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SECONDARY REINFORCEMENT FOR FIBER REINFORCED POLYMERS REINFORCED CONCRETE PANELS

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OURE

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1. Introduction

1.1 Background

Today's automobile population is increasing like never before. Due to the economic growth in developing countries around the world, exponential growth is inevitable. This increase in automobiles will bring new demands to the existing and future infrastructure system around the world. Because of the enormous amount of money invested in infrastructure, it is important that the product be as durable and long-lasting as possible.

A major issue with today's infrastructure and particularly bridges is corrosion of steel reinforcement within the concrete deck due to chloride penetration from deicing salts. Chloride ions mixed with water and oxygen leach into the structure through surface cracks and react with exposed steel rebar. This reaction creates rust and produces an increase in the original steel volume that causes the reinforcement to de-bond from the concrete and in turns causes delamination of the bridge deck and loss of structural integrity. This corrosion problem reduces serviceability life and in turn costs billions in rehabilitation and/or replacement.

There are generally two answers to this dilemma. One is to limit crack widths. Limiting the crack widths will help prevent the penetration of water, oxygen, and chloride ions from deicing salts into the bridge deck. Although cracks cannot be totally eliminated they can be controlled with proper reinforcement design. The goal is to replace the larger, less numerous cracks that provide major exposure to reinforcement with smaller, more numerous cracks that restrict penetration.

The second solution is to find a suitable replacement for steel that will not corrode as easily and in turn will increase the serviceability life of the structure. Bridge deck deterioration shows that even epoxy-coated rebar, galvanized steel rebar, and cathode protection only delay corrosion instead of preventing the matter. One possible source for this replacement is fiber-reinforced polymers (FRP). FRP has been available for several decades and is advantageous because of its relatively lightweight and high-strength qualities. Even so, only since the 1990's has it been strongly considered as a replacement for steel reinforcement in highway bridge decks.

Several types of FRP exist, including glass (GFRP), carbon (CFRP), and aramid (AFRP). These different types of FRP all behave relatively similar, and have virtually the same advantages and disadvantages over steel reinforcement. The advantages and disadvantages of FRP reinforcement when compared to steel reinforcement are shown in Table 1 below. Looking at the comparison, FRP evidently seems to be a promising replacement for steel as reinforcement for concrete structures.

Table 1: Advantages and disadvantages of FRP over steel when used for concrete reinforcment

Advantages
Impervious to chloride ion and chemical attack
Tensile strength greater than steel
1/4th weight of steel reinforcement
Transparent to magnetic fields and radio frequencies
Electrically and thermally non-conductive
Disadvantages
Low modulus of elasticity
no yielding point before rupture
high initial cost
lack of familiarity within the industry

1.2 Scope and Objectives

This study deals with GFRP only and was performed in order to develop an empirical secondary reinforcement ratio for FRP based on experimental test data; more specifically to determine the optimum amount of GFRP to control cracking within a concrete slab. The current ACI guidelines for secondary reinforcement for FRP reinforced concrete are based on a steel reinforcement ratio of 0.0018 and account for the stiffness and strength of the FRP material by using a secondary reinforcement ration design equation. This design equation has no experimental data for backup and is considered excessive by many experts. Even the steel guidelines in ACI 318 for secondary reinforcement have little experimental data attributed to their basis. They are primarily the result of field observations over many years of structures that have yielded acceptable results. This research will hopefully aid the development of guidelines based on actual experimental validation of GFRP reinforced concrete specimens.

Currently, there is no standard test method to evaluate secondary reinforcement. Although previous research similar to this particular project has been attempted in the past, the mix design, specimen dimensions, and reinforcement ratio design combination are unique. Only Information on test setup and procedure from these studies was considered. The study is divided into two phases. Phase I will investigate the early-age effects of the various reinforcement ratios while Phase II deals with the later-age effects. Because data is still currently being taken on this project, only Phase I observations are contained in this report. Phase II observations will come at a later date.

2. Literature Review

2.1 Cracking

As previously stated, exposure of reinforcement to weathering elements must be limited in order to prevent corrosion. One proven method is eliminating crack potential because cracks present a direct unrestrained path to the reinforcement. Cracking in concrete can be attributed to many different factors but they all correlate back to induced tensile strains as a result of deformation of the concrete. The typical factors affecting cracking in concrete include plastic shrinkage, drying shrinkage, and thermal shrinkage and expansion.

Plastic shrinkage cracks form soon after placement but before curing when the concrete is still in its plastic state. They result most commonly when the evaporation rate of water from the surface of the concrete exceeds the bleed water rate to the surface from the bottom of the slab. When this occurs, negative capillary forces are produced and cause the paste volume of the concrete to contract. This contraction of the paste induces tensile stresses that the still fresh concrete cannot support and results in cracking. Plastic shrinkage cracks can be minimized by controlling the evaporation rate of water from the concrete surface.

Besides plastic shrinkage cracks, drying shrinkage cracks also present a problem for reinforcement corrosion. Drying shrinkage occurs during the curing process and is a result of internal water loss from the hardened cement paste as opposed to surface water loss. Factors that influence drying shrinkage cracks include cement composition, aggregate type, water content, and the mix proportions. Mix water left over from the hydration process is the main cause of drying shrinkage however. Adding no more water than just enough necessary to complete the hydration process will significantly aid with the prevention of drying shrinkage.

Thermal cracking is the result of deformations of the slab as a result of thermal gradients through its thickness. During a typical hot summer day the top surface of the concrete slab will become warm and expand as it is heated by solar radiation while the bottom surface remains cool and unchanged. This causes the slab to bow upward in a convex fashion. The opposite occurs at night when cool temperatures cause the top surface to contract. This slab deformation may be capable of producing stresses large enough to induce thermal related cracks. Also, while the slab is deformed it cannot adequately support traffic loads without cracking.

2.2 ACI 440

As discussed, most cracks form due to the high tensile stresses created by internal or external restraints produced by deformation of the concrete through shrinkage or temperature differentials. Using reinforcement will not eliminate cracks, and in some cases is thought to actually encourage cracking. However, the right design of reinforcement can distribute shrinkage strain along the bond of the bar and can produce several thin cracks instead of a few wide ones. This is significant since finer crack widths result in less durability problems. Crack widths for FRP have more tolerance than for steel since corrosion is not an issue. The optimum amount of FRP to obtain acceptable crack width must now be determined. ACI 440 uses Equation 2-1 below to govern secondary reinforcement design for FRP.

$$\rho_{f,ts} = 0.0018 \times \frac{60{,}000}{f_{fu}} \frac{E_s}{E_f} \leq 0.0036 \qquad \text{(Equation 2-1)}$$

Equation 10-1 (ACI Committee 440-03)

"Due to limited experience, it is recommended that the ratio of temperature and shrinkage reinforcement given by Eq. (10-1) be taken not less than 0.0014, the minimum value specified by ACI 318 for steel shrinkage and temperature reinforcement. Spacing of shrinkage and temperature FRP reinforcement should not exceed three times the slab thickness or 12 inches (300 mm), whichever is less" (ACI Committee 440-03).

Notice that the equation uses the secondary reinforcement ratio for steel as its primary basis and factors in the ratios of strength and stiffness of the two materials to account for the properties of FRP. A limit of 0.0036 is provided to limit excessive amounts of reinforcement due to the stiffness of steel being much greater than the stiffness of FRP.

2.3 Previous Research

Studies regarding temperature and drying shrinkage reinforcement are scarce but available. Papers discussing plastic shrinkage are more available. An extensive foundation for this particular experiment was laid by Daniel Koenigsfeld and his work at the University of Missouri – Rolla with Dr. John

Myers, P.E. This project is an addition onto advancements already proven through his research. His work was considered during the design of the test specimen and the testing procedure.

3. Experimental Program

To simulate FRP reinforcement within a bridge structure a model having a similar behavior was designed. A total of five model bridge deck spans were cast; four of the spans containing different reinforcement ratios of GFRP reinforcement. A wide range of ratio values was implemented into the design. The fifth span was the control and contained steel reinforcement with the minimum allowed secondary reinforcement ratio. Each span's specific reinforcement design is shown in Table 2 below. The spans were joined at the end-sections in order to better control shrinkage during the experiment.

Table 2: Reinforcment design of individual spans

Slab #	Reinforcement Type	Bar #	Size of Bar	Number of Bars	Area of Bar	Total Area	Reinforcement Ratio
1	Steel	#3	0.375	2	0.11	0.22	0.0018
2	GFRP	#3	0.375	2	0.1307	0.2614	0.0022
3	GFRP	#3	0.375	3	0.1307	0.3921	0.0033
4	GFRP	#3	0.375	4	0.1307	0.5228	0.0044
5	GFRP	#4	0.5	3	0.2245	0.6735	0.0056

The five spans were eight feet long and two feet wide and were separated by a gap of three inches. Each slab thickness was four inches. Both the end sections of the specimen were four feet long, had a total width of eleven feet, and a thickness of eleven inches.

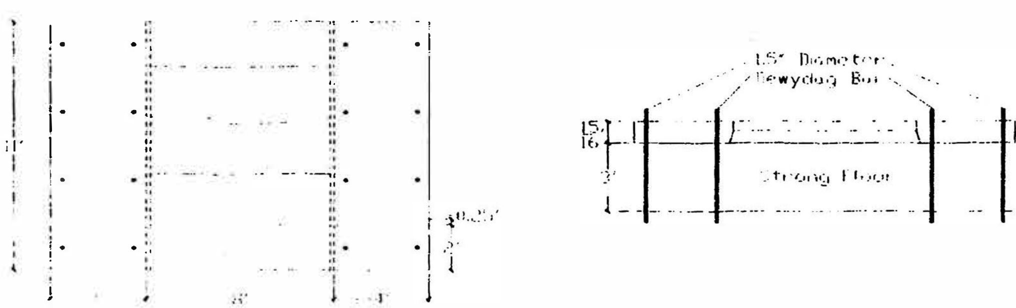
Throughout the testing period it was important to ensure that no shrinkage was allowed and that all deformation developed as cracks within the structure.

To account for this, eight tie rods were placed in each end section, extending through the entire thickness of the structure and on beneath the strong floor of the high-bay lab. The tie rods were tightened by a calibrated torque wrench with enough force to ensure no shrinkage but gently enough to avoid breaking up the concrete at the pressure point. Calculations for the torque wrench calibration can be found in the appendix. Figure 1 is a plan and side view of the testing specimen and may give a clearer picture of the design.

Figure 1: Plan and side view of specimen

Plan view of specimen

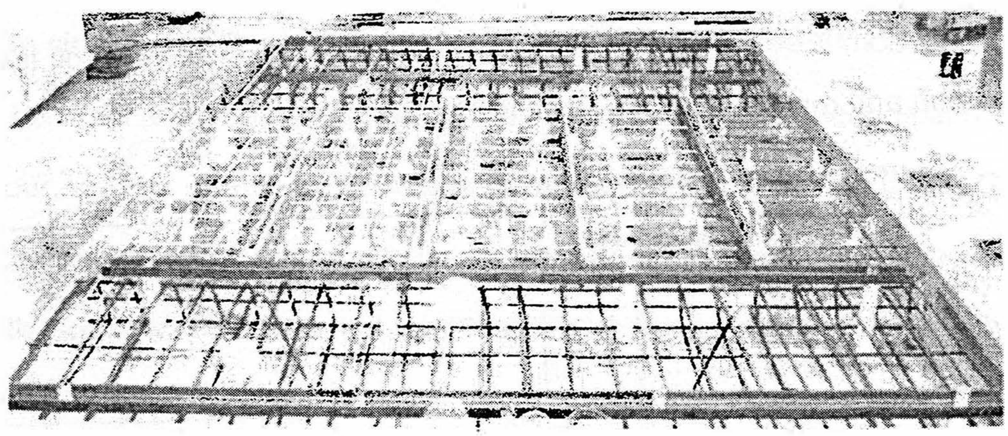
Side view of specimen



The concrete forms were entirely of wood fastened together with wood screws. It was manually manufactured within the high-bay lab. The reinforcement was held in place using three inch chairs. The chairs were nailed to the bottom support form to prevent movement during concrete placement. Reinforcement spacing was figured by evenly dividing the slab width by the number of reinforcing bars. Minimum concrete cover on the reinforcement was one inch. The reinforcement design can be seen in Figure 2 below.

Four strain gauges were located in each slab on the longitudinal reinforcing bars. This was done in order to verify throughout the study that the concrete specimen was inducing shrinkage strain on the structure. Three strain gauges per slab would be sufficient to obtain the desired readings, however the fourth strain gauge per slab was added for the possibility of damaging one during the concrete placement.

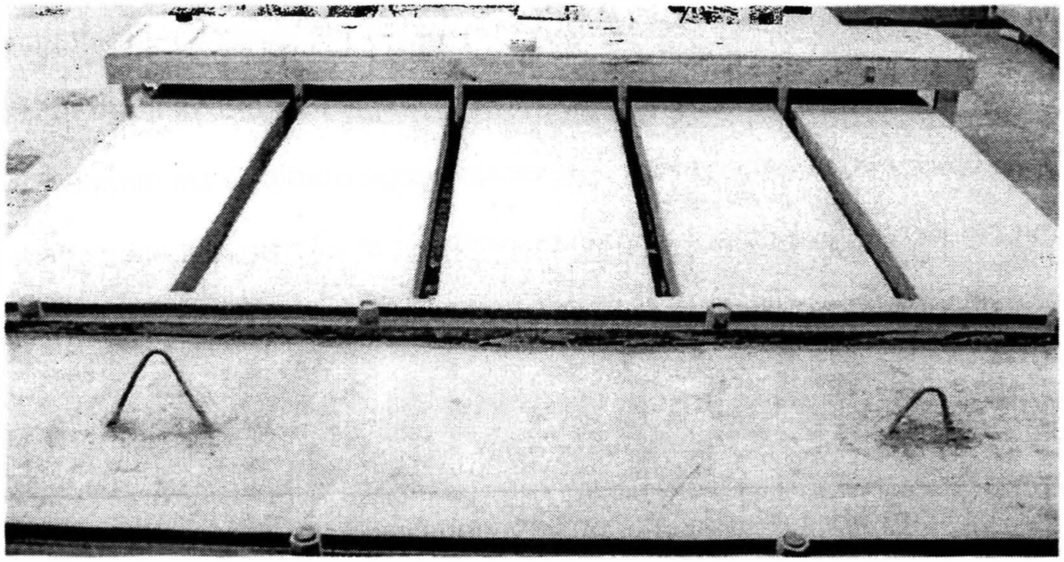
Figure 2: Reinforcement design



The concrete mix was a typical Missouri Department of Transportation bridge deck mix design. The specific design can be found in the appendix. The specimen took four men nearly three hours to cast. Cylinders and beams were cast along with the structure in order to perform strength tests throughout the duration of the experiment.

A few obstacles were encountered during and after the placement of the concrete and should be noted. As predicted above, the first problem during the placement was the damage inflicted on the two strain gauges in the spans. This most likely was the result from contact with the consolidation vibrator. Another problem encountered was the poor consolidation of some of the cylinders being cast for strength tests later on. This error is the cause of low strengths for two cylinders. Other than these small occurrences the casting of the structure went well. A picture of the cast specimen is shown below in Figure 3.

Figure 3: Cast specimen



After the specimen was cast and sufficient strength was achieved the tie bars were tightened down. Because of the intricate design of the forms, their removal was not completed until several days after the placement. Because this study depends on shrinkage effects to determine the significance of the reinforcement and because late removal of forms can prevent shrinkage, this could prove to be the most critical error. As of now, no shrinkage cracks have formed in the specimen.

4. Experimental Results

Phase I which involves the short term observations of the experiment is complete and somewhat disappointing. To date, the only experimental results gathered for Phase I are the strength breaks of the cylinders through twenty-eight days and strain readings on day 14. Data concerning cylinder breaks and strain readings can be found in the appendix. Since the mix design for the concrete is unique, no comparison can be made to ensure the quality of the concrete. The only assurance that the concrete is acceptable is that the strength gaining curve (see appendix) seems typical. The strain gauge readings are disappointingly low but are logical given the fact that no shrinkage cracks have formed. More results and observations will follow in Phase II of the study.

5. Summary

Phase I or the early-age observations of this project verified little from the results gathered to date. The concrete quality control seems to be adequate as determined from the cylinder breaks. It can be gathered that inadequate restraint of the structure was the result of no formation of shrinkage cracks. This could be accounted to the long amount of time required for form removal or an error in the tie rod tensioning in the end section of the structure. Phase II, later-age observations, will continue where Phase I ends and will hopefully encounter developments in cracking. Because cracking is critical to the basis of this project, if no self induced cracking occurs outside measures may need to be taken to induce cracking of the structure. These measures may include heating, wetting and drying, or loading the specimen. Again Phase II will determine what measures will be taken.

References

Koenigsfeld, Daniel. “Secondary Reinforcement for Fiber Reinforced Polymers Reinforced Concrete Panels.” 2003

Mahmood, Hamid. “Cracking of Concrete Members Reinforced with Glass Fibre Reinforced Polymer Bars.” 2002

Appendix

- Torque wrench data
- Strain gauge data
- Cylinder break data
- Mix design specifications

Torque Wrench Calculations

torque on wrench	Load Cell Measurements			
	Trail 1	Trail 2	Trail 3	Trail 4
0	0	0	0	0
10	38.90454	55.88107	45.97809	46.92124
15	55.17371	69.32082	56.58842	60.36099
20	69.32082	85.58999	69.32082	74.74388
25	82.76057	102.5665	80.6385	88.6552
30	98.32239	119.543	98.32239	105.3959
35	108.9327	130.1534	118.1283	119.0715
40	123.7872	142.8858	135.1049	133.9259
45	147.1299	161.277	154.9108	154.4392
50	171.18	176.8388	170.4726	172.8305
55	181.083	194.5227	197.3521	190.9859
60	196.6448	218.5728	219.2801	211.4992
65	215.7434	233.4272	232.0125	227.0611
70	232.7199	248.9891	252.5258	244.7449
75	258.1847	265.2582	264.5509	262.6646
80	273.0391	277.2833	286.4789	278.9338
85	296.3819	301.3334	310.529	302.7481
90	308.4069	320.432	321.8467	316.8952
95	326.0908	340.9453	343.7747	336.9369
100	344.482	364.288	358.6291	355.7997

$y = 3.4378x - 3.2091$
$x = (y + 3.2091) / 3.4378$

3 day

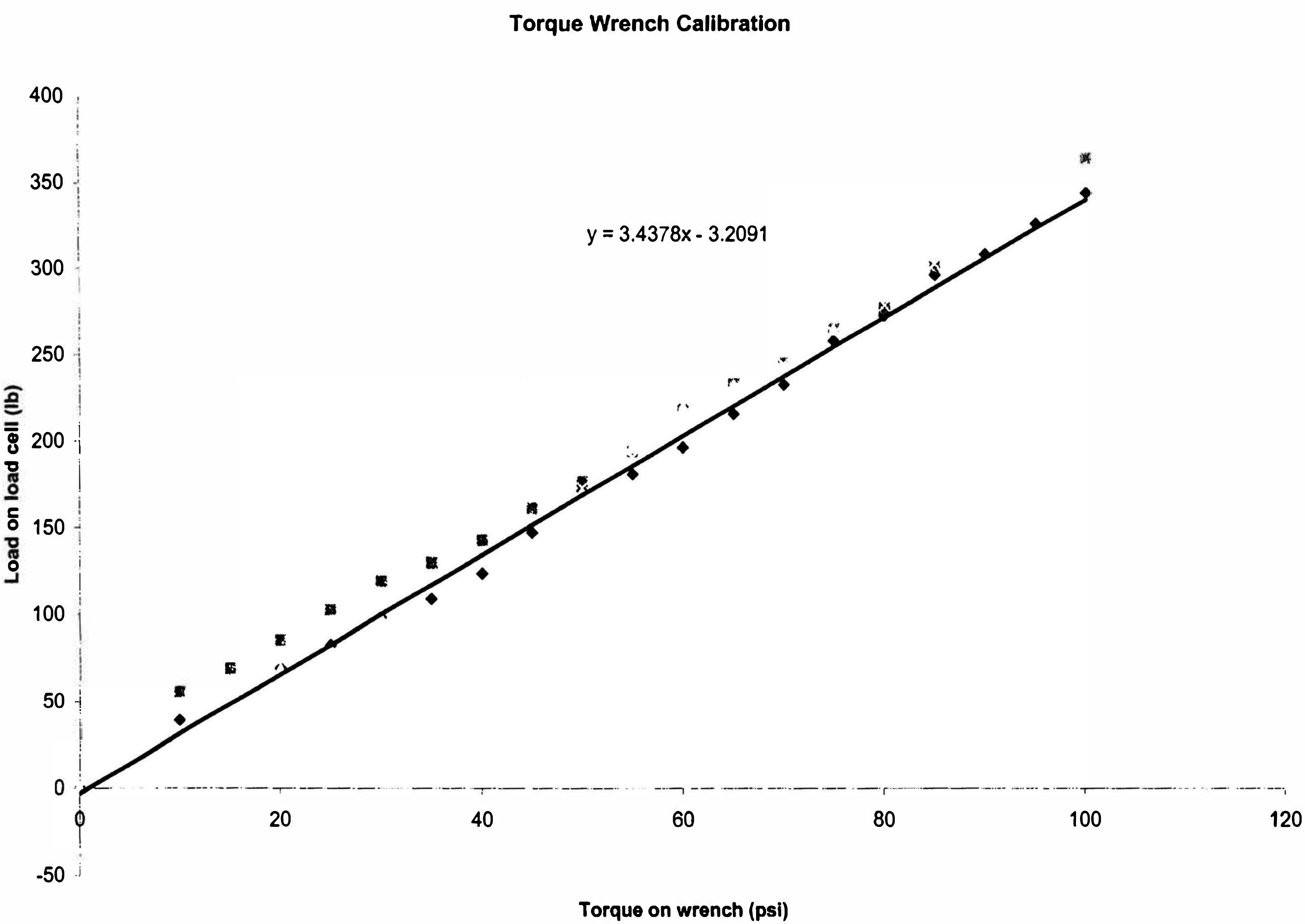
Compressive Strength =	4,090	psi
Tensile Strength =	409	psi
Torque to =	120	ft-lbs

7 day

Compressive Strength =	5,146	psi
Tensile Strength =	515	psi
Torque to =	151	ft-lbs

14 day

Compressive Strength =	5,970	psi
Tensile Strength =	597	psi
Torque to =	175	ft-lbs


Torque Wrench Calibration
y = 3.4378x - 3.2091
Load on load cell (lb)
400
350
300
250
200
150
100
50
0
-50
0
20
40
60
80
100
120
Torque on wrench (psi)

Strain Gauge Values

Strain Gauge ID #	14 Day Micro Strain
1 - 1	1238
1 - 2	1116
1 - 3	---
1 - 4	642
2 - 1	1238
2 - 2	1004
2 - 3	552
2 - 4	606
3 - 1	361
3 - 2	409
3 - 3	101
3 - 4	824
4 - 1	1009
4 - 2	1000
4 - 3	1075
4 - 4	1546
5 - 1	1902
5 - 2	1368
5 - 3	1196
5 - 4	---

Cylinder Break Data

Diameter (inches)

	Day 1		Day 3		Day 7		Day 14		Day 28	
	Diameter #1	4.000	Diameter #1	3.918	Diameter #1	3.929	Diameter #1	3.919	Diameter #1	3.923
Cylinder #1	Diameter #2	4.000	Diameter #2	3.905	Diameter #2	3.919	Diameter #2	3.920	Diameter #2	3.923
	Avg.Diameter	4.000	Avg.Diameter	3.912	Avg.Diameter	3.924	Avg.Diameter	3.920	Avg.Diameter	3.923
	Diameter #1	4.000	Diameter #1	3.922	Diameter #1	3.924	Diameter #1	3.919	Diameter #1	3.920
Cylinder #2	Diameter #2	4.000	Diameter #2	3.930	Diameter #2	3.915	Diameter #2	3.840	Diameter #2	3.924
	Avg.Diameter	4.000	Avg.Diameter	3.926	Avg.Diameter	3.920	Avg.Diameter	3.880	Avg.Diameter	3.922
	Diameter #1	4.000	Diameter #1	3.931	Diameter #1	3.925	Diameter #1	3.928	Diameter #1	3.926
Cylinder #3	Diameter #2	4.000	Diameter #2	3.914	Diameter #2	3.918	Diameter #2	3.924	Diameter #2	3.926
	Avg.Diameter	4.000	Avg.Diameter	3.923	Avg.Diameter	3.922	Avg.Diameter	3.926	Avg.Diameter	3.926

Ultimate Compressive Strength (lbs)

	Day 1	Day 3	Day 7	Day 14	Day 28
Cylinder #1	28,230	48,135	64,350	71,820	72,345
Cylinder #2	29,715	50,955	59,190	70,785	81,615
Cylinder #3	29,475	49,005	62,940	66,390	78,660

Ultimate Compressive Stress (psi)

	Day 1	Day 3	Day 7	Day 14	Day 28
Beam #1	2,246	4,006	5,321	5,952	5,985
Beam #2	2,365	4,209	4,906	5,988	6,756
Beam #3	2,346	4,055	5,211	5,484	6,498
AVG =	**2,319**	**4,090**	**5,146**	**5,970**	**6,627**

Concrete Mix Design

-----Original Message-----
From: Myers, John (UMR) [mailto:jmyers@umr.edu]
Sent: Thursday, October 21, 2004 3:18 PM
To: Branham, Nathan Dale (UMR-Student); Cox, Jason
Subject: Mix Design
Importance: High

Here is a suggested mix design that is representative of MoDOT bridge decks:

Coarse Aggregate (3/4" max or smaller preferred; limestone) 1783 #/cy
Fine Aggregate (Natural river sand preferred) 1074 #/cy
Cement 728 #/cy
Water 320 #/cy
Add air entrainment to get ~ 5% air; perhaps ~ 3.3 oz/cy

Use enough water to attain target of 4" slump at placement. Perhaps less than 320 #/cy is needed to get the target slump so **do not** have the ready mix producer add all of the water at once unless he is checking the slump at the plant. Weight/measure any additional water added in the lab to make sure we have a very accurate w/c ratio for the report. Obtain the plant ticket and any material mil sheets they might have.

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