SCC and other special concrete mixes to mitigate cracking and enhance the service life of bridge decks in Alabama.

5. Based on results from (2) and (3) above make recommendations to ALDOT regarding future concrete deck mix designs, and structural design changes to test in a more fully instrumented field deck testing program.

Work Plan

Service life/durability performance assessment by direct means is not reasonable from a time standpoint. Hence, material compressive and tensile strengths, modulus of elasticity, rapid chloride ion penetration (RCP) value, air content, slump, w/c ratio, density, dimensional stability, rate of loss of slump, and set time will be monitored for the concrete mixes employed in this study. Deck top surface crack developments and dimensional changes in the longitudinal and transverse directions will be monitored for the field test decks. These will be used, along with engineering judgement, to predict service life/durability enhancement or nonenhancement of the SC and other special concrete bridge decks relative to ordinary PC concrete decks.

Detailed descriptions of the

- Test Bridge Requirements
- Test Bridge Construction and Instrumentation Requirements
- Test Bridge Performance Monitoring Requirements

in the deck field testing program are presented below. Implementation of this program constitutes the work plan for this project.

REQUIREMENTS FOR CONCRETE BRIDGE DECK FIELD TESTING PROGRAM

I. Test Bridge Requirements

Initially, special site and structural conditions were identified for a field test bridge to allow direct comparisons of deck behaviors using various concrete mix designs and curing methods. After conferring with ALDOT personnel it appeared that

(1) the number of spans required, and

(2) the timing schedule for the project

might pose significant problems. However, the ALDOT indicated that they were just beginning a deck rehabilitation project on the Keller Bridge in Decatur, Alabama. This bridge and its rehabilitation appear to be very well suited to the purpose of this investigation. Hence, the Project Work Plan has been prepared based on using the Keller Bridge.

1. Site Layout

   a. The geometrical and deck structural support system layouts for the Keller Bridge are as shown in Figure 7.1, and the proposed test decks locations and materials are identified in Figure 7.2.

   b. The seven 34' long simple span decks supported on AASHTO girders (see Figure 7.2) render all test deck materials with identical span length, width, support girders, support bearings, and foundations. This in turn will allow direct comparisons of the test deck performances.

   c. The 34' long simple span decks will allow comparison of an ALDOT standard PC mix with normal curing practices, and a standard PC mix with CONFILM (if needed) and 2 layers of continuous wet burlap curing.

   d. The three (3) continuous spans supported on steel plate girders (see Figure 7.2) decked with SCC will provide an excellent comparison situation with the almost equal span lengths of the four (4) continuous spans supported on steel plate girders (see Figure 7.2) decked with standard PC concrete.

2. Deck Design Requirement

   a. Use same f'_c (except for PC control mix and for HPC mix) in all test deck designs.

   b. Have each test deck be identical in grade steel (Grade 60), in reinforcing and in cover.

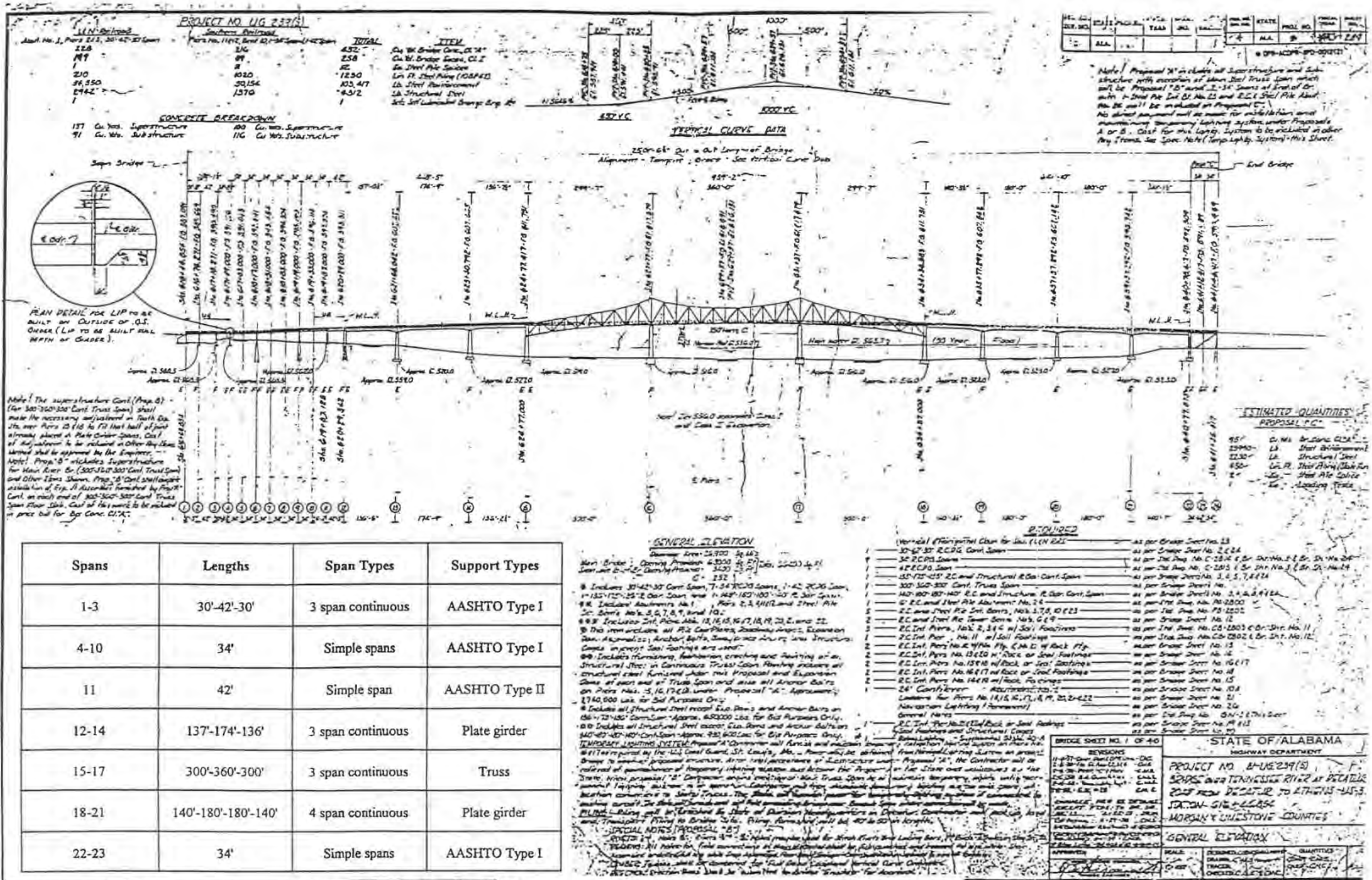
   c. Use same armor joint design for each test deck.

   d. Use same guardrail and/or curb design for each test span.

3. Test Deck Concretes

   a. ALDOT normal PC deck concrete (for control). Use $f'_c = 4000$ psi concrete.

   b. Shrinkage compensating concrete (SCC). Use $f'_c = 4500$ psi concrete. ACI Committee 345 Report on "Guide for Concrete Highway Bridge Deck Construction" indicates that SCC can significantly reduce deck shrinkage cracking, has higher flexural tensile strength and significantly higher abrasion resistance relative to PC concrete. Various researchers have found improved tensile strength, greater tensile strain at failure, and somewhat reduced modulus of elasticity for SCC relative to PC concrete. Reduced drying shrinkage cracking and improved tensile characteristics should result



Spans	Lengths	Span Types	Support Types
1-3	30'-42'-30'	3 span continuous	AASHTO Type I
4-10	34'	Simple spans	AASHTO Type I
11	42'	Simple span	AASHTO Type II
12-14	137'-174'-136'	3 span continuous	Plate girder
15-17	300'-360'-300'	3 span continuous	Truss
18-21	140'-180'-180'-140'	4 span continuous	Plate girder
22-23	34'	Simple spans	AASHTO Type I

Figure 7.1 Keller Bridge Geometrical and Structural Layout

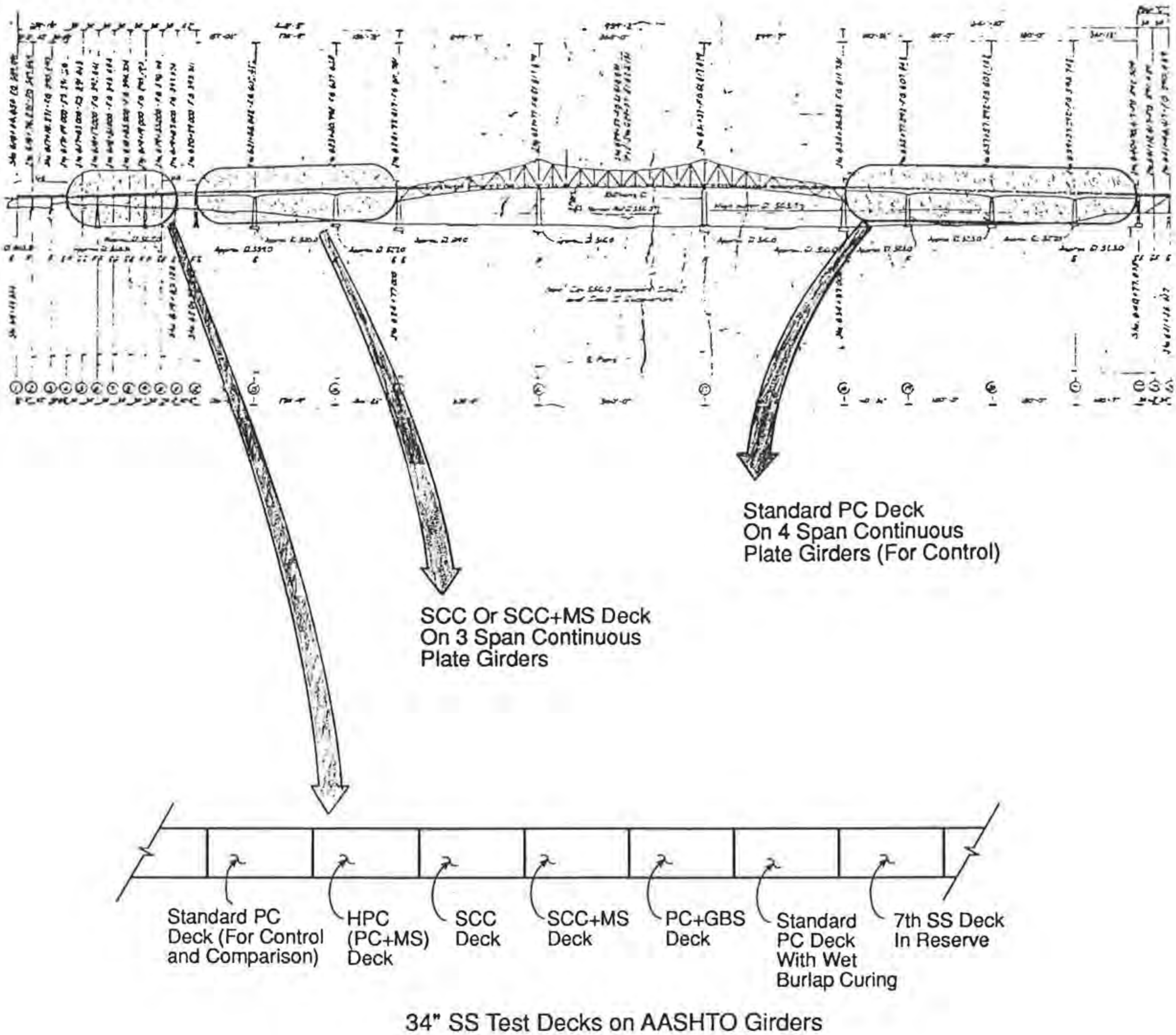


Figure 7.2. Proposed Test Deck Materials and Locations

in a concrete with reduced macro and micro cracking and probably superior fatigue resistance. In turn, this should result in a more durable concrete.

c. High performance concrete (HPC or PC + MS). Use $f'_c = 6000$ psi concrete. HPC has improved strength (static and fatigue) and impermeability relative to PC concrete. In turn, this should improve durability. However, its level of cracking in flexural members such as a deck, relative to PC concrete, is unknown.

d. Shrinkage compensating concrete with microsilica (SCC + MS). Use $f'_c =$ 4500 psi concrete. Use SCC to improve crack performance and MS to improve impermeability and fatigue performances. In turn, this should improve durability.

e. PC concrete with ground blast furnace slag (PC + GBS). Use $f'_c = 4500$ psi concrete. Ground blast furnace slag concrete has improved impermeability and hardness and these should improve durability. There is now a manufacturer of GBS in New Orleans and hence its availability is now good and its cost is reasonable.

4. <u>Other Material Requirements</u>

a. Use same aggregates, PC cement type and supplier, SCC cement supplier, and fly ash other admixture suppliers for all test decks. The contractor shall submit sufficient materials (aggregate, cement, admixtures) as per the ALDOT specifications, but not less than 60 days prior to construction to allow development of mixture proportions.

b. Use same ready-mix concrete supplier for all test decks.

c. Use normal ALDOT specifications and acceptance requirements for PC control deck.

d. Require mix designs with projected high durability characteristics and f'_c equal to 4500 psi for test deck concretes (except HPC and PC control mixes).

e. Require CONFILM, or equivalent, to reduce evaporation after finishing and before curing if recommended by concrete manufacturers (cement company). Its use should be decided on for each of the 5 test materials.

5. <u>Construction Requirements</u>

a. Have same contractor (and subcontractors) construct all test decks.

b. For simple spans, place each test deck in one placement.

c. For continuous spans, place each deck in the same or equivalent sequence.

d. Construct all test decks at the same time of year (same month) under approximately the same weather conditions, and at the same time of day.

e. Use same deck forming, construction sequence and procedure, etc.

f. Place, consolidate, and finish all test decks in an identical manner. If any of test deck concretes have a recommended method of placement, e.g., by pumping for SCC concrete, then placement by the preferred method should take precedence over requiring uniformity of placement for all.

g. Cure all decks (except PC control) with CONFILM (if appropriate) and cover with 2 layers of wet burlap immediately after finishing (within 2 hours) and maintain wet curing for 10 days. Cure the PC control deck in the manner normally employed for ALDOT decks, i.e., spray-on membrane and cover with polyethylene sheeting for 10 days.

h. Place CONFILM or equivalent, if used, in the manner specified by the manufacturer.

i. Finish construction of bridge curb and/or guardrails in an identical manner (same construction, people, equipment, timing, procedure, etc) for each test span.

II. Test Bridge Construction and Instrumentation Requirements

Special performance monitoring sensors, practice placements, construction documentation, and material testing requirements which must be emplaced and/or performed during construction of the test bridge in order to achieve the objectives of the study are listed below:

1. <u>Test Deck Layout</u>

   Construct the test bridge decks using the 5 test concrete mixes as indicated in Figure 7.2.

2. <u>Concrete Monitoring/Testing During Construction</u>

   a. Concrete temperature

   b. Slump

   c. Air Content

   d. Cylinders for 3, 10, and 28 day compressive strength and splitting tensile strength tests

   e. Cylinders for modulus of elasticity and Poisson's ratio testing

f. 3" x 3" x 10" prisms for restrained and unrestrained shrinkage testing

g. 4" x 8" cylinders for rapid chloride permeability (RCP) testing

h. Concrete time of workability/set

3. Preconstruction Meeting and Trial Sections

Prior to bridge deck construction, a preconstruction meeting should be held to discuss all aspects of the bridge deck construction. Parties at this meeting should be the contractor, concrete foreman, concrete supplier, test lab, inspector and project manager. Cement and admixture reps should also attend if possible. The meeting should conclude with a discussion of the preconstruction trial sections which should be built 1 or 2 days prior to each test deck placement.

The contractor should construct a small (2 cubic yard) trial section prior to placing the actual bridge deck for each of the different deck materials being tested except for the normal PC mixture. The purpose of this exercise is to familiarize the concrete supplier, contractor, finishers, testing lab, inspectors, etc. of what to expect with each concrete mixture. The 2 yard quantities should be batched, delivered, placed into a dummy form, consolidated, finished, and initial curing applied (including wet burlap) before the first deck placement is made. The trial section for each test material can provide concrete for testing to assure the earlier accepted mix design can be delivered to the site and properly emplaced. The testing to be performed on the trial section concrete are the same as those listed in II (2) above. Placement of the actual bridge deck should not commence until all parties are satisfied with the performance of the contractor and the fresh concrete.

4. Construction Documentation

Monitor and record (via written descriptions) all significant deck construction activities during the construction phase of the test bridge. This should include appropriate activities before, during, and after concrete placement, performance sensors being emplaced, material QC/QA activities, etc. It should include preparing a time-log of significant activities from the beginning of the test bridge emplacement through opening of the bridge to traffic. Date and time of day of pouring of each deck should be recorded along with the weather conditions (T, RH and wind speed). It should also include developing a test deck cost table which reflects the in-place cost of each of the different material decks. The table should allow comparisons of initial cost (with the cost of special test sensor emplacements not included) of the various deck test materials. It should also allow estimates of the required years of additional service life that would need to be achieved for the higher initial cost deck materials (SCC, HPC, and SCC + MS) to prove to be cost effective.

5. <u>Deck Placement and Consolidation Quality</u>

Removal of approximately 5% of the SIP forms will be made at select locations for each test deck to visually check for quality of concrete placement and consolidation. These SIP form removals will be made approximately 1 month after construction.

6. <u>Deck Length Change Gage Requirements</u>

Emplace length change gage plugs in the top of deck as indicated in Figure 7.3 in order to monitor longitudinal and transverse length changes over a prolonged period of time (1 year by AU researchers and later during biennial inspections by ALDOT bridge inspectors). The deck gage plugs will be emplaced by the AU research team after placement of the deck.

III. Test Bridge Performance Monitoring Requirements

Performance monitoring and evaluations of the test bridge which will be performed in order to achieve the objectives of the study are listed below.

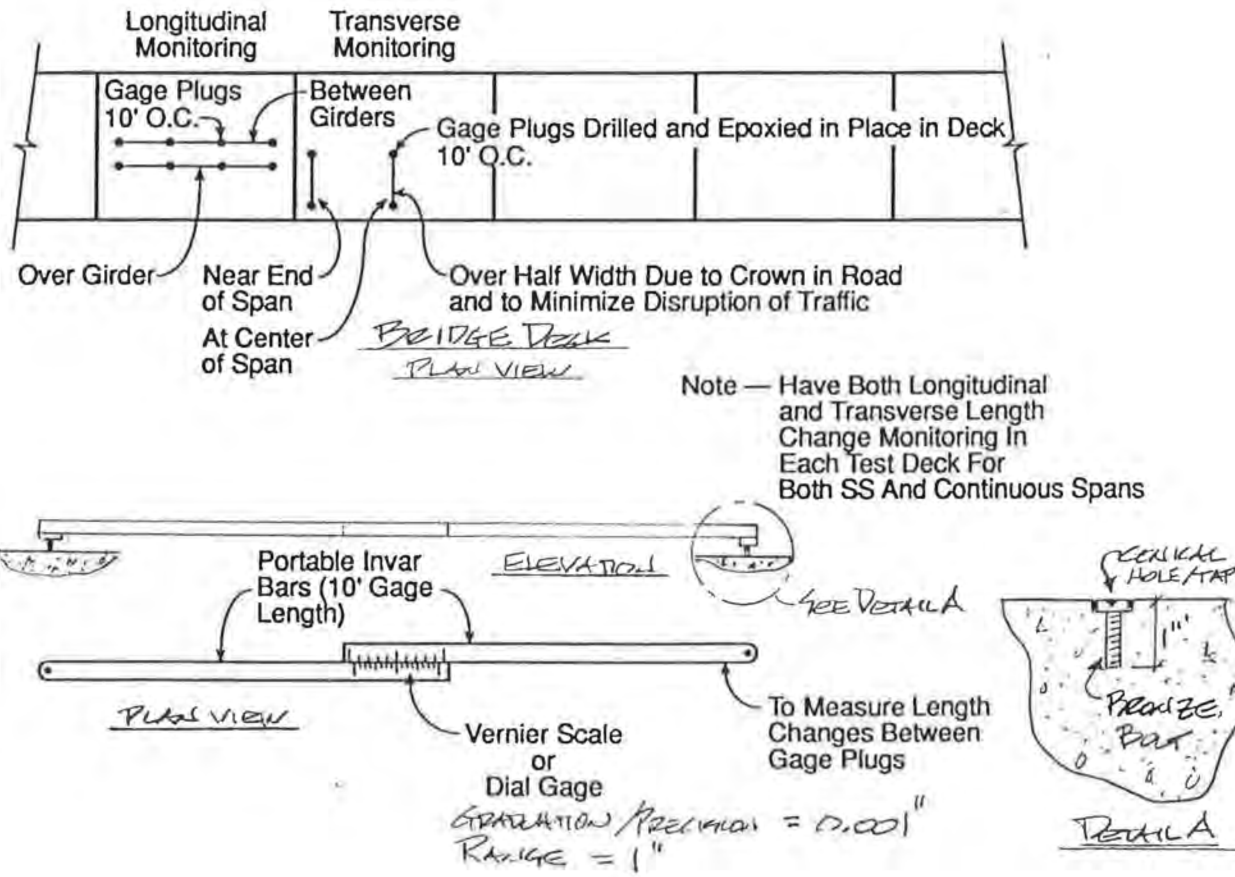


Figure 7.3. Deck Length Change Monitoring and Instrumentation

1. Deck Crack Monitoring (Visual)

Chronological records of deck cracking (top surface) will be developed commencing with early cracking (1-day) through 1-year. Formal visual inspections for cracking will be made at the following ages:

1-*day*	Limited to select area inspections due to limited access due to wet curing burlap and forms
2-*day*	
5-*day*	
10-*day*	
30-*day*	
180-*day*	
365-*day*	

See (5) below for long-term (beyond 1 year) monitoring of deck top surface cracking during biennial inspections.

2. Deck Length/Width Change Monitoring

Deck length and width changes will be monitored (see Figure 4) for a year following the time schedule below.

1,2,3,4,5,6,7-day
10-day
15-day
20-day
30-day
60-day
180-day
365-day

Length change measurements will be made at the same time of day (in the early morning when temperatures are lower and more stable). Also a few length change measurements will be made during the day (at 4-hour intervals) to monitor changes due to temperature changes. These data will be of interest in their own right, and will be helpful in separating creep and shrinkage length changes from thermal changes in the 1-day through 365- day length change data. See (5) below for long-term (beyond 1 year) monitoring of deck length/width changes during biennial inspections.

3. Deck Corings

The decks will be cored at select locations to assess the in-situ characteristics of the concretes. Cores will be taken by the ALDOT at 30 days and at 365 days with 1 replica core for each situation. These cores will be used to determine concrete compressive strength, RCP testing, and to perform visual inspections for microcracking, bonding with rebar and rebar corrosion. It should be noted that a sacrificial rebar will be added to each test deck at a predetermined location of some of the corings to allow cores to be taken which include a rebar.

4. Traffic Volume and Character

Conventional methods for determining traffic volume (ADT) and truck traffic (ADTT) will be employed sometime during the bridge's first year of operation by the ALDOT.

5. Long-Term Monitoring (Biennial Inspections)

Regular biennial inspections made by ALDOT of the test bridge will be expanded to include measurements of the bridge length and width changes (see III (2) above), and documentation of the top surface cracking (see III (1) above) as determined from visual inspection. Copies of these data will be forwarded to the project researchers at Auburn in order to maintain a record of the long-term bridge deck performances.

Later, decisions regarding the use of the deck materials tested will be revisited if the long-term performance data so indicates.

Results from execution of the Concrete Bridge Deck Field Testing Program will be analyzed and synthesized, and recommendations will be made regarding concrete mixes appropriate for the enhancement of bridge deck durability in Alabama. The results and recommendations from this study can be immediately implemented by the ALDOT in their new bridge and old bridge rehabilitation designs.

Also, it is anticipated that attractive deck concrete mixture(s) evolving from this study will be further tested for constructability issues, development of a construction training video tape, and assessment of deck performance. It is further anticipated that this follow-up testing will include some structural design changes such as increased deck thickness, rearrangement of rebars, and additional distribution/temperature/shrinkage steel in the longitudinal direction. It is envisioned that this future testing will have fewer test spans (than the project proposed here), but the spans will be more heavily instrumented to monitor deck temperature changes, strains/stresses, dimensional changes, and strains/stresses developed under truck load tests, in addition to short-term nonstructural cracking and longer-term structural cracking.

7.3 Second Proposed Field Testing Program

The research objectives and work plan for the second field testing program proposed to the ALDOT in January, 1996 are given below.

Objectives

The objective of the proposed project is to mitigate premature cracking in concrete bridge decks and in-turn improve their durability/service life. To achieve this objective, specific subobjectives of the project are as follows:

1. Determine the primary types and causes of early (1-day to 1-year) cracking of bridge decks in Alabama.
2. Determine ALDOT's present structural design, materials and construction requirements regarding bridge decks.
3. Identify and recommend changes in ALDOT's structural design, material, and construction requirements as appropriate to mitigate the premature cracking of bridge decks.

Work Plan

The work plan to accomplish the above objectives is outlined below.

1. Continue to review the literature and discuss deck cracking with ALDOT personnel and personnel of other state DOTs to determine the primary causes of premature deck cracking in the U.S. and in Alabama in particular. Secure and review copies of recently completed studies sponsored by the ALDOT and Penn DOT on bridge deck cracking.

2. Conduct a comprehensive and detailed review of ALDOT's present structural design, materials, and construction requirements regarding bridge decks.

3. Synthesize the results of (1) and (2) above to identify potentially fruitful changes which would reduce early cracking and enhance service life of bridge decks.

4. Develop a comprehensive testing program to assess the merits of the changes identified in (3) above. This testing program will consist of 3 major components, i.e.,

   - A materials laboratory component to assess basic concrete properties and characteristics to, (1) allow later correlation with the structural laboratory and field testing results, and (2) make later decisions on what should be the concrete materials requirements for mix design acceptance and acceptance of in-place concrete.

   - A structural laboratory component to serve as a screening program to evaluate the merits of structural design, materials, and construction changes identified in (3) above. This program will primarily consist of laboratory monitoring of simulated concrete deck segments to asses the effects of various parameters (structural design, materials, and construction) on early deck cracking. It is envisioned that the parameters having the greatest impact on early cracking, and the best corrective measures to mitigate this cracking, will be able to be identified from this laboratory program. The program will not attempt to simulate actual construction and environmental conditions (except in a few special cases), existing in the field. Rather, the uniform environmental conditions of the laboratory will allow isolation of each of the parameters being evaluated to assess its affect (and only its affect) on early cracking of a simulated deck panel. This can not be achieved in field testing and is one of the advantages of laboratory testing.

   - A field testing program to assess and/or verify changes (structural design, materials, and construction) coming forth from the two laboratory components discussed above. In some cases, field testing will be the only realistic means of assessing a change, while in other cases it will serve more as demonstration/verification testing. The field program will consist of constructing and monitoring the early (1-day to 1-year) cracking performances for two new bridge decks which incorporate various combinations of the tentatively identified changes (along with a portion using current requirements as the control). One of these bridges will be constructed during the summer work period (approximately July) and the other during cold weather (approximately January) to provide extremes on construction environmental conditions.

Each of the above three testing programs is discussed later in greater detail.

5. Based on the results from (1)-(4) above, finalize the assessment of effective and appropriate structural design, materials, and construction changes to mitigate premature deck cracking, and make specific and implementable recommendations to ALDOT regarding these changes.

After careful consideration the most common types and sources of deck deteriorations in Alabama, along with the parameters identified in ALDOT's research solicitation brief, it was decided to focus this research on the following potential corrective parameters.

Bridge Design:	Deck thickness
	Rebar arrangement
	Additional shrinkage/temperature/distribution steel
Materials:	Concrete Mixes
	Concrete Aggregate
	Concrete Curing
Construction:	Environmental conditions (temperature, humidity and wind)
	SIP metal forms vs. stripped wood forms
	Placement sequence

The effect of each parameter on deck cracking can best be assessed by some combination of

- laboratory materials testing
- laboratory structural testing
- field bridge testing

In some cases, all 3 types of testing and assessment will be appropriate, while in others only one of the 3 is necessary and/or appropriate. Also, only those materials which perform satisfactorily in the materials testing will go on to the structural testing, and only those performing satisfactorily in the structural testing will go forward to the field testing. Each of the 9 corrective parameters/actions and their assessment approach are discussed below.

Deck Thickness. Many states have gone to increased deck thicknesses to stiffen their bridges in order to mitigate deck cracking. An increase in thickness by 1", or to a uniform value of 8" or to a value based on equal concrete stresses or deck deflections are all viable alternatives. The relative merits of each of these alternatives will be assessed based on analytical and construction considerations and an appropriate recommendation made on deck thickness changes. The recommended change will be tested via laboratory structural testing of a simulated bridge deck segment and by field bridge testing and compared with decks of current thickness. It is expected that this corrective action will significantly improve structural and fatigue cracking performances.

Rebar Arrangement. Because transverse cracking of decks vertically above transverse rebar is the most common type of deck cracking, various groups have proposed reversing the arrangement of deck rebar to have the longitudinal steel on the outside and the transverse steel on the inside. It should be noted that whereas increasing the deck thickness is not necessary to accomplish this reversal of rebar, the two actions would be a good combination. Assessment of the benefits of this action will be made by laboratory structural testing and by field bridge testing.

Additional Longitudinal Steel. Because most deck cracking is transverse and the purpose of longitudinal shrinkage/temperature and distribution steel is for crack control, it seems appropriate to increase the percentage of this steel for better crack control. A minimum p of 0.002 for both top and bottom longitudinal steel, or #4 bars at a maximum spacing of 12", should provide better transverse crack control. The exact value of the increased longitudinal steel will be finalized after the literature review, and its benefit will be assessed by laboratory testing and by field bridge testing.

Concrete Mixes. Candidate mixes for improving the early cracking performance and/or later structural crack performance of bridge decks are

- mix using Type II (AASHTO Type II) cement as a replacement for Type I cement

- mix using ground blast furnace slag as a partial replacement for Portland cement
- mix using shrinkage compensating cement and microsilica as the cementions material
- mix using less water and lower w/c ratio via use of water reducers and a higher strength ($f'_c = 4500$ psi)

Each of these mixes will be designed/developed using ALDOT's present specification/requirements for bridge concrete with the assistance of Mr. Serigo Rodriguez of ALDOT's Materials and Tests Bureau. The mixes will be assessed for volume stability/shrinkage characteristics, strength and other traditional quality control tests (see Laboratory Materials Testing Program section for listing of tests to be conducted).

The mixes which perform well and show promise for improvement of early cracking performance or reduced structural cracking performance will be tested as simulated deck panels in the laboratory structural testing program to assess early cracking performances. In turn, those which perform well will be field tested.

Concrete Aggregate. Since volume stability and thermal expansion characteristics of concrete mixes are primarily controlled by the mix aggregates, and since volume stability and thermal characteristics are primary parameters affecting the early cracking of bridge decks, it is important to look at concrete aggregates. Based on past studies and the experience of ALDOT materials engineers, course aggregate characteristics which have a significant affect on deck cracking and structural fatigue and durability problems are

- Crushed aggregate vs. rounded river gravel
- Chert bearing aggregate vs. nonchert bearing gravel
- Well graded vs. poorly graded aggregate deck (perhaps PC concrete should have an aggregate gradation requirement similar to asphalted concrete).

ALDOT's present bridge concrete mix will be used with an appropriate sample of each type or gradation aggregate above and tested as indicated in the Concrete Mix section above. Additionally, upon completion of the 30-day unrestrained expansion/shrinkage bar testing, these same bars will be tested for expansion/contraction due to alternate wetting and drying and later due to heating and cooling. Those aggregates exhibiting performances superior to those currently used will be tested for early cracking in the structural laboratory simulated deck panel set-up.

Concrete Curing. Everyone acknowledges the paramount importance of quality curing to achieving quality concrete. However, the quality of the curing required is strongly related to the exposure environment in which the fresh concrete is placed, e.g., the curing requirements on a windy and hot July day are much greater than they would be on a damp-calm-cool day in March. Four curing conditions under two environmental exposures will be assessed in the structural laboratory simulated deck panel testing. These conditions and exposures are shown below

<u>Curing</u>

- Immediate wet curing for 10-days
- Delayed wet curing for 10-days (1-day delay)
- Immediate use of curing compound
- No curing

<u>Exposure</u>

- Normal indoor laboratory conditions
- Simulated hot weather (90°F) and windy (15 mph) conditions

Decisions regarding curing conditions to be tested in the field will be based on results of the above laboratory testing.

Environmental Conditions. Environmental conditions (ambient temperature, concrete temperature, humidity, and wind conditions) at the job site significantly affect concrete properties/characteristics, placement, finishing, and curing (both the initiation and type of curing required). Since field environmental conditions are hard to control, variances in environmental conditions will be investigated in the laboratory in the manner discussed in the Concrete Mixes and Concrete Curing sections. Also, since hot summer placement of concrete decks cause the greatest problems with early shrinkage and thermal cracking, these worst case conditions will be given special attention in the laboratory testing.

SIP Metal Forms. Economics and availability of carpenter craftsmen have driven bridge construction contractors to almost exclusive use of SIP metal forms for bridge decks. Unfortunately, there are a number of negative aspects of using these forms. Namely,

1. Was concrete properly consolidated, i.e., are there honeycombs on the bottom.
2. They add an extra 1" thickness of concrete deck DL.
3. In summer placements, the forms maybe very hot when fresh concrete is placed on them--does this affect the quality of the concrete.
4. In cold weather placements, the forms may be very cold when fresh concrete is placed on them--does this affect the quality of the concrete.
5. Form ribs in the transverse direction restrain deck concrete on the bottom from shrinking in the longitudinal direction when drying, and in-turn causes transverse cracking at the deck top.
6. Bottom concrete can not properly dry out and gain strength as it should.
7. They trap rain water penetrating through the deck which could lead to rebar corrosion and/or freeze-thaw type deterioration problems.

Item (1) above has probably been examined sufficiently by stripping SIP forms when they first began to be utilized and by random spot removals today. Items (3) and (4) will be assessed in the laboratory by heating and cooling SIP forms, and then placing concrete on the heated and cooled forms. Later, the specimens will be cored and the cores sliced into a top 2+", a middle 2+", and a bottom 2+" slice. Split-cylinder tensile testing will be conducted on the slices to assess concrete tensile strengths and thus assess damage to the concrete from the hot or cold SIP form. Item (5) will be assessed by comparing the cracking performance of simulated deck panel testing in the structural laboratory when panels are formed on SIP forms vs. oiled wood forms. Depending on these results, comparative field tests of SIP metal forms vs. removable wood forms may or may not be undertaken. Items (6) and (7) will be assessed by requesting that the ALDOT take corings (completely through the deck and metal form) at crack locations on some of its oldest and most heavily traveled bridges where SIP metal forms were used. These corings will be analyzed for evidence of trapped water and/or concrete damage or deterioration.

Placement Sequence. Improper sequencing in the placement of deck concrete can have the following detrimental effects.

1. Flexural loading and cracking of freshly placed and green concrete.
2. Excessive movement of rebars and breaking of bond in freshly placed and green concrete.
3. Requiring excessive movement of construction workers over and around freshly placed concrete and resulting in excessive movement of rebar and subsidence of concrete.
4. Not attaining the full thickness of deck nor rebar cover at the centerline of spans.

This parameter will be investigated in the field via looking at common concrete placement sequences and assessing the affect that these sequences have on early deck cracking and on constructed deck thickness/rebar cover at select key locations. Visual monitorings of deck cracking will be made and packometer measurements will be made to determine deck depth and rebar cover at select locations for each placement scheme. These field results will be correlated with predictions from theoretical analyses in order that analytical predictions can be used and trusted in the future when a contractor wants to deviate from a specified placement sequence.

As indicated earlier, each of the 9 corrective parameters/actions discussed above will be experimentally assessed in this investigation via

- materials laboratory testing
- structural laboratory testing
- field testing.

In general, the sequence of testing will be the same as the order above but some concurrent materials and structural laboratory testing will be appropriate. Table 7.1 gives a summary of the test parameters and program planned.

The complete testing program in each case cannot be finalized at this time, as the review and synthesis of damage causes, current ALDOT procedures, and assessments of potentially fruitful

Table 7.1 Project Testing Program Summary

Test Parameter	Testing Programs		
	Laboratory Materials Testing	Laboratory Structural Testing	Field Bridge Testing
Deck Thickness		X	X[b]
Rebar Arrangement		X	X[b]
Additional Longit. Steel		X	X[b]
Concrete Mixes	X	X[a]	X[b]
Concrete Aggregate	X	X[a]	X[b]
Concrete Curing		X	X[b]
Environmental Conditions	X	X[a]	
SIP Metal Forms		X	X[b]
Placement Sequence			X[b]

[a]Only the most promising values from the materials testing program.
[b]Only the most promising values from the structural testing program.

changes have not been completed. However, the testing program planned is sufficiently general and flexible to allow adjustments as necessary once the listing of primary corrective actions to consider is finalized during work items (1) - (3) in the Work Plan. Though the complete program of corrective action/parameter values have not yet been finalized, the experimental program to be conducted can be and is discussed in detail below.

Laboratory Materials Testing Program. The laboratory materials testing program will focus on measuring the drying shrinkage and thermal properties of various concrete mixtures, in addition to the traditional quality control tests such as compressive strength, slump, and air content. Since the drying shrinkage of concrete is influenced primarily by the paste volume and water content of the mixture, and the thermal properties are dominated by the aggregate type, these factors will be varied as much as possible within the typical range of values used in concrete bridge decks.

A base concrete mixture (to which other mixture will be compared) will be designed to conform to the Alabama DOT A-1c mixture requirements (i.e., minimum 4,000 psi compressive strength at 28 days, maximum 3.5 in. slump, 4 to 6 percent entrained air content, minimum cement content of 620 lbs/yd^3, maximum water/cement ratio of 0.44, No. 57 or 67 coarse aggregate grading), and will use a Type I cement (without flyash), natural grave., and a well-graded combined (coarse plus fine aggregate) gradation. The effects of water/cement ratio (mixes with and without water reducer), use of flyash, aggregate type (river gravel, crushed limestone, and chert), aggregate gradation (well-graded versus gap-graded), and type of cementitious material (Type I, Type II, Type K, and blast-furnace slag cement) on the drying shrinkage and coefficient of thermal expansion of the concrete would be measured.

The tests to be conducted on the concrete materials are as follows:

<u>Aggregates</u>

1. Gradation
2. Moisture Content
3. Absorption
4. Specific Gravity

<u>Concrete</u>

1. Slump
2. Time to Set
3. Air Content
4. Unit Weight
5. Temperature
6. Compressive Strength (3 and 28 days)
7. Splitting Tensile Strength (3 and 28 days)
8. Modulus of Elasticity and Poisson's Ratio (28 days)
9. Drying Shrinkage (to 28 days) (except Type K)
10. Restrained Expansion (to 28 days) (Type K only)
11. Coefficient of Thermal Expansion/Contraction (28 days)
12. Wet-dry expansion and contraction
13. Absorption

Laboratory Structural Testing Program. Twelve (12) simulated bridge deck panels (5' transverse x 10' longitudinal x 6.5" or 8" thick) will be constructed in on the floor of the structures laboratory in the Civil Engineering Department as indicated in Figure 7.4. Each of the panels will be placed on top of simulated steel bridge girders in a composite manner. Steel plates with shear lugs will be welded to existing embedded steel plates in the laboratory floor. These new plates will simulate the longitudinal in-plane constraints to the deck provided by the bridge girders and the

Figure 7.4. Laboratory Simulated Deck Testing Layout/Plans

embedded plates in the floor will simulate the in-plane, constraints offered by the girders and diaphragms in the transverse direction. Thus, the 5' transverse x 10' longitudinal deck panel will be significantly constrained from in-plane drying and thermal shrinkage much as actual decks are in the field.

The uniform environment of the laboratory will allow isolations of the affect of the various parameters to be investigate, i.e., there will be no varying environmental or construction conditions to contaminate the performance results of the various parameters. In a few cases, we may purposely create an adverse environment for 1 or 2 of the test panels to assess this parameter. However, the overall affect of the bridges' real exposure environment and construction practices such as concrete placement sequence, will need to be evaluated in the field portion of the investigation.

The simulated test deck panels do not lend themselves to strain gage monitoring because of their significant in-plane constraints. Each panel will have 2 thermocouples embedded at half the

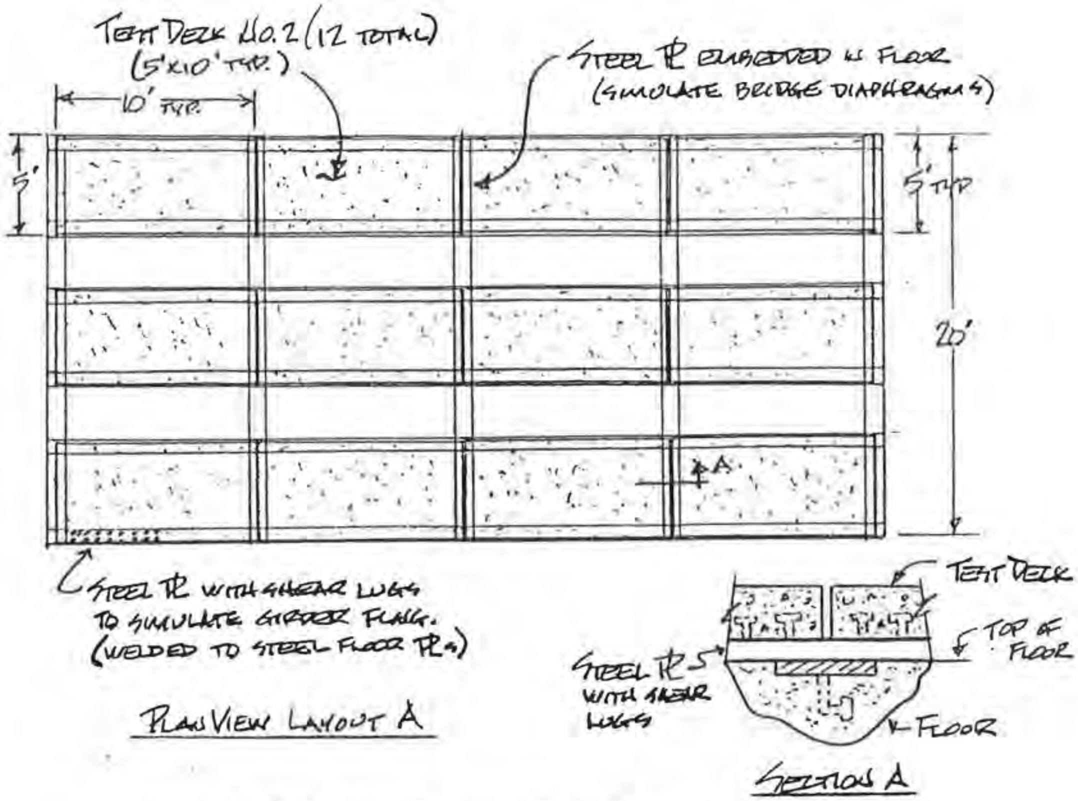


Figure 7.4. Laboratory Simulated Deck Testing Layout/Plans

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The simulated test deck panels do not lend themselves to strain gage monitoring because of their significant in-plane constraints. Each panel will have 2 thermocouples embedded at half the deck thickness near the center of the panel. Temperature readings from these thermocouples will be monitored along with cracking of the top surface of the panel. The cracking monitoring will be by visual inspection and assisted by wetting and drying if necessary.

It is expected that the simulated deck panels will serve as a good screening test to help identify parameters and/or parameter values which are not good at correcting early cracking problems in decks. In these cases, these parameters will not be carried forward to the field bridge testing program. Whereas, those parameters and/or values showing good early cracking mitigation performances will be carried forward to field testing in some manner.

It should be noted that a side benefit of the simulated deck panel testing is that it will provide an opportunity to evaluate (1) the ability of ready-mix companies to batch and deliver quality shrinkage compensation concrete and other noncommon concrete mixes and, (2) the sensitivity of some of the noncommon mixes to set-time, workability, and finishing. It is important that difficulties/problems which may be associated with the batching, deliver, placement, finishing, etc. of the various concrete mixed investigated be identified <u>before</u> we try to use them in the field in an actual bridge. A bridge construction contractor will be used to place, consolidate, and finish the concrete for each test panel in order that we can get actual contractor opinion and input on the above.

It is recognized that because of size limitations and the mild laboratory environment, that the simulated deck panels may not crack and thus not provide any useful data. Because of this concern, a few panels (worse and best case conditions) will be tested before proceeding with setting up the entire panel set. If these panels do not crack, then this portion of the test program will either be aborted or modified, e.g., we may still want to place modified panels of the different concrete mixes to identify potential problems such as referred to in the previous paragraph. If there is a need to abort this portion of the test program, there will be a need to carry more of the test parameters and/or values to the field testing program for evaluation.

Last, in addition to preparing and monitoring the 5' x 10' test panels, special testing will be conducted in the laboratory on SIP forms to assess the affects of extremely hot and cold forms at the time of placement on the quality of the concrete.

Field Testing Program. The "bottom line" testing to assess the effectiveness of various early deck crack mitigation actions is through an actual field bridge testing program. The ideal case would be where earlier material and structural laboratory testing have screened and pinpointed the most appropriate corrective actions to take, and then these actions are validated or demonstrated in a field program. However, the ideal doesn't exist and it is fully anticipated that it will be necessary to look at various combinations of parameter improvements in the field program. One of the parameters, i.e., concrete placement sequence, can only be evaluated in a field program. Also, it should be noted that parameters such as deck thickness and crushed aggregate vs. gravel should not significantly affect early cracking and thus their affect will probably not be seen in a short term evaluation whether in the laboratory or in the field.

Because it is desired to identify the better corrective actions (to mitigate early deck cracking) via laboratory testing to the extent possible, it will not be possible to define the specific field testing program at this time. However, it will basically consist of ALDOT contracting to have bridge decks of varying concrete mixes and aggregates, thickness, rebar arrangement, curing, forming, and concrete placement sequences constructed under otherwise identical conditions in order to assess the affect of these parameters on early deck cracking. Figure 7.5 illustrates the field testing program planned for this investigation.

The field test bridge(s) need to be as identical as possible in order not to contaminate the test data and results. More specifically, the requirements for the test bridge(s) are as follows:

a. Use same contractor (and subcontractors) for all test decks.

b. Use same aggregates, cement type and supplier, and flyash other admixture suppliers for all test decks (except where need different value for test parameter). The contractor shall submit sufficient materials (aggregate, cement, admixtures) as per

Figure 7.5. Field Test Deck Layout/Plan

the ALDOT specifications not less than 60 days prior to construction to allow development of mixture proportions.

c. Use same ready-mix concrete supplier for all test decks.

d. Use normal ALDOT specifications and acceptance requirements for PC control deck.

e. Use same span lengths and supportive girders for all decks.

f. Use simple spans and place each test deck in one placement if possible. If not possible, place a longitudinal construction joint at the bridge center line.

g. Have test decks be part of same bridge if possible. If not possible, have bridges be close to each other and the same design.

h. Construct all test decks at the same time of year (same month) under approximately the same weather conditions, and at the same time of day.

i. Use same deck forming, concrete placement and other construction sequences and procedures, etc.

j. Place, consolidate, and finish all test decks in an identical manner (except in cases where looking at the affect of placement sequence). If any of test deck concretes

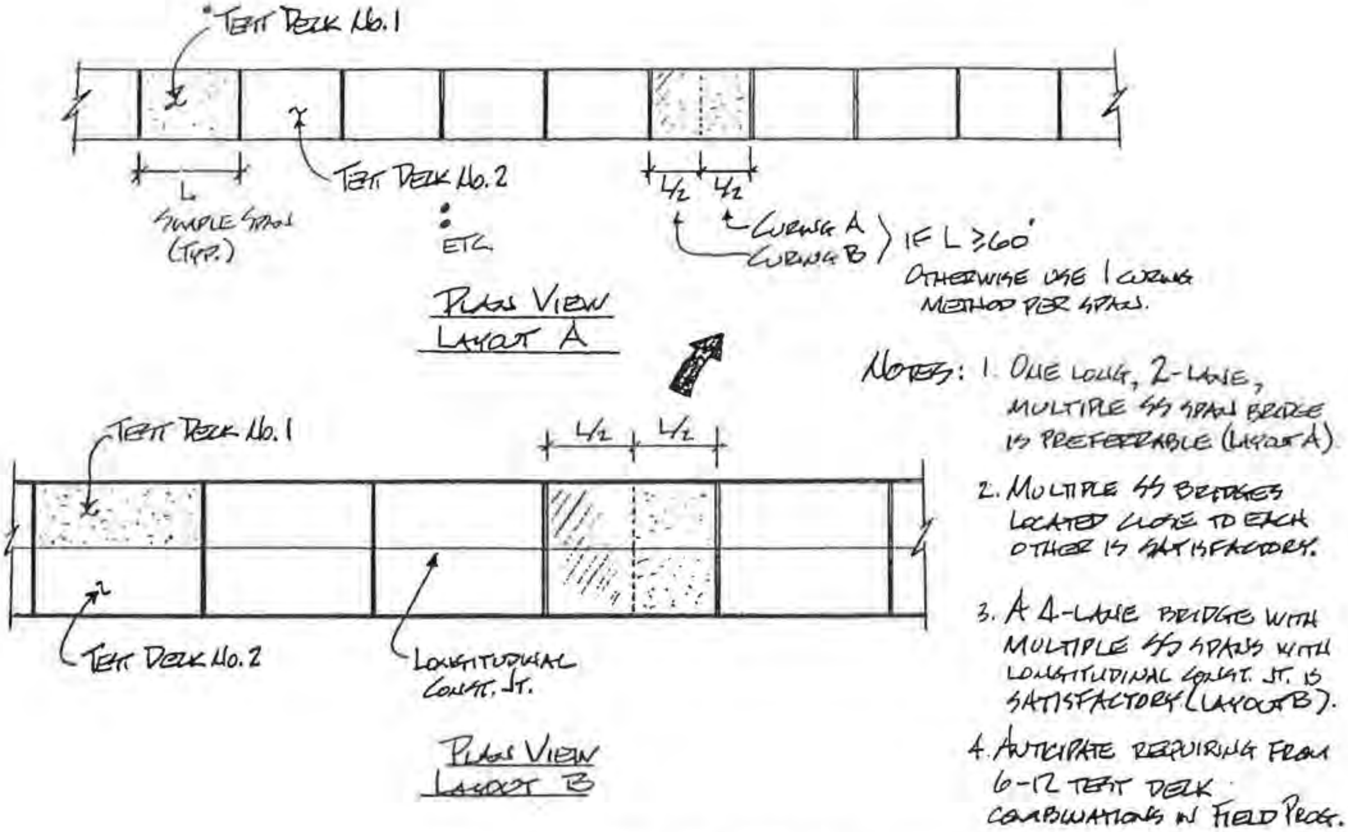

Figure 7.5. Field Test Deck Layout/Plan

the ALDOT specifications not less than 60 days prior to construction to allow development of mixture proportions.

c. Use same ready-mix concrete supplier for all test decks.

d. Use normal ALDOT specifications and acceptance requirements for PC control deck.

e. Use same span lengths and supportive girders for all decks.

f. Use simple spans and place each test deck in one placement if possible. If not possible, place a longitudinal construction joint at the bridge center line.

g. Have test decks be part of same bridge if possible. If not possible, have bridges be close to each other and the same design.

h. Construct all test decks at the same time of year (same month) under approximately the same weather conditions, and at the same time of day.

i. Use same deck forming, concrete placement and other construction sequences and procedures, etc.

j. Place, consolidate, and finish all test decks in an identical manner (except in cases where looking at the affect of placement sequence). If any of test deck concretes have a recommended method of placement, e.g., by pumping for SCC concrete, then placement by the preferred method should take precedence over requiring uniformity of placement for all.

k. Cure all decks in the manner normally employed for ALDOT decks, i.e., spray-on membrane and cover with polyethylene sheeting for 10 days (except in cases where looking at the affect of curing).

l. Finish construction of bridge curb and/or guardrails in an identical manner (same construction, people, equipment, timing, procedure, etc.) For each test span.

Again, as in the laboratory testing program, the field assessment does not lend itself well to strain gaging and strain-stress evaluations. Hence, the primary information to be collected during the field testing will be as follows:

- Concrete testing and specimen preparation at the time of placement. More than normal such testing will be conducted. Basically the same set of tests as in the Laboratory Material Testing Program will be conducted for comparisons and for later explaining and/or predicting deck cracking performance.

- Deck constructability and cost information to assist in later decisions on final corrective actions to recommend. This will consist of monitoring and recording all significant deck construction activities during the construction phase of the test bridge. This should include appropriate activities before, during, and after concrete placement, performance sensors being emplaced, materials QC/QA activities, etc. It should include preparing a time-log of significant activities from the beginning of the test bridge emplacement through opening of the bridge to traffic. Date and time of day of pouring of each deck should be recorded along with the weather conditions (T, RH and wind speed). It should also include developing a test deck cost table which reflects the in-place cost of each of the different material decks. The table should allow comparisons of initial cost (with the cost of special test sensor emplacements not included) of the various deck test materials. It should also allow estimates of the required years of additional service life that would need to be achieved for the higher initial cost deck materials to prove to be cost effective.

- Deck early crack performance (1-day to 9 months) to assess the effectiveness of the various parameters tested in mitigating early deck cracking. Chronological records of deck cracking (top surface) will be developed based on visual inspections commencing with early cracking (1-day through 9-months). Formal visual inspections for cracking will be made at the following ages:

1-*day*
2-*day*
5-*day*
10-*day*
15-*day*
30-*day*
60-*day*
180-*day*
270-*day*

Limited to select area inspections due to limited access due to wet curing burlap (1-day, 2-day, 5-day)

It is anticipated that the field test bridge will end up with anywhere from 6 to 12 test decks of varying combination of parameters. The final number and combination of parameters will be decided on by the project PIs in concert with ALDOT after the laboratory investigations are completed. However, coordination to begin arranging the integration of the field test deck program into a bridge(s) RR project or a new bridge(s) project must begin early even though all variables are not yet finalized.

It should be noted that the superior stiffness and deflection reduction properties of the thicker test deck combinations can easily be demonstrated and assessed analytically. However, since the various test deck combinations will be altogether and readily available, ALDOT may want to experimentally confirm the superior structural performance of the thicker decks. This could probably be best done by having ALDOT's bridge load testing team (in the Maintenance Bureau) conduct their loading/strain/deflection measuring testing on the thicker and control deck spans. If ALDOT chooses to do this, it is recommended that they do it in the time frame of this project and coordinate it with the project PIs. This will allow use of the resulting data to be compared with analytically predicted difference in performance. Last, the test decks to be constructed in this investigation should also be monitored for longer term performances under traffic loading. This could be done by ALDOT or by the PIs, in a later follow-up project.

7.4 Third Proposed Field Testing Program

The research objectives and work plan for the third field testing program, which was proposed to the Auburn University Highway Research Center in July 1996 are given below. The proposed project was much smaller in scope and cost than the first 2 proposals.

Objectives

The objective of this proposed project is to assess the effectiveness, construction practicality, and cost efficiency of using Type K shrinkage-compensating concrete in reducing early shrinkage cracking of bridge decks. This assessment should be made via comparing the early cracking performances of identical SCC and PCC bridge decks in the field.

Work Plan

Service life/durability performance assessment by direct means is not reasonable from a time standpoint. Hence, material compressive and tensile strengths, modulus of elasticity, rapid chloride ion penetration (RCP) value, air content, slump, w/c ratio, density, dimensional stability, rate of loss of slump, and set time will be monitored for the concrete mixes employed in this study. Deck top surface crack developments will be monitored for the field test decks. These will be used, along with engineering judgement, to predict service life/durability enhancement or nonenhancement of the SCC deck relative to the ordinary PCC deck.

A listing of the work steps/tasks for the project is presented below:

1. Coordinate with the Alabama Department of Transportation (ALDOT) to identify a test bridge site where a SCC deck can be emplaced along with a standard PCC for comparison. A minimum of 2 spans/decks are needed, i.e., one of SCC and one of PCC. If possible, a third span should be included as indicated in Figure 7.6.

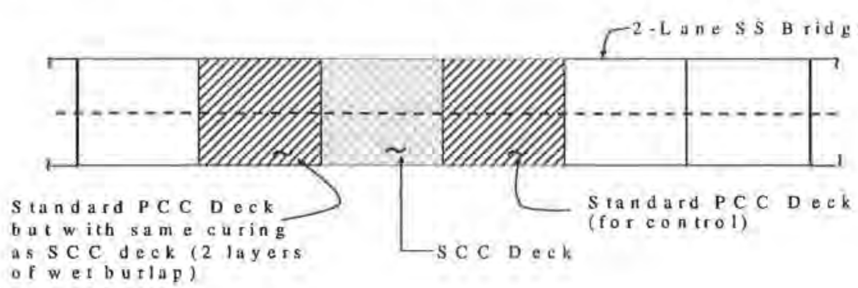

Figure 7.6. Field Test Deck Layout

2. Have test bridge/spans be as identical as possible in order not to contaminate the test data results. More specifically, the requirements for the test bridge/spans are as follows:

   a. Use same contractor (and subcontractors) for the test decks.

   b. Use same coarse and fine aggregates.

   c. Use same ready-mix concrete supplier for the test decks.

   d. Use normal ALDOT specifications and acceptance requirements for PCC control deck.

   e. Use same span lengths and supportive girders for all decks.

   f. Use simple spans and place each test deck in one placement if possible. If not possible, place a longitudinal construction joint at the bridge center line.

   g. Have test decks be part of same bridge if possible. If not possible, have bridges be close to each other and the same design.

   h. Construct test decks at the same time of year (same month) under approximately the same weather conditions, and at the time of day.

   i. Use same deck forming, concrete placement and other construction sequences and procedures.

   j. Place, consolidate, the finish the test decks in an identical manner. However, since pumping is the recommended method of placement for SCC concrete, then placement by the preferred method should take precedence over requiring uniformity of placement.

   k. Cure PCC deck in the manner normally employed for ALDOT decks, i.e., spray-on membrane and cover with polyethylene sheeting for 10 days. Cure SCC deck by wet curing with 2 layers of wet burlap for 10 days. If a second PCC test deck is utilized, then cure this deck in the same manner as the SCC deck.

   l. Finish construction of bridge curb and/or guardrails in an identical manner (same construction, people, equipment, timing, procedure, etc.) for each test span.

3. Prior to test deck construction, have a preconstruction meeting to discuss all aspects of the bridge deck construction. Parties at this meeting should be the contractor, concrete foreman, concrete supplier, materials test lab, inspector and project manager. Cement and admixture reps should also attend if possible.

4. At the time of emplacement of the test decks, concrete material testing, and test specimens will be prepared for later testing as indicated below.

a. Concrete temperature

b. Slump

c. Air Content

d. Unit weight

e. Cylinders for 3, 10, and 28-day compressive strength

f. Cylinders for 3, 10, and 28-day splitting tensile strength

g. Cylinders for modulus of elasticity and Poisson's ratio testing

h. 3" x 3" x 10" prisms for restrained shrinkage testing

i. 4" x 8" cylinders for rapid chloride permeability (RCP) testing

j. Concrete time of workability/set

Later test concrete specimens at the appropriate time.

5. Monitor and record (via written descriptions) all significant deck construction activities during the construction phase of the test decks. This should include appropriate activities before, during, and after concrete placement, and material QC/QA activities. It should include preparing a time-log of significant activities from the beginning of the test bridge emplacement through opening of the bridge to traffic. Date and time of day of placement of each deck should be recorded along with the weather conditions (T, RH and wind speed). It should also include developing a test deck cost table which reflects the in-place cost of each of the different material decks. The table should allow comparisons of initial cost (with the cost of special test sensor emplacements not included) of the various deck test materials. It should also allow estimates of the required years of additional service life that would need to be achieved for the higher initial cost SCC to prove to be cost effective.

6. Visually monitor the top surfaces of the test decks for cracking. Chronological records of deck cracking (top surface) will be developed commencing with early cracking (1-day) through 6 months. Formal visual inspections for cracking will be made at the following ages:

$$\left.\begin{array}{r} 1\text{-}day \\ 2\text{-}day \\ 5\text{-}day \end{array}\right\} \quad \text{Limited to select area inspections due to limited access due to wet burlap curing.}$$

$$\begin{array}{r} 10\text{-}day \\ 30\text{-}day \\ 60\text{-}day \\ 180\text{-}day \end{array}$$

7. Analyze and synthesize results from the work above and draw appropriate conclusions and recommendations.

8. Prepare Final Report for ALDOT.

It should be noted, in executing the Work Plan above, that the principal participants in this research effort are (1) the ALDOT, (2) a bridge construction contractor, (3) the Type K Cement Manufacturer - Blue Circle Cement Company, and (4) the Highway Research Center at Auburn University. Each participant will provide the following in support of the project.

- The ALDOT will furnish the bridge, allow the placement and monitoring of the test SCC and PCC decks, conduct standard concrete testing at the site, and prepare additional test specimens as indicated.

- The bridge construction contractor will coordinate the delivery and placement of the SCC and PCC concretes for the test decks, and allow monitoring of the test deck for cracks as indicated in item (6).

- Blue Circle Cement Company will provide the contractor/ready-mix supplier the required amount of Type K cement for the same price as Type I/II cement. They will provide technical assistance in finalizing the SCC mix design, and in placing the SCC as needed. Last, they will arrange rapid chloride permeability (RCP) testing of the PCC and SCC mixes.

- Auburn University will coordinate the placement of the SCC and PCC test decks with the ALDOT to allow valid comparisons and assessments, as well as, monitor

    - constructability parameters
    - cost parameters
    - concrete material properties (with ALDOT)
    - deck cracking performances

  for the SCC and PCC test decks.

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 General

Based on results of reviewing the literature, talking with cement manufacturers, observing SCC placements in the field, visiting with highway agencies (Ohio Turnpike Commission and NY Thruway Authority) having first hand experience with SCC bridge decks, and performing laboratory mix designs and associated testings, it is the authors opinion that the use of SCC in bridge decks to reduce early shrinkage cracking is an attractive and viable option. Technically, SCC does what it is advertised as doing, i.e., it significantly reduces early shrinkage cracking of bridge decks. Its somewhat higher initial cost could prove to be a lower life-cycle-cost due to reduced shrinkage cracking and potentially reduced deterioration and thus longer service life. This latter statement, i.e., a potentially longer service life, is a key issue and question that should be answered if SCC is to be seriously considered as a viable deck material by the ALDOT. Currently it appears that for bridge decks,

1. SCC is more expensive than PCC.
2. SCC is more demanding of rapid concrete placement and proper curing than PCC.
3. Deck shrinkage cracking, while undesirable, is not viewed as that serious a problem by the ALDOT.

If, in fact, bridge deck shrinkage cracking is primarily an aesthetics problem in Alabama and does not significantly affect deck service life/durability, then SCC should not be used. However, if early shrinkage cracking of bridge decks in Alabama is, in fact, a significant cause of premature deck deterioration, then SCC is a logical and rational potential improvement which should be explored and tested in the field.

8.2 Conclusions

Visits with the largest user of SCC bridge decks in the country, the Ohio Turnpike Commission (over 500 SCC bridge decks) and the second largest user, the NY Thruway Authority (over 40 SCC bridge decks) allowed the PIs to observe bridge after bridge of nearly crack free decks. The NY Authority decks did, however, have some decks which exhibited serious scaling damage which has been attributed to lack of proper air content and freeze-thaw deterioration. Ohio Turnpike Commission (OTC) engineers concluded that SCC requires

- high w/c ratio (extra water is needed for ettringite development)
- faster placement than PCC with the preferred method of placement being by pumping (SCC has a faster set time and shorter "pot life" than PCC)
- faster implementation of the curing process than with PCC and a continuous wet curing for 7-days.

On the positive side, the OTC indicated that SCC

- is easier to place and consolidate than PCC because of its higher water content and greater fattiness/cohesiveness.
- is easier to finish than PCC because of its greater fattiness/cohesiveness.
- is easier to tine than PCC because of its greater fattiness/cohesiveness

A field visit to a large warehouse floor placement (17,000 ft^2) in Calera, AL in May 1996 allowed a first hand observation of the excellent workability and ease of placement, consolidation, and finishing of SCC. The concrete was placed by pumping which allowed placement at a fairly rapid rate. It should be noted that the demands of SCC for relatively rapid placement (faster than PCC), attention to early curing, and the providing of continuous 7-day wet curing are all actions which really should be employed with PCC and would result in enhanced concrete quality and durability.

Results of the laboratory concrete mix design and property evaluation and sensitivity testing verified that SCC mixes should be viable mixes for bridge decks to reduce shrinkage cracking. The testing revealed that,

1. SCC mixes require more water (as expected) to achieve the ettringite growth and expansive properties.

2. Time of set was quicker for SCC (as expected) by about 100 minutes, as was the rate of slump loss. However, both of these were not so quick to cause construction problems assuming placement of the concrete by pumping.

3. The compressive and splitting tensile strengths of the SCC mixes were superior to those of PCC.

4. The compressive strengths of both the Type K and Type I cement mixes were enhanced by the addition of microsilica. However, the splitting tensile strengths varied with some mixes showing higher tensile strength with the addition of microsilica and others showing the reverse.

5. The SCC concrete length bar specimens exhibited significant early (first 7 days) expansion. This would be followed by later shrinkage at the end of the 7-day cure period. However, the net volume change with SCC would probably be a small net expansion, or at worst a small net shrinkage which would be much less than with PCC. This in turn would result in much less deck shrinkage cracking.

6. Type K cement mortar bars exhibited approximately 4 times more expansion than mortar bars made with Type I cement.

7. Results of the simulated winter (C), summer (H), fall/spring (M), and low humidity (D) construction and curing conditions indicated the following:

   - Compressive strengths were greater for cold (C) conditions and worse for dry (D) conditions.

   - Tensile strengths were surprisingly greater for dry (D) conditions and about the same for the other three condition. Without exception, the Type K cement tensile strengths were greater than those using Type I cement.

   - Construction/curing conditions had significantly greater effects on strengths (both compressive and tensile) for Type I cement than for Type K cement. This was true even for the dry (D) condition which is surprising.

   - Hot (H) conditions yielded the larger slump losses, and cold (C) conditions the smaller. However, there was considerable data scatter among the cold (C), moderate (M), and dry (D) conditions. Weather conditions were much less a factor on slump loss than the type of cement. Type K cement exhibited significantly greater slump losses.

   - Length bar expansions were greatest for moderate (M) and hot (H) conditions, and were about 10-15% lower for cold (C) conditions. For dry (D) conditions, the length bars exhibited shrinkages of about the same magnitude as the expansions for the other three conditions. This demonstrates the great sensitivity of Type K mixes to proper wet curing.

8.3 Recommendations

Based on the results of this study, which are presented in this report, the authors recommended the following to the ALDOT regarding the possible use of SCC for their bridge decks.

1. Assess in a detailed and quantitative manner the impact/effect of early shrinkage cracking on the performance and durability of concrete bridge decks in Alabama. This assessment will probably have to be made by using the historical bridge records of the ALDOT along with numerous interviews with bridge inspectors and maintenance personnel in all of ALDOT's Divisions.

2. Use the results of the assessment in (1) above to judge the seriousness of early shrinkage cracking on future bridge deck performance and durability. If the results reflect zero or minimal impact of early shrinkage cracking on future deck maintenance cost and durability, then dismiss consideration of SCC as an option for bridge decks in Alabama. If the results reflect significant negative effects, then the use of SCC for bridge decks should be closely looked at and considered.

3. If the results of (2) above indicate the early shrinkage cracking has a significant negative impact on future deck performance and durability, then the ALDOT should undertake a field performance evaluation of SCC bridge decks. As a minimum, this field evaluation should consist of 3 identical test bridge/spans at the same, or approximately the same location, where the following 3 test decks can be emplaced and monitored.

    - Standard ALDOT PCC Deck (material and construction requirements)
    - SCC Deck
    - Standard ALDOT PCC Deck but cured the same as the SCC Deck

    Since bridge deck service life/durability performance assessment by direct means is not reasonable from a time standpoint, early shrinkage cracking of the 3 test decks should be used as the primary parameter to compare performances and to assess service life. Material properties for all 3 test deck materials should be evaluated for comparative purposes and also to allow correlations with deck cracking performances.

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APPENDICES

APPENDIX A

Guidelines for Use of Type K Cement/Concrete

Plant and In-Transit

1. Presoak Sand and Coarse Aggregates - This is very important especially during hot weather. Less than saturated aggregates (especially gravels and lightweights) can severely increase mix temperature, reduce slump, accelerate setting and cause plastic shrinkage cracks.

2. Admixtures - Only use admixtures which have been included and approved in the test mix. Any proposed increases in dosages should be reviewed with the Engineer.

3. Truck - Wet the mixer drum and then expel all water. Make sure all wash water is expelled from return trucks before proportioning concrete ingredients in trucks. Proportion the concrete such that the volume placed in a truck is at least 2 cubic yards less than the rated capacity of the mix truck. This is necessary because Type K mixes are more cohesive than regular PC concrete and require additional space for proper mixing.

4. Batching Procedure - Charge mixer with approximately three-fourths of the mixing water with the air entrainment admixture if needed. With the mixer rotating at mixing speed start ribbon feeding of the coarse and fine aggregates. With approximately one-half of the aggregates in the truck, begin ribbon feeding of Type K cement slowly to prevent balling. Proceed until all aggregates and cement are loaded. Add all remaining water and water reducers as required.

5. Mixing Procedure - The truck should be pulled out from under the weigh batcher and washed down with the drum rotating at mixing speed. The shrinkage compensating concrete should be mixed for 70 revolutions or timed 5 minutes at mixing speed. A slump should be taken before the truck leaves the yard to assure a 7-inch slump.

6. Delivery Time - In accordance with ACI Standards, this should be limited to 90 minutes.

7. Mix Temperature - The site temperature for the mix is limited to 90°F. Therefore, steps should be taken to ensure that the plant mix temperature is adjusted to compensate for ambient temperature, travel time and other variables. (Note that the maximum ambient temperature at placement is 80°F.) Like PC concrete, high temperatures and long mixing periods usually produce lower strength SC concrete. Additionally, they also reduce the amount of expansion and thus reduce the effectiveness of SC concrete in mitigating drying shrinkage cracking.

8. Slump - The specifications limit slump at the site to 4" - 6". Since these mixes need more water than most standard mixes (to allow growth of the ettringite) and the w/c ratio may even be as high as 0.45, it is suggested that the plant slump be a minimum of 7" for typical placements. Within reason, this higher slump and water content are needed and will not detract from the performance of the hardened concrete.

At Site

1. Ambient Temperature - Maximum ambient temperature at placement is 80°F.

2. Cool off the rebars and forms - The mix temperature at placement is limited to 90°F. Since the ambient temperature is limited to 80°F, take steps to cool off (via wetting with clean fresh water) any deck components which might increase the inform temperature of the mix. If wooden forms are used, the wetting should be done to prevent the form from "sucking" water from the concrete mix.

3. Slump - The slump range is 4" - 6". Target the high end. Slump loss due to pumping will be in the same order as that experienced with higher slump standard mixes.

4. Air Entrainment - The ability of the mix to entrain air should be normal. The higher slump and water content should help. Air losses due to pumping will not be unusual.

5. QC/QA of Concrete - Monitor slump, air content, concrete temperature, and concrete strength at point of placement of the concrete, i.e., at the rear of the truck if placing by bucket, or at the end of the pump line if placing by pumping.

6. Preplacement Meeting - Prior to making the first Type K placement, a preplacement meeting should be held to discuss all aspects of bridge deck concrete placement. Parties at this meeting should be the contractor, concreting foreman, concrete supplier, test lab, inspector and project manager. Cement and admixture reps should also attend if possible. The meeting should conclude with a discussion of the "dry-run" (see below), which should be made within 1 or 2 days.

7. Dry Run - Conduct a complete and full size "dry run" prior to placing the first Type K deck to familiarize supplier, contractor, finishers, tests lab, inspectors, etc., of what to expect with Type K concrete. A 3 yard quantity should be batched, delivered, poured into a dummy form, finished, and initial curing applied as a dry run shortly before (1 or 2 days) making the first Type K deck placement. This same dry run can provide concrete for testing to assure the earlier accepted mix design can be delivered to the site and properly emplaced.

8. Placing Concrete - Delaying placing activities at the job site should be avoided since shrinkage compensating cement concrete loses slump faster than normal concrete. Type K has more cohesiveness or "fat" than PC concrete, and thus has less tendency to segregate and is more easily pumped than PC concrete. Because of its higher slump, reduced tendency to segregate , and greater sensitivity to prolonged mixing time and time delays, it is highly recommended that Type K deck concrete be placed by pumping. Placing of the concrete should be scheduled early in the day before temperatures rise; the deck should not be placed if the ambient temperature is above 80°F. Also evening placement has advantages in that the greatest exothermic heat dissipation occurs during the cooler nighttime ambient temperatures.

9. <u>Bleed Water</u> - There will be a definite lack of surface bleed water. Typically only a sheen will appear. Therefore, spraying with Confilm (or equivalent) <u>may</u> be required in hot weather to limit rapid evaporation immediately behind the screed. This will reduce the tendency for plastic shrinkage cracking.

10. <u>Finishing</u> - The concrete will be creamy but may be sticky. Generally it finishes very well with 1 or 2 passes with a bull float. Do not over-finish with float and do not steel trowel. Do not retouch concrete after Confilm or other monofilm have been applied. It should be noted that concrete made with shrinkage-compensating cement will exhibit little or no bleed water; therefore, care must be taken to begin finishing operations at the proper time.

11. <u>Evaporation</u> - The use of Confilm or other acceptable monomolecular film is recommended for placement during hot weather for the purpose of reducing evaporation and thus prevention of drying of the deck surface prior to beginning the curing process. It acts to retard stiffening of the surface and thereby its tendency for plastic shrinkage cracking. This is particularly important for Type K concrete because of little bleed water. Apply film as a spray immediately after finishing.

12. <u>Curing</u> - Two layers of pre-wet burlap sheets should be laid on the surface as soon after texturing as possible. These sheets should be maintained continuously wet for a period of 10 days after pouring. The wetted burlap serves two purposes, (1) it provides water as needed for the concrete to properly hydrate and grow the ettringite, and (2) it keeps the deck cooled from heat of hydration and solar temperature build-ups. In turn, these greatly reduce drying shrinkage and thermal cracking, which are the dominant causes of nonstructural/early deck cracking. Care should be taken that greatly excessive water is not added that would cause run-off in the bridge gutters and damage the fresh concrete surfaces at those locations.

APPENDIX B

Supplementary Specifications for Concrete Test Decks

A. Description - This item shall consist of furnishing and placing Portland cement concrete bridge decks using shrinkage compensating cement concrete (SCC), micro silica concrete (MSC), shrinkage compensating cement and micro silica concrete (SC/MSC), and as a control the standard Class A-1c concrete (CC). These special test concretes will be furnished and placed in accordance with these Specifications, and in reasonably close conformity with the lines, grades, and dimensions shown on the Plans. All applicable provisions of Division 500 of the Specifications shall apply except as modified herein.

B. Preplacement Meeting - Prior to placing the first test span, a preplacement meeting will be held (at a time determined by the Engineer) to discuss all aspects of the test deck concretes and construction. Parties at this meeting will be the contractor, concreting foreman, concrete supplier, cement and admixture manufacturing representatives, ALDOT personnel, research project PI's, and any others deemed appropriate by the Engineer.

C. Materials - The cement for the SCC and SC/MSC shall be expansive hydraulic cement conforming to ASTM C845, Type K.

The cement for the MSC and CC shall be Type I or II Portland Cement meeting the requirements of AASHTO Designation M85.

Admixtures used in the concrete mixture must be compatible and shall be dispensed in accordance with the manufacturer's recommendations. Admixtures shall be obtained from one source.

Coarse and fine aggregates shall be the same as those used in the standard Class A-lc concrete.

Adequate quantities of all materials, sufficient to complete the proposed pour, shall be on hand at the batch plant prior to all pours.

Course aggregate stockpiles shall be saturated. Saturation shall be completed a minimum of 24 hours prior to use; however, the application of water by sprinkling shall continue as directed by the Engineer.

D. Proportions - The mix designs for the test decks will be as shown in Table 1 below.

E. Slump - Slump at the time and place of concrete placement shall be as shown in Table 1.

F. Entrained Air - Concrete shall contain 4 to 6 percent of entrained air at the time and place of concrete placement as indicated in Table 1.

G. Time and Location of Concrete Quality Monitoring - Concrete slump, air content and cylinders for strength evaluation shall be monitored at the time and place of concrete placement. If the concrete is pumped, this means at the deposit end of the pumping operation. If the concrete is placed by bucket, this means at the end of the delivery truck

Table I
Proportions for Test Deck Concrete Mixes

One Cubic Yard Proportions				
Type Concrete	SCC	SC/MSC	MSC	ALDOT Class A-1c CC**
Cement Factor (lbs.)	*	*	*	620
Microsilica	None	*	*	None
Maximum Water in Gallons	*	*	*	33
Fine Aggregate (lbs.)	*	*	*	1097
Coarse Aggregate (lbs.)	*	*	*	1857
Entrained Air % by Volume	4-6	4-6	4-6	4-6
Maximum Consistency Slump (in.)	6	6	3½	3½
Coarse Aggregate Size No.	57/67	57/67	57/67	57/67
Min. psi Strength 28 Days	4500	4500	4500	4000

* See Table E1. However, the proportions in Table E1 need to be adjusted to achieve w/c ratios and slumps which are more consistent with those needed for quality field placement of mixes containing Type K cement. See mix designs given in Appendix D.

** The Class A Type lc mix will require an approved water reducer or a water reducing set retarder in the job mix in order to obtain the 3.5 inch slump.

chute. Concrete temperature and ambient temperature shall be monitored at end of the delivery truck chute.

H. Mixing Concrete - If an approved set-retarding or a water-reducing and set-retarding admixture is used, discharge shall be completed within 75 minutes after the combining of the water and the cement.

I. Concrete Delivery - When supplying concrete using Type K shrinkage-compensating cement, i.e., the SCC and SC/MSC, ready-mix plants identified as "Truck Mix" shall proportion the concrete such that the volume placed into the truck is at least 2 cubic yards less than the rated capacity of the mix truck. The cost for complying with this requirement shall be included in the appropriate concrete bid item. Concrete trucks should be inspected for being free of "wash out" water prior to batching another load.

J. Placing Concrete - Maximum ambient temperature at the time of placement of concrete shall be 80°F, and maximum temperature of the fresh concrete shall be 90°F.

If required by the Engineer, the deck formwork, beam flanges, and reinforcing shall be thoroughly sprinkled prior to placement of the concrete. A water storage tank and pump or other source of clean water shall be available at the site to allow wetting of deck forming and rebar and/or cooling of forming and rebar prior to placement, and for curing of the concrete after placement.

Concrete pumpers, if used, shall be set up in such a manner as to prevent excess free fall of concrete in the pump lines. Pumpers shall be repositioned if so directed by the Engineer to prevent excess free fall. When concrete is pumped, an appropriately located plywood runway should be provided for slick lines, i.e., water hose, pump line, etc., and for testing and monitoring concrete quality at the end of the pump.

Immediately after the deck or slab has been floated, an approved monomolecular film shall be applied to the concrete surface. The film material shall be Master Builders-Confilm or approved equal, and shall be applied in accordance with the manufacturer's printed instructions. It should be noted that the monomolecular film <u>is not</u> a curing compound, but rather a compound to reduce surface evaporation and thus plastic shrinkage cracking.

Placing, screeding, floating and monomolecular film application shall be a continuous, close operation with adequate manpower and equipment.

K. <u>Curing</u> - Concrete shall be continuously wet cured for a period of 10 days via using 2 layers of wet burlap. Plastic coated burlap blankets cannot be used. The burlap shall be soaked for 24 hours prior to pouring of the deck, and should be placed on the deck immediately after finishing (after screeding and floating and initial set where the wet burlap does not mar the concrete surface). The burlap should be kept continuously wet for a period of 10 days. The burlap shall be retained in place for an additional 10 days after which it may be removed at the Contractor's discretions.

Storage tanks for curing water shall be on site and filled before a placement will be permitted to start. Storage tanks shall remain on site throughout the entire cure period. They shall be replenished, as required, with a shuttle tanker truck or a local water source such as a fire hydrant. Care shall be taken to avoid thermal shock or excessively steep thermal gradients due to the use of cold curing water. Curing water should not be more than 20°F cooler than the concrete, because of surface temperature stresses which could cause cracking.

L. <u>Loading</u> - To protect the fresh and green concrete during and immediately following pouring operations, plywood walkways should be provided over rebar which is continuous and directly adjacent to that in the section being placed. The width of these walkways should be sufficient to handle all construction "traffic" around the deck section being placed and should remain in place for 3 days after placing.

No heavy construction traffic (cranes, ready mix trucks, paving machines, rollers, etc.) will be permitted on the deck and abutment slab concrete until the loading requirements for opening to traffic have been met. Construction equipment weighing 6 tons or less will be permitted on the deck when the requirements for removing falsework have been met.

M. Surface Finish - All vertical faces of deck edges shall have a rubbed finish in accordance with 501.03. Defects shall be corrected prior to rubbing with a cement mortar of the same type and proportions as the concrete. Where specified on the plans, deck edges shall be grout cleaned in accordance with 501.03 using white Portland cement for the SCC, SC/MSC, and MSC decks. This is for matching of color as all three of these concretes are noticeably whiter than normal Type I or II cement.

N. Preplacement Testing & Familiarization - The supplier shall batch a minimum of 3 cubic yards of concrete of each type just prior to emplacing that concrete type for two purposes,

(1) for preplacement field and laboratory testing.

(2) for familiarization for the construction inspectors, contractor managers, and concreting foreman.

The concrete shall be delivered to the job site or batched at the concrete plant to simulate job conditions, as directed by the Engineer. Upon completion of the field and laboratory testing, a portion of the concrete will be placed into a prepared 4' x 8' x 8" simulated deck form to allow familiarization with workability, compaction, finishing, application of the monomolecular film, initial set time, and application of the wetted burlap. The remaining concrete shall be wasted and disposed of by the supplier off of the ALDOT right-of-way unless otherwise approved by the Engineer. The cost for complying with this requirement shall be in accordance with "Basis of Payment."

0. Concrete Test Specimens - On each deck, six test cylinders will be made from each 50 cubic yard "lot", or fraction thereof, of concrete that is incorporated into the work each day. From the six test cylinders, 3-day and 10-day compressive strengths shall be determined using one cylinder each. At 28 days, three cylinders are to be tested and the average compressive strength recorded and defined as the "strength test." The remaining cylinder shall be used for determining 28-day split tensile strength.

Methods of sampling, curing and testing concrete test specimens shall be in accordance with the "American Society for Testing Materials," applicable sections.

Responsibility of Contractor - To facilitate testing and inspection the contractor shall:

a. Furnish any necessary labor to assist the ALDOT in obtaining and handling samples at the project.

b. Provide and maintain for the sole use of the ALDOT wooden cure boxes, unless other methods are specifically approved by the Chief Engineer, for safe storage and proper curing of concrete test specimens on the project site for the first 24 hours as required by "Method of Making and Curing Concrete Test Specimens in the Field" (ASTM C31).

No separate payment will be made to the contractor for this work but will be included in the unit price bid.

Acceptance of Concrete - The strength level of the concrete will be considered satisfactory so long as the strength test" equals or exceeds the specified strength.

Test results failing to meet the above requirements will be the basis for determining replacement of the concrete at the expense of the Contractor or a proportionate credit to the ALDOT.

P. Method of Measurement - The quantity shall be measured as per ALDOT's Specifications and shall include all labor, materials, equipment and incidentals necessary to complete this item of work.

Q. Basis of Payment - The payment will be made at the contract unit price bid for the following:

Unit	Item/Description
CY	Special concrete mix for decks using shrinkage compensating cement (SCC).
CY	Special concrete mix for decks using shrinkage compensating cement and micro-silica admixture (SC/MSC).
CY	Special concrete mix for decks using micro-silica admixture (MSC).
CY	Standard Class A-lc concrete for decks (PC).
CY	3 CY batch of each of the four concrete mixes above for use in preplacement testing.

APPENDIX C

ALDOT Master Concrete Proportion Table and Aggregate Sizes

Table C1: ALDOT Master Concrete Proportion Table*

Concrete Class -Type	A-1a	A-2a	B-3	A-1c	C-4	E-6a	S-7a
Cement Factor (lbs.)	620	620	508	620	620	658	696
Fly Ash (lbs.)**	None	None	None	None	None	None	None
Maximum Water (gallons)	36	39	36	33	33	35	39
Fine Aggregate (lbs.)	1088	1204	1487	1097	1504	947	1096
Coarse Aggregate (lbs.)	1839	1829	1712	1857	1677	1974	1696
Entrained Air % by Volume	3 - 5	None	None	4 - 6	None	3 - 5	3 - 5
Maximum Consistency Slump (in.)	3	3	3	3.5	3	3.5	7
Coarse Aggregate Size Number	57/67	57/67	57/67	57/67	57/67	57/67	57/67
Minimum PSI Strength 28 Days	3000	3000	2000	4000	3000	4500	3000
Water/Cement	0.48	0.52	0.59	0.44	0.44	0.44	0.47

*Master Proportion Table on page 5-2 of ALDOT Standard Specifications for Highway Construction, 1992 Edition, for one cubic yard proportions

**Fly ash use in mixes shall be as specified in Item 501.02 (c)6.

Type 1 - Mix "a", bridge substructure concrete, box culverts, retaining walls and concrete safety barriers.
Type 1 - Mix "c", bridge superstructure concrete.
Type 2 - Mix "a", headwalls, inlets and miscellaneous concrete units.
Type 3 - Slope paving.
Type 4 - Machine laid curbs, gutters or combination curbs and gutters.
Type 6 - Bridge concrete for use when the structure is exposed to salt water, where shown by the plan details or directed by the Engineer.
Type 7 - Underwater concrete.

Table C2: Table of A.H.D. Coarse Aggregate Sizes*

No.	Percent Passing by Weight, Seive No.															
	4"*	3-½"*	3"*	2-½"*	2"	1-½"	1"	¾"	½"	$^{3}/_{8}$"	# 4	# 8	# 16	# 50	# 100	# 200
1	100	90-100		25-60		0-15		0-5								
2			100	90-100	35-70	0-15		0-5								
24			100	90-100		25-60		0-10	0-5							
3				100	90-100	35-70	0-15		0-5							
357				100	95-100		35-70		10-30		0-5					
4					100	90-100	20-55	0-15		0-5						
467					100	95-100		35-70		10-30	0-5					
410					100	85-100	60-85		30-60		18-30	11-20	8-15	5-9		2-6
5						100	90-100	20-55	0-10	0-5						
56						100	90-100	40-75	15-35	0-15	0-5					
57						100	95-100		25-60		0-10	0-5				
6							100	90-100	20-55	0-15	0-5					
67							100	90-100		20-55	0-10	0-5				
63							100	90-100		30-65	5-25	0-10	0-5			
610							100	90-100		25-60		7-30		0-15		
7								100	90-100	40-70	0-15	0-5				
78								100	90-100	40-75	5-25	0-10	0-5			
710								100	90-100	50-85		12-35		0-15		
8									100	85-100	10-30	0-10	0-5			
89									100	90-100	20-55	5-30	0-10	0-5		
810									100		70-90	50-74	38-62	20-42		9-24
8910									100	90-100	60-85	40-70		10-25		1-5
4										100	85-100	10-40	0-10	0-5		
10										100	85-100				10-30	

Source: Alabama Highway Department

*Explanation of Table

1. Tabulated figures are percentages by weight of material finer than each laboratory sieve.
2. Exclusive of light weight aggregates, the minimum dry rodded weight per cubic foot shall be 65 pounds of Sizes 1, 3 and 4, and 70 pounds for other sizes. See Article 801.12 for weights of light weight aggregates.

Table C3: Table of A.H.D. Fine Aggregate Sizes

Agg. Size No.	Description	PERCENT PASSING BY WEIGHT, SIEVE NO.						
		$^{3}/_{8}$"	No. 4	No. 8	No. 16	No. 50	No. 100	No. 200
100	Concrete Sand	100	95-100	80-100	50-90	5-80	0-10	
101	Mortar Sand			100		15-40	0-10	
104	Plant Mix Sand	100						0-12
105	Manufactured Sand	100	95-100		50-80	20-50	10-25	5-12
106	White Concrete Fine Aggregate	100	95-100	75-100	50-90	10-35	5-15	0-5

Source: Alabama Highway Department

APPENDIX D

Type K Mix Designs of Other Agencies

PENNDOT TYPE K MIX DESIGNS

TR-4221A

MIX DESIGN WORKSHEET

CLASS AAAK Concrete
DISTRICT: 8-0
DATE: 1/16/95

Concrete Producer Zeiglers' Project or Plant York, PA.
Mix Design by PennDOT Method

Square Screen Analysis % Passing

Screen Sizes	1 1/2	1	1/2	3/8	#4	#8	#16	#30	#50	#100
F.A.				100	97.4	82.2	68.6	50.1	18.7	4.5
#57	100	98.3	29.0		1.4	1.0				

F.I 2.79

Maximum Density 91.94 P.C.F., S.S.D. % Solids (PDT only) 53.0
S.G. Factor of Slag Volume of C.A.

TRIAL MIX

Wt./Volume Calculations: W/C. RATIO .47 by Wt.

Cement	7.61 Bags x 0.478	= 3.64 C.F.	x (3.15 x 62.4)	= 715	lbs.
Pozzolan		= C.F.	x (x 62.4)	=	lbs.
C.A.	#57	= 10.51 C.F.	x (2.8 x 62.4)	= 1843	lbs.
Water	40.58 Gals. x 0.133	= 5.40 C.F.	x 62.4	= 337	lbs
6.5% Air =	27.0 x .065	= 1.76 C.F.			
	Sub Total	21.31 C.F.			
F.A. =	27.00 − 21.31	= 5.69 C.F. +.37= 6.06			
Weight of F.A.	5.69 X 2.63 x 62.4			= 994	lbs.
Totals/C.Y. =	40.58 Gals.	27.00 C.F.		3889	lbs.

Temperature: Concrete 60° F Air 71° F #/C.F. Actual
Initial: Slump 7 In. Air Content 7.1 % Unit Wt. 142.05 #C.F. Calculated
At Point of Placement: Slump 5½ In. Air Content 6.5 % Mortar Content 16.86 C.F.

STRENGTH	DAYS	1	2	AVG.	DAYS	1	2	AVG.
Compressive	7	5146	4952	5049				
Compressive	28	5836/	6119	5978				

MIX NO. #1 (ADJUSTED)

Wt./Volume Calculations: W/C. RATIO .46 by Wt.

Cement	7.61 Bags x 0.478	= 3.64 C.F.	x (3.15 x 62.4)	= 715	lbs
Pozzolan		= C.F.	x (x 62.4)	=	lbs.
C.A.	#57	= 10.51 C.F.	x (2.8 x 62.4)	= 1843	lbs.
Water	39.58 Gals. x 0.133	= 5.26 C.F.	x 62.4	= 328	lbs.
6.5% Air =	27.0 x .065	= 1.76 C.F.			
	Sub Total	21.17 C.F.			
F.A. =	27.00 − 21.17	= 5.83 C.F. +.37= 6.20			
Weight of F.A.	6.20 X 2.63 x 62.4			= 1017	lbs.
Totals/C.Y. =	39.58 Gals.	27.00 C.F.		3903	lbs.

At Point of Placement:
Slump 5½ In. Air Content 6.5 % Calc. Unit Weight 144.6 #/C.F. Mortar Content 16.86 C.F.

TR-4221A (10-94)

CONCRETE MIX DESIGN FORM

JMF No. 95 3A2

MAT'L CLASS AAAK
DATE: 1/16/95
DISTRICT: 8-0
CMS NO.:

S.R.: Section/Segment FA No.

Concrete Producer/Code Walter W. Zeiglers' Sons, Inc. Plant York, PA.

MATERIAL	TYPE	PRODUCER-LOCATION	SUPPLIER CODE	S.G.	ABS.	LAB. #
Cement	K	Southwestern Cement Co.	SW-PC15			
Pozzolan						
Fine Aggregate	A	York Building Products/York, PA.	YOPMOCJ4	2.63	.39	93-036073
Coarse Aggregate	A (57)	York Building Products/York, PA.	YOP66B14	2.81	.48	93-030067
Water		York Water Co./York, PA.				
Admixture-AEA	AEA	Admixtures, Inc./Sillington, PA.	GRAC115	6.5	oz./c.y. (as required)	
	Retarder	Admixtures, Inc./Shillington, PA.	GRAC115	43.0	oz./c.y. (as required)	
	Super-Plasticer	Admixtures, Inc./Shillington, PA.	GRAC115	43.0	oz./c.y. (as required)	
					oz./c.y. (as required)	

Strength Data Based On: .47 W/C Ratio Taken From Worksheet Dated: 11/16/93

Compr. Str.: 7 days 5049 avg. psi 28 days 5978 avg. psi

%Solids Used: 53.0 F.M.: 2.79

Concrete Mix Summary (One Cubic Yard) by PennDOT Method

Mix No.	Trial	#1		
W/C. Ratio, by Wt.	.47	.46		
Cement, lbs.	715	715		
Pozzolan, lbs.				
Water, lbs.	337	328		
Coarse Agg. (S.S.D.), lbs.	1843	1843		
, lbs.				
Fine Agg. (S.S.D.), lbs.	994	1017		
Total, lbs.	3889	3903		
Unit Weight, lbs./C.F.	144.4	144.6		
Water, gals.	40.58	39.58		
Mortar Content, C.F.	16.86	16.86		
At Pt. of Placement: Slump	5½ in	5½		
Air	6.5 %	6.5		

RECEIVED
JAN 23 1995
CONSTRUCTION UNIT
DISTRICT 8-0
HARRISBURG, PA

Designed by J. C. Zeigler Date 1/16/95

Approved & Submitted by Date 1/25/95

Reviewed by Mat'ls Engineer Date

RECEIVED
JAN 26 1995
MAT'LS UNIT

NY THRUWAY TYPE K MIX DESIGN

Table 1
Observed Batched Mix Proportions(1)

Mix ID	500AAEK	600AAEK	715AAEK	825AAEK
Batch Size, cubic yard	1/20	1/20	1/27	1/27
Ingredients(2)	Mix Proportions used per Batch (1)			
Cement, lbs.	25.0	30.0	26.5	30.5
Stone,lbs.*	90.9	92.1	68.2	68.2
Sand 1,lbs.*	52.5	47.1	29.4	25.3
Sand 2, lbs.*	12.9	11.5	7.3	6.3
Water, lbs.	11.2	12.3	11.9	12.5
Admixture 1	30 c.c.	35.5 c.c.	31.3 c.c.	39 c.c.
Admixture 2	3 c.c.	4.0 c.c.	3.5 c.c.	5.9 c.c.
Property				
Initial Slump, in.	4	3-1/2	2-1/4	6
Final Slump, in.	5	5	4-3/4	5
Air Content, %	6	6	5	6.2
Unit Weight, pcf	140.8	141.6	144.4	140.8

* Material prewieghed.

(1) After moisture content adjustments and additional water for slump adjustment.
(2) Sources of ingredients as given by Williams Brothers:
Cement: Blue Circle Atlanta Type K
Stone: Blue circle Lithonia #57
Sand 1: Dravo Waugh
Sand 2: Blue Circle Lithonia M-10
Admixture 1: W.R. Grace WRDA 79
Admixture 2: W.R.Grace Darex AEA

NY THRUWAY TYPE K MIX DESIGN

Table 2 Actual Mix Proportions per Cubic Yard from Batched Mixes				
Mix ID	500 AAEK	600 AAEK	715 AAEK	825 AAEK
Ingredients	Quantities in lbs/yard^3			
Cement	493	594	720	812
Stone (SSD)	1794	1824	1857	1815
Sand 1 (SSD)	988	887	791	666
Sand 2 (SSD)	248	223	198	167
Water	279	296	332	341
Admixture 1	20.0 oz.	23.7 oz.	28.8 oz.	35.0 oz.
Admixture 2	2.0 oz.	2.7 oz.	3.2 oz.	5.3 oz.

BLUE CIRCLE
COURSE AND FINE AGGREGATE EVALUATION
NEW YORK STATE THRUWAY AUTHORITY
FINE AGGREGATE ANALYSIS

Date:	04/13/93	Plant:	
Sand 1		Sand 2	
Supplier:	Dravo	Supplier:	Blue Circle
Location:	Waugh	Location:	Lithonia
Type:	Natural	Type:	M 10

Sand 1

Screen	Wt. Ret.	Total % Ret	Net % Ret	% Pass	ASTM % Pass
No. 4	11.1	2.1	2.1	97.9	95 to 100
No. 8	91.2	17.6	15.5	82.4	80 to 100
No. 16	169.7	32.8	15.2	67.2	50 to 85
No. 30	315	60.9	28.1	39.1	25 to 60
No. 50	460.9	89.1	28.2	10.9	10 to 30
No. 100	503.3	97.3	8.2	2.7	2 to 10
Pan	517.3		2.7		
	FM	3			
Washed 200:	NA	%		Organics #	1.0

Sand 2

Screen	Wt. Ret.	Total % Ret	Net % Ret	% Pass	ASTM % Pass
No. 4	8.1	1.6	1.6	98.4	95 to 100
No. 8	85.8	17.2	15.5	82.8	80 to 100
No. 16	179	35.8	18.6	64.2	50 to 85
No. 30	250.6	50.1	14.3	49.9	25 to 60
No. 50	333.8	66.8	16.6	33.2	10 to 30
No. 100	409.3	81.9	15.1	18.1	2 to 10
Pan	499.8		18.1		
	FM	2.5			
Washed 200:	NA			Organics #	1.0

Blend

	Sand 1	80.00%		Sand 2	20.00%
Screen	Total % Ret	Net % Ret	% Pass		ASTM % Pass
No. 4	2	2	98		95 to 100
No. 8	17.5	15.5	82.5		80 to 100
No. 16	33.4	15.9	66.6		50 to 85
No. 30	58.7	25.3	41.3		25 to 60
No. 50	84.6	25.9	15.4		10 to 30
No. 100	94.2	9.6	5.8		2 to 10
Pan		5.8			
	FM	108		Wash 200: N	%

This blend meets ASTM C-33 Gradation Specifications

COARSE AGGREGATE ANALYSIS

Date:	03/04/93	Supplier:	Blue Circle
Plant:	Glenwood	Location:	Lithonia
Sample Wt.:	18.4	Stone Size:	57

Sieve	Cumm. Wt. Ret.	Cumm. % Ret.	Total % Pass	ASTM & DOT Spec. Percent Passing
2"	0.0	0.0	100.0	100
*1 1/2	0.0	0.0	100.0	100
1"	0.0	0.0	100.0	95-100
* 3/4"	3.1	16.8	83.2	- - - - - - - -
1/2"	9.6	52.2	47.8	25-60
* 3/8"	12.9	70.1	29.9	- - - - - - - -
* No. 4	17.1	92.9	7.1	0-10
* No. 8	17.5	95.1	4.9	0-5
	F.M.	6.8		

* Included in coarse aggregate F.M. Series
Meets ASTM Gradation Specifications

BLUE CIRCLE

CONCRETE VERIFICATION TYPE K CEMENT
NEW YORK THRUWAY AUTHORITY

MATERIALS

Cement: Blue Circle Type K
Sand: Blend of Blue Circle Lithonia M-10 and Dravo Waugh
Stone: #57 Blue Circle Lithonia
Air Entraining Agent: W.R. Grace, Darex
Water Reducer: W.R. Grace, WRDA-79

Mix Proportions: S.S.D. Pounds per Cubic Yard

Cement	500	600	715	825
Sand	1252	1121	982	846
Stone	1818	1843	1843	1843
Water, Gallons	34.0	35.9	39.2	41.6
A.E.A., oz	2.0	2.7	3.2	5.4
Water Red., oz	20.0	24	28.6	33

Test Results - ACI 223 Mix Methods "B"

Slump, inches	5.0	5.0	4.75	5.0
Air %	6.0	6.0	5.0	6.2
W/C	0.57	0.50	0.46	0.42
Initial Set	4:20	4:25	4:25	4:00
Final Set	5:30	5:40	5:35	5:05
7 Day ASTM C878 Expansion %	0.030	0.038	0.051	0.054
7 Day, str, psi	3350	3600	4390	4790
14 Day, str, psi	4050	4300	5370	6240
28 Day, str, psi	4750	5530	6510	6770

TYPE K MIX DESIGNS
for Blue Circle Cement Co.
by Law Engineering Co.

TABLE 1

MIX PROPORTIONS PER CU. YD. AND MIX SUMMARY

Mix No.	1	2	3	4	5	6	7	8
Type K Cement, lbs.	750	750	750	750	750	750	750	900
Sand (SSD), lbs	1170	1190	1200	1215	1235	1245	2695	1095
#57 Gravel (SSD), lbs.	1800	1800	1800	-----	-----	-----	-----	-----
#8 Gravel (SSD), lbs.	-----	-----	-----	1660	1660	1660	-----	1660
Water, lbs.	375	353	308	368	368	345	473	369
Retarder (Hy-Kon 2000R)	0	30	15	0	30	15	15	15
Superplasticizer (Cormix 2000)	0	0	64	0	0	73	73	77
Water Cement Ratio	0.50	0.47	0.41	0.49	0.49	0.46	0.63	0.41
Slump, initial	9 1/2	9	7	9 1/2	9 1/2	8 1/2	8	7 3/4
Slump, @ 45 min.	7 3/4	7 1/2	4 1/2	7 3/4	7 1/2	4 1/2	5 1/2	4 1/2
Slump, w/superplasticizer	N/A	N/A	8 3/4	N/A	N/A	8 3/4	9	8 1/2
Temp., initial	78	78	78	77	76	74	78	78
Temp., @ 45 min.	81	79	80	81	76	78	81	82

Compressive Strength, psi

2 Days	2250	3540	4390	3010	3380	4060	2520	5100
3 Days	3340	4590	4780	3500	4150	4830	2810	5480
7 Days	4160	5220	5480	4600	5040	5500	3170	6420
14 Days	4660	5820	6220	5390	5630	6170	4020	6960
21 Days	5300	6230	6640	5770	6030	6490	4140	7340
28 Days	5420	6530	6630	6130	6150	6850	4530	7620
56 Days	6040	7040	7510	6640	6680	7280	4910	8320
90 Days	6810	7670	7860	7000	7270	8100	5190	8830
180 Days	7450	8580	9310	7920	7830	8730	5510	9580
275 Days	7790	8920	9580	8240	8040	9010	5680	10550
365 Days	7800	9300	9200	8400	8140	9680	5840	10520

APPENDIX E

Final Mix Designs

Table E1 Research Project Final Mix Designs

Mix	OO	OM	KK	KM
Coarse Aggregate (lb)	1777	1777	1777	1777
Fine Aggregate (lb)	1219	1219	1219	1219
Cement (lb)	700	630	700	630
Microsilica (lb)	0	70	0	70
Water (lb)	309	309	309	309
w/c Ratio	0.44	0.44	0.44	0.44
Coarse Aggregate Size No.	57	57	57	57
Slump (in.)	3	3	3	3
Entrained Air (% By Volume)	4-6	4-6	4-6	4-6
40 Degrees F				
Air Entrainer (ml)	192	240	192	224
Water Reducer (ml)	879	2256	2716	4873
72 Degrees F				
Air Entrainer (ml)	192	272	192	272
Water Reducer (ml)	799	2556	3196	4314
90 Degrees F				
Air Entrainer (ml)	192	272	192	272
Water Reducer (ml)	799	2237	3196	4314

Admixture amounts changed with temperature in an attempt to maintain consistent slump.

Weights given are for 1 cubic yard.

OO - Type I Portland Cement

OM - Type I Portland Cement with Microsilica

KK - Type K Cement

KM - Type K Cement with Microsilica