LIGHTWEIGHT AGGREGATE IN PRESTRESSED CONCRETE

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

INTRODUCTION

If one had to write on the subject of "Lightweight Aggregate in Prestressed Concrete" ten years ago, he had to write his own observations and to record the results of his own experiments, since up to that time almost nobody had written anything on the subject.

Prestressing applied on ordinary (gravel and sand) concrete has been practiced for structural use to a limited extent since the dawn of the century. The fact that prestressing required a concrete of high compressive strength made it obvious that concrete of first class gravel and pure sand would be used.

At the same time while prestressing ordinary (heavy weight) concrete was being used in structures, there was another kind of concrete, namely lightweight aggregate concrete, invading the market as a new structural material. But this lightweight aggregate concrete did not become accepted very quickly as a real competitor of the heavy aggregate concrete in the rapidly developing prestressing field. Yet, the curiosity of some engineers and architects made it possible for this new aggregate to be introduced to the prestressing yards to test its ability to stand prestressing.

The dreams of these few pioneers were not disappointed, and the lightweight aggregate concrete passed the test and has proved to be a promising material for use in prestressed structural members.

In the following pages some of the structural qualities, the advantages and the deficiencies of lightweight aggregate in prestressed concrete will be indicated.

The author would like to point out, at this first step in this report, that he did not run any experiments or tests to furnish new data and information. All that he did was to review all of the literature which he could obtain that dealt with the subject. So, all of the original credit goes to those who ran the tests, performed the experiments, and published their results and observations without which it would have been impossible to write this report.

CHAPTER I

HISTORICAL REVIEW

It is believed that the Romans were the first to use lightweight aggregate since some of their main buildings contained large pieces of pumice.

Slag was used in Germany in 1822; it was introduced as a concrete aggregate in the United States around the year 1890.

Cinders from coal burning furnaces were used in some industrial areas in this country early in this century.

In 1917, the process of producing expanded shale was perfected by Stephen Hayde. At the same time Mr. Wig, a Marine Engineer, was conducting research on the possibility of building ships, badly needed because of the First World War, from reinforced concrete which should be exceptionally light. In 1918, and after corresponding with Mr. Hayde, it was possible for Mr. Wig to produce enough expanded shale aggregate to build the 3000-ton Atlantus Ship. This was done in Alabama. Meanwhile, the rotary kiln as a better producing method was introduced.

The first patent (the Haydite Patent) to produce lightweight aggregate from bloated clay and shale was granted in 1918.

Cellular or foam concrete has been developed and mostly used in Europe.

There are two important factors which accelerated the development and use of lightweight aggregate.

The first goes back to the end of the 19th Century when there was a revolutionary change in building design and construction, by introducing structural steel and concrete as the main structural materials. Some of the existing skyscrapers and long span bridges owe their existence in the first place to the lightweight aggregate concrete.

The second factor has been brought up during the first world war when there was a shortage of steel. The minds of some of the ship designers focused on lightweight aggregate concrete as the best substitute for steel to build their ships.

The very many good merits of lightweight aggregate concrete felt by designers and construction men made them have more interest in it, and tempted some of them to do more research and to run many experiments to reveal more good qualities, if any, in that baby material of construction. During the last twenty five years, many new lightweight aggregates were put on the market such as: pumice, vermiculite, permite, denilite (16-b)*, pozzolith (16-c), permalite, and idealite (16-a).

It is only during the last decade that prestressing was applied on lightweight concrete and it proved to be favorable.

Adrian Pauw and R. L. Reid(11) presented a paper on "Lightweight Prefabricated Joint Slab-Beams of Prestressed Concrete" at the First United States Conference on Prestressed Concrete in Cambridge, Massachusettes, August, 1951.

Fred E. Koebel (6) presented a paper on "Lightweight Prestressed Concrete (Using Expanded Shale)", at the Sixth Regional Meeting, Houston, Texas, October 30, 1953.

The A.C.I. - Journal of June, 1955, published a paper by Arthur M. James

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*Number in paranthesis refers to the number of reference in the Bibliography.

(3) under the title "Precast Prestressed Lightweight Concrete Construction". In this paper, Mr. James described two jobs which have been actually constructed using precast prestressed lightweight concrete beams.

These are some of the developments in the use of lightweight aggregate in prestressed concrete during the last decade.

CHAPTER II

CHARACTERISTICS OF LIGHTWEIGHT AGGREGATE FOR STRUCTURAL CONCRETE

Lightweight concrete can be produced by one of the three methods:

(i) adding air to the cement paste

(ii) the use of lightweight aggregates

(iii) a combination of (i) and (ii).

The emphasis of this report will be on the lightweight aggregate concrete.

There are a variety of materials which can be processed into lightweight aggregates; but the ones which haveshowed the best structural values are:

(i) expanded shales

(ii) expanded clays.

These materials are sometimes referred to as "Haydites".

They are produced by burning raw shale or clay in a rotary kiln at 2000 degrees fahrenheit. Then they are crushed, screened into commercial separate sizes (usually two or three), and stored. When these aggregates are ready for mixing extreme care should be taken in grading these materials since they have a high degree of angularity. Because of this angularity, higher percentage of fines is usually required to produce a workable mix.

Some of the most favorable properties of lightweight aggregates are:

(i) Low Density - This is the basic and the most important property of these aggregates. It cuts down the unit weight of concrete from 150 pounds per cubic foot (conventional concrete) to 100-115 pounds per cubic foot (lightweight concrete). It adds to the economy of construction since smaller footings, shallower sections, and longer spans are possible. This has also an economical advantage in hauling precast members of lightweight concrete.

(ii) Insulation - Lightweight aggregate concrete has better insulating properties against heat and sound than conventional concrete. On the other hand, lightweight aggregate concrete has some unfavorable properties such as: (i) low modulus of elasticity - this is the most unfavorable property of this type of concrete especially when the concrete is to be of the prestressed type. The modulus of elasticity of lightweight concrete is about 50-80% of that of the ordinary concrete. The immediate results of low "E" are: more deflection of the member and hence less rigidity, and more loss in prestress. The loss in prestress in ordinary concrete is in the range of 15-20% while in lightweight concrete, it is in the range of 20-30%. (ii) Segregation and high absorption - lightweight aggregates segregate very easily; proper control and constant checking is imperative. Also, these aggregates have a high absorption for water. This property affect the effective water cement ratio. Prewetting or presoaking helps reduce the tendency of these aggregates to absorb water.

The grading of lightweight aggregates changes from one producing company to the other. But in general, the table below gives average values from 28 different companies (8).

SIEVE NO.	3/8	4	8	16	30	50	100
% RETAINED	0.5	21.0	25.5	17	11.5	9	6.5

As has been pointed out before, the coarse and fine aggregates have to be stored separately. They will be mixed on the site a few minutes before the mixing of the concrete ingredients.

Experience on many jobs indicated a ratio of fine to total aggregate of 60-70% (by volume).

For the cement aggregate ratio, the Brick and Clay Record suggests the values of 1:6 to 1:9. But it should be kept in mind that this ratio depends on the type of aggregate used and on the required strength of the concrete prepared.

For the water cement ratio there has not been any value set; each bid should indicate this ratio independently.

In general, it can be said that the mix proportioning is a matter of trial and error.

Summing up, the following remarks are worth restating:

(i) Lightweight aggregate concrete has advantages and disadvantages. Having these in mind, it will not be too difficult to decide whether to use this type of concrete on a given job or not.

(ii) Careful control and constant checking of all properties of the mix is of extreme importance.

(iii) The peculiar behavior and the individuality of each mix makes it difficult to set general specifications. Judgement plays a role in solving each individual problem.

CHAPTER III

GENERAL PROPERTIES OF LIGHTWEIGHT PRESTRESSED CONCRETE (4)

The basic physical properties which should be studied in the design of prestressed concrete are: the modulus of elasticity, the compressive strength, shrinkage and creep, and the loss of prestress.

The Modulus of Elasticity

The method of test makes a great difference in the value of the modulus of elasticity.

It has been found that, for a first class concrete suitable for prestressing, the modulus of elasticity of lightweight concrete is almost half that of ordinary concrete of the same quality. This low value of "E" of the lightweight concrete is its most serious deficiency. So, it is not advisable to use lightweight concrete in pre-tensioned members. In post-tensioned members, no serious results are expected. See Tables (2) and (3).

Compressive Strength

Tests showed that most of the expanded shale and clay aggregates produced in the United States have enough compressive strength to be used in prestressed concrete structures.

It is only a matter of cement factor that is required to produce the necessary compressive strength called for by prestress concrete specifications (see Table (4)).

Creep and Shrinkage

Creep of concrete is defined as "The inelastic deformation which occurs as time goes on due to the loads applied".

Shrinkage, on the other hand, is defined as "The contraction of concrete due to drying and chemical changes. It is a function of time alone and it has nothing to do with the loads applied".

High temperature and low humidity tend to increase the creep and shrinkage factors. Water-cement ratio and every variable in the concrete mix have an appreciable effect on creep and shrinkage values. The mineral composition and size of aggregates were found to have some effect on the increase or decrease of shrinkage values.

The Bureau of Public Roads suggested that in case of lightweight prestressed concrete, the allowance for creep and shrinkage should be increased by 50%.

Batching and handling of lightweight aggregate should be supervised carefully since honeycombing increases the creep tremendously.

Proper and steady curing is very essential in lightweight prestressed concrete.

Grading of aggregates is important. More fines means always an increase in the creep and shrinkage factors.

An excess of cement paste and water tends to increase creep and shrinkage values.

In this respect, it is worth mentioning that 5% to 7% of entrained air will make the concrete mix workable instead of adding more cement or more water to achieve workability.

Guyon, in his book, Prestressed Concrete, page 62, uses the following formula for the variation of creep in concrete with time:

$$\text{PERCENT CREEP} = 100\left(1 - 10^{-\sqrt{\frac{m}{4}}}\right)$$

For example, after one month, we have a creep of 44% of the total creep. And when m = 60 months (five years), we have a creep of 99% of the total possible creep.

The above formula can be used in case of creep in steel but "m" should be in days rather than in months.

The shrinkage of ordinary concrete ranges from .03% to .08%; while the shrinkage of lightweight concrete ranges from .04% to .3%.

Loss of Prestress

Many investigators found that the loss of prestress in lightweight concrete is less than they anticipated.

In the University of Michigan (1955), it was found by N. V. Campomanes that the loss of prestress in lightweight concrete was 21% to 22% while the loss in ordinary concrete was 16%.

The Freyssinet Company uses the following formula in evaluating loss of prestress at any point "x" along the member when post-tensioning is used:

$$T_o = T_x\, e^{(k_x + f\alpha)}$$

$$Tav. \doteq T_x \frac{e^{(k_x + f\alpha)} - 1}{k_x + f\alpha}$$

where: T_o = unit stress at the jack (psi)

T_x = unit stress at x distance from the jack (psi)

Tav. = average unit stress (psi)

k = a constant depending on the straightness of the duct in the beam.

f = the coefficient of friction between the duct and the tendon

α = the change in direction between the jack and the point "x".

TABLE 1

EXPANDED CLAY AND EXPANDED SHALE AGGREGATE GROUP CONCRETE MIX DATA

Batch No.	Agg. Vol. Ratio CA:FA	Quantities per c. y. concrete Type I Cement				Air Content %	Slump In.	Mixing Time Min.	Initial Unit Wt. Lb./c.f.	Aggregate Data			
		Sacks	Lb.	Total Aggreg. Lb. (Dry)	Total Water (Lb.)					Moisture Content % (Dry Wt.)	Finness Modulus No.	Pozzolanic Fines % (Dry Wt.)	Lb.
T-15	2:1	4.01	377	2058	635	5.0	2	10	113.5	18.1	4.68	9.6	198
T-16	1:1	3.93	369	1966	666	5.0	2	10	111.0	21.0	4.14	11.0	216
T-17	1:2	3.84	361	2032	666	4.5	2	10	113.5	19.8	3.84	11.4	232
T-18	2:1	5.59	525	1909	644	5.0	2	10	114.0	19.0	5.04	6.4	122
T-19	1:1	5.70	536	1883	635	5.1	2	10	113.0	16.3	4.10	10.0	188
T-20	1:2	5.79	544	1871	634	5.2	2	10	113.0	16.4	3.82	10.1	188
T-21	2:1	7.69	723	1801	609	4.3	2	10	116.0	15.0	4.92	7.5	135
T-22	1:1	7.52	706	1730	593	5.9	2	10	112.0	15.5	4.00	10.5	181
T-23	1:2	7.49	704	1756	621	5.5	2	10	114.0	14.0	3.82	9.7	170
D-15	1:1	5.41	508	1550	588	7.0	$\frac{1}{2}$	15	98.0	15.4	3.96		
D-16	1:1	5.83	548	1541	575	6.6	$2\frac{1}{4}$	15	99.0	13.9	3.96		
D-17	1:1	5.80	545	1443	595	7.5	5	15	96.0	14.8	3.96		
D-18	1:1	5.77	542	1565	542	7.2	$\frac{1}{2}$	9	99.0	12.4	3.96		
D-19	1:1	5.67	533	1508	573	7.2	2	9	97.3	11.5	4.26		
D-20	1:1	5.41	509	1514	585	7.2	5	9	97.0	8.9	4.26		
D-21	1:1	5.62	528	1533	546	7.9	$\frac{1}{2}$	3	97.3	9.1	4.26		
D-22	1:1	5.82	547	1505	593	7.5	2	3	98.5	13.0	4.26		
D-23	1:1	5.68	533	1529	610	6.6	5	3	99.0	11.9	4.26		

ST: Stands for Clay Aggregates

D: Stands for Shale Aggregates

TABLE 2

EXPANDED CLAY AGGREGATE GROUP STATIC MODULUS OF ELASTICITY ($E_c \times 10^{-6}$ psi)

Batch Design	Storage	3 Day	7 Day	14 Day	28 Day	42 Day	60 Day	120 Day	180 Day	3
ST-15	Wet	1.24	1.67	2.04	2.07	2.59	2.22	2.18	2.14	2
	Field			1.83	2.13	2.24	1.82	1.83	1.86	1
-16	Wet	1.42	1.80	2.19	2.31	2.88	2.21	2.29	2.31	2
	Field			2.03	1.93	1.82	1.88	1.71	1.89	1
-17	Wet	1.27	1.71	1.83	2.04	2.21	2.23	2.33	2.22	2
	Field			1.77	1.87	1.80	1.93	1.88	1.98	1
-18	Wet	1.64	2.09	2.14	2.51	2.31	2.75	2.62	2.33	2
	Field			1.90	1.96	1.93	2.25	2.00	2.11	1
-19	Wet	1.50	1.88	2.20	2.21	2.69	2.44	2.27	2.53	2
	Field			2.00	2.12	2.00	2.15	2.25	2.22	2
-20	Wet	1.44	1.82	2.10	2.27	2.27	2.50	2.42	2.31	2
	Field			2.07	2.13	2.05	2.33	2.19	2.33	2
-21	Wet	1.67	2.13	2.31	2.50	2.70	2.56	2.37	2.86	2
	Field			2.21	2.19	2.10	2.33	2.27	2.35	2
-22	Wet	1.47	2.00	2.38	2.31	2.55	2.71	2.92	2.75	2
	Field			2.18	2.31	2.33	2.60	2.56	2.24	2
-23	Wet	1.86	1.98	2.38	2.64	2.49	2.63	2.58	2.55	2
	Field			2.25	2.16	2.40	2.33	2.20	2.29	2

DYNAMIC MODULUS OF ELASTICITY IN FLEXURE

ST-22	Wet	-	-	-	2.40	2.53	2.17	3.09	3.24	3
	Field	-	-	-	2.46	2.46	2.71	2.00	2.08	2
-23	Wet	1.80	2.40	2.53	2.50	2.73	2.75	2.85	2.82	3
	Field	-	-	2.46	2.31	2.62	2.43	1.88	1.68	2

TABLE 3

EXPANDED SHALE AGGREGATE GROUP

DYNAMIC MODULUS OF ELASTICITY IN FLEXURE $E_c \times 10^{-6}$ psi

ASTM METHOD - C215 - 55T

Batch Design	Storage	3 Day	7 Day	14 Day	28 Day	42 Day	60 Day	120 Day	180 Day
D-15	Wet	2.12	2.20	2.44	2.47	2.56	2.63	2.73	2.81
	Dry			2.38	2.20	2.44	2.38	2.02	2.25
	Field			2.44	2.23	2.36	2.06	1.65	1.18
-16		1.89	2.19	2.35	2.28	2.54	2.48	2.65	2.64
				2.09	2.23	2.16	2.19	2.27	2.11
				2.11	2.22	1.91	1.59	1.44	1.31
-17		1.81	1.99	2.12	2.24	2.41	2.58	2.59	2.53
				2.10	2.16	2.13	2.20	2.16	1.67
				2.11	2.07	2.00	1.90	2.08	1.65
-18		2.14	2.39	2.52	2.93	2.79	2.88	2.35	2.96
				2.34	2.58	2.44	2.21	1.78	1.82
				2.34	2.33	1.85	2.02	1.27	2.08
-19		1.90	2.35	2.46	2.56	2.65	2.72	2.54	2.39
				2.42	2.23	2.25	1.96	1.96	2.08
				2.42	1.83	1.73	1.38	1.54	1.62
-20		1.92	1.97	2.22	2.34	2.45	2.40	2.36	2.28
				2.20	2.10	2.00	1.57	2.00	1.97
				2.07	1.73	1.08	1.95	1.67	1.85
-21		2.26	2.39	2.47	2.65	2.70	2.76	2.68	2.26
				2.33	2.37	2.34	2.46	2.35	1.73
				2.04	1.93	2.00	1.97	2.04	1.76
-22		2.04	2.23	-	2.53	2.45	2.52	2.55	2.56
				-	2.08	1.96	2.07	1.92	2.04
				-	1.80	1.56	1.74	2.23	1.99
-23		1.86	2.07	2.21	2.28	2.43	2.19	2.33	-
				2.19	2.14	2.00	2.09	1.03	-
				2.12	1.90	2.15	1.82	2.14	-

TABLE 4

CONCRETE COMPRESSIVE STRENGTH IN PSI (ASTM METHOD C116 - 49)

atch esign	Storage	3 Day	7 Day	14 Day	28 Day	42 Day	60 Day	120 Day	180 Day	365 Day
T-15	Wet	2010	3540	5050	5690	6070	6230	6050	6130	6080
	Field			4920	6260	6250	6510	6090	6140	6130
-16	Wet	2550	4030	5340	6220	5620	6080	4790	6180	6070
	Field			5640	6480	7050	6650	5120	6190	6440
-17	Wet	2200	4020	4950	5310	5210	5340	5840	5390	5760
	Field			4630	5370	5390	5370	5900	5940	6000
-18	Wet	4210	5730	6640	7400	7080	7440	6860	7430	7470
	Field			6830	8110	8030	7350	7700	7490	7850
-19	Wet	3030	5490	6330	7210	7800	7660	7300	7230	7720
	Field			6030	6950	7950	8120	7620	8330	7560
-20	Wet	3100	4640	5400	6340	6600	6330	6460	7280	7850
	Field			5580	6800	7030	7240	7270	7670	7040
-21	Wet	4510	6350	7290	7440	7680	7800	7280	7940	7940
	Field			6700	7460	7770	8000	7510	7690	7620
-22	Wet	3220	4920	6000	6070	7920	7650	8460	8310	7290
	Field			5210	5150	6390	7820	8640	8440	7370
-23	Wet	3800	4840	6060	7520	6950	7150	6730	7140	7150
	Field			5220	7050	7140	6990	7510	7320	7430
D-15	Wet	2790	4230	5050	5210	5250	6231	6200	6190	
	Dry			4790	4620	5310	5630	5620	6020	
	Field			4700	4980	5480	5770	5840	5920	
-16	Wet	2330	3200	3990	4300	4660	4730	5530	5640	
	Dry			3500	3860	4340	4540	4900	5510	
	Field			3670	4180	5320	5320	5450	5780	
-17	Wet	1870	2510	2930	3430	4330	4310	4640	4610	
	Dry			3080	3500	4010	3760	4340	5060	
	Field			3010	4110	4270	3700	5090	4930	
-18	Wet	2550	3570	4270	5250	5760	5440	5930	4780	
	Dry			4200	4820	5570	5710	5790	4770	
	Field			4460	4770	5340	5560	5860	4930	
-19	Wet	2490	3500	3690	4480	4720	4580	4200	4530	
	Dry			3850	4610	4830	4920	4930	5040	
	Field			3660	4490	5030	4860	4540	4260	
-20	Wet	1720	2400	3190	3520	4190	4010	4580	4130	
	Dry			3180	4000	4190	4280	4280	4080	
	Field			3160	4460	4430	3650	4670	4150	
-21	Wet	2540	3660	4730	4950	5010	5300	4630	5160	
	Dry			3850	4490	5420	5430	5130	5920	
	Field			4910	4790	5210	5490	4600	4750	
-22	Wet	2540	3310	3650	4010	4200	4160	4310	4020	
	Dry			3900	4700	4460	4220	4250	4380	
	Field			3980	4250	4940	4380	4090	4440	
-23	Wet	1670	2460	3150	3840	2970	3240	3980	-	
	Dry			3270	3880	3270	3590	3990	-	
	Field			3260	3680	2860	4170	3480	-	

CHAPTER IV

SPECIAL PROPERTIES OF EXPANDED SHALE IN LIGHTWEIGHT CONCRETE

This chapter is a resume of an investigation done by Fred E. Koebel (6) to study the properties of lightweight concrete made of expanded shale aggregates. Some of the properties studied were:

(i) the modulus of elasticity

(ii) the shearing strength

(iii) the amount of creep

(iv) a comparison test between grouted and non-grouted prestressed concrete.

Tests were run on three-20' beams using post-tensioned steel. The first beam with the properties listed below was used for a short time test.

Area = 10.83 in.^{2}

Ic = 5166.0 in.^{4}

r^2 = 47.6 in.^{2}

r = 6.9 in.

$\bar{y}_b$ = 10.0 in.

Design Load = 19,500 pounds

Calculated Cracking Load = 37,400 pounds

Modulus of Rapture = 700 psi

Total initial prestressing force = 113,500 pounds.

The beam was 32 days old when tested.

The properties of the other two beams which were tested over a period of four months were:

Area = 145 in.^{2}

I_c = 6454 in.^{4}

r^2 = 44.5 in.^{2}

r = 6.65 in.

y_b = 10 in.

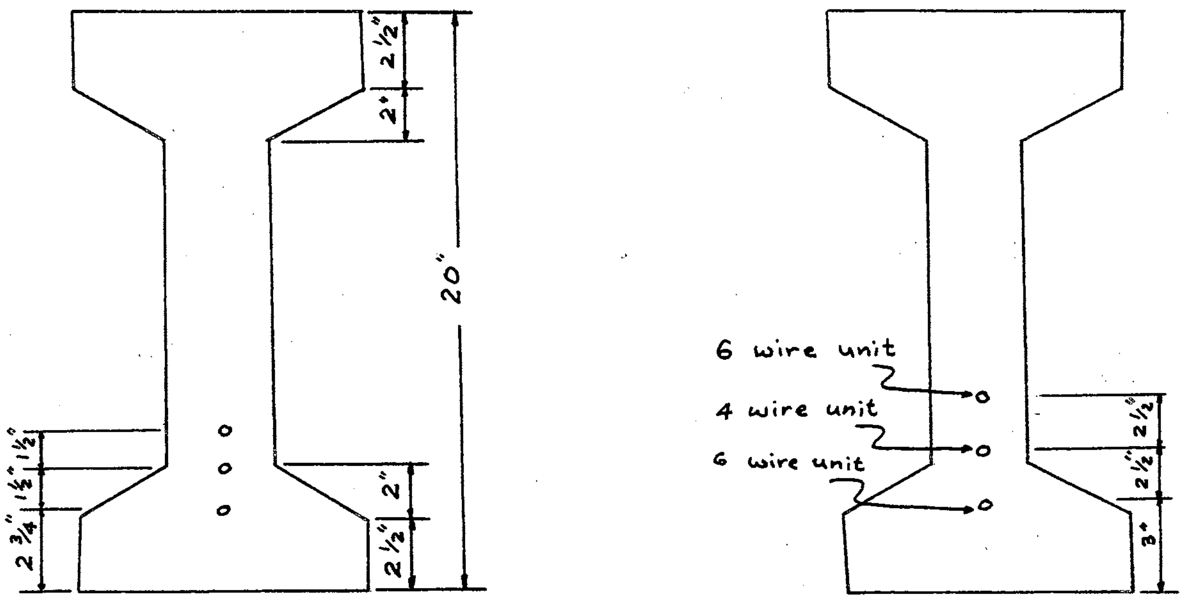
Design Load = 26,600 pounds

Calculated Cracking Load: Beam No. 1 = 38,259 pounds (non-grout

Beam No. 2 = 39,470 pounds (grouted)

Total initial prestressing force = 127,800 pounds.

The cross section of the beams is as shown:



Section Near the Support

Section of Mid-Span

Figure (1) - Section of the Beam

The materials used in the beams were: lightweight concrete from expanded shale, and steel of high tensile strength.

The prestressing steel was designed and placed so that it will resist the moments due to third-point loading.

The properties of the tensile steel were:

Diameter = 0.250 in.

Area = 0.049 in.^{2}

Min. Ultimate Strength = 220,000 psi

Yield Strength = 183,000 psi

Initial Strength = 145,000 psi

Modulus of Elasticity = 27.5 x 10^6 psi.

The properties of the concrete were:

Design Strength (28 days) = 5000 psi

Cement Factor = 6.75 sacks/cubic yard

Slump = 1-2 inches

Water (including absorption in aggregate) = 7.5 gallons per sack

Aggregate = 1.1 cubic yard BX Haydite per cubic yard of concrete

Unit Weight of Concrete = 105 pounds per cubic foot

Cylinder Strength Tests (av.) 2 days = 3000 psi
14 days = 4500 psi
28 days = 6000 psi

Short-Time Test

When the prestress was applied, readings of deflection and strain were taken until the operation of prestressing was finished. Four types of loadings were applied. Each one at a time, then the first load was removed to allow complete recovery and the second load was applied and so on. The loads applied respectively were: .70 L.L.

1.25 L.L.

Cracking Load

Failure Load

All of the loads were applied at the third points of the span. Deflection and strain were measured during the test.

Results of the Short Time Test

The initial modulus of elasticity was obtained from the initial deflection at 70% design load using the formula:

$$E_c = \frac{23}{648} \quad \frac{P}{2} \; \frac{L^3}{\Delta I} \quad \left[\Delta = .188"; \; P = 13{,}600 \text{ pounds}\right]$$

After 46 hours this value dropped down to 2.85 x 10^6.

At a loading of 1.2 L.L., the modulus of elasticity was found to be 3.15 x 10^6 psi.

Web cracking occured under a load of 43,000 pounds. The load at failure was between 51,000 pounds and 53,000 pounds. Failure was due to diagonal tension in the web.

All of the test runs showed a straight line relationship between the load and deflection up to the design load confirming the assumed elastic behavior of the beam.

Long-Time Test Results

The initial modulus of elasticity under full design load was found to be:

$$E_c = 3.58 \times 10^6 \text{ psi} \qquad \left[\Delta = 0.3", \quad P = 26{,}600 \text{ pounds}\right]$$

After two days this value dropped down to 2.75 x 10^6 psi; and after 121 days, it was 1.73 x 10^6 psi.

It was noticed that half of the inelastic deformations occured in the first 15 days.

Due to the shrinkage and creep of the concrete, the loss in prestress was found to be 19% in the first beam and 17% in the second beam. The assumed value of loss was 25%.

After the completion of the test, the non-grouted beam failed in compression in the top flange at a load of 44,000 pounds.

The grouted beam failed in diagonal tension at a load of 59,900 pounds.

After inspection, it was found that the ultimate bending strength was reached in case of the grouted beam, but not in case of the non-grouted beam.

The following remarks are worth mentioning in this respect:

(i) The expanded shale can produce concrete having enough compressive strength to stand prestressing.

(ii) The modulus of elasticity is not too low to be suitable for prestressing. The recovering property which this concrete has (recovering E) when the loads are removed adds to the merits of this aggregate.

(iii) An adequate value of 25% allowance for loss in prestress is a good practice.

(iv) The grouted wires add to the ultimate strength of the beam.

(v) The beam showed an elastic behavior up to the design load.

CHAPTER V

LIGHTWEIGHT AGGREGATE IN PRECAST PRESTRESSED CONCRETE MEMBERS

This chapter deals with two categories of precast prestressed lightweight concrete members. The first category includes some members built in the laboratory to be experimented on. The second category includes structural members which have been actually used in some existing buildings in several areas over the United States.

The First Category

Before the year, 1950, precast prestressed structural members were used in many countries in Europe in building construction. In the United States, however, it was only concrete pipes and cylindrical tanks which were precast and prestressed.

The increase in labor cost and the expected shortage in steel focused the interest of "Adrian Paw PAW" and "R.L. Reid (11) both from Houston, Texas, on the practice of precasting and prestressing. So in the year, 1950, these two men started an investigation of the factors to be considered in the manufacturing of precast prestressed units for building construction. A precast prestressed joist-slab-beam was the object of their investigation.

Prestressing units of lightweight concrete reduces material cost in two ways: (i) prestressing requires less steel, and (ii) the dead load of lightweight concrete is low compared to that of ordinary concrete, hence longer spans and longer economical hauling distances are possible. On the other hand, the increase in labor cost and plant cost tend to offset the savings in material. But still it is possible to reduce the plant cost to a minimum.

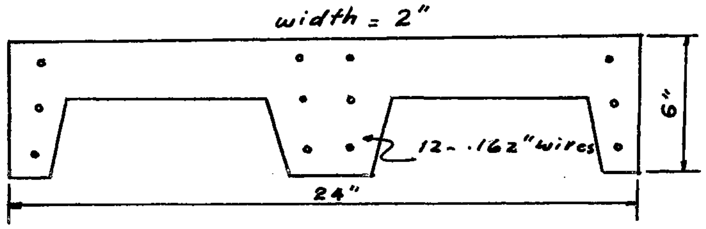
The materials and methods used in this experiment were as follows: Expanded clay was used as the aggregate. The mix design was eight sacks of high early strength cement for each one cubic yard of mix. The mix had the following qualities: A two inch slump, and a compressive strength of 4,800 psi (after 28 days).

The reinforcing wires were oil tempered wires, .162 inch in diameter, pretensioned and bonded, having a yield strength of 180,000 psi and an ultimate strength of 230,000 psi. The wires had an elongation of 4 - 5%, and a modulus of elasticity of 29.2×10^6 psi.

The bond between the wires and the concrete was carefully studied. The values of the bond found ranged from 100 to 150 pound/linear inch. For the 0.162 inch diameter wires, there were no instances where the bond decreased with the age of the specimen.

Several scale models were constructed to investigate some of the technical problems which may be encountered while testing the full scale specimens.

After these model tests, a full scale joist was constructed, (see Figure 2).



Area = 70.2 in.^2

I = 205 in.^4

S = 52.2 in.^3

Section of the Joist

Figure (2)

The span of the joist was 20 feet - 7 3/4 inches. The design load was 25 psf. When testing these joists, third point loading was used.

The following results were obtained: The load at the jack, when the first crack appeared, was 455 pounds; while the calculated value was 465 pounds. The maximum load on each jack was 1000 pounds. The maximum deflection noticed was 11½ inches.

Conclusion Derived from this Experiment

There are no serious technological problems which cannot be solved in manufacturing precast prestressed lightweight concrete units for building construction.

The principal technological problem which was encountered was the need for a rapid curing means so that the stress can be transferred to the concrete section in a short period of time.

The Second Category

Several projects will be described here. Two of these projects have many similarities so they will be discussed together.

These two projects are: (i) A two-story warehouse and office with 40 feet - 0 inches clear spans. The second floor was supported on prestressed beams of lightweight aggregate (expanded shale) designed for 100 pound/square feet live load. The beams were 32 inches deep and 20 inches wide at top flange. (ii) A television studio and transmission station. Sections of the beams for both projects are shown in Figure (3).

Design of Beams and Slabs

The slabs consisted of precast expanded shale blocks with grouted-in reinforcing, and a certain amount of prestressing. The blocks had key-joints on the edges so that they could be locked together with little grouting.

The beams for both jobs were designed with the same basic stresses:

fc' = 5000 psi @ 28 days (ultimate)
fc = .4 fc'
fco' = 2/3 fc' = 3300 psi (stress at transfer)
fco = .6 fco' = 2000 psi (maximum stress at prestressing)
final tension under load = 0
tension at transfer (fto) = - 167 psi
stress in steel at transfer = .67 x 250,000
= 167,000 psi
stress in steel after losses = .8 x 167,000
= 134,000 psi.

Due to the low modulus of elasticity of the lightweight concrete, the allowance for creep was increased from 15% of steel stress to 20%. Shrinkage was reduced by steam curing and the use of no-slump concrete. The jobs were 120 miles from the casting yard; and the contractors who executed the work were unfamiliar with this type of construction. So it was decided to precast the beams in rugged shape to stand the hauling and erection stresses caused by rough handling.

Four - #5 unstressed bars were used in each flange of the I-beams, and .192 inches^{2} in the rectangular beams, to take care of rough handling and overhang in loading.

The safety factors used in the design of the beams were:

cracking moment = D.L. moment + 1.5 times the L.L. moment.
ultimate moment = D.L. moment + 3 times the L.L. moment.

The shear, the bond, and the maximum principal tensile stress were found to be small; so they were neglected.

Deflections were calculated and they were used as a good check on the prestressing forces and the assumed values of the modulus of elasticity. The calculated values of deflection checked very closely with the measured values.

The values of the modulus of elasticity used in calculating the deflections was 2,000,000 psi, which means, since the calculated and measured deflections were almost the same, that this was a reasonable value of E_c for this type of concrete. The same value was obtained using the same aggregate in another test, done at the Oregon State College of Engineering.

Beam Manufacturing

No-slump concrete was used in constructing the beams. It was found that vibration from outside is better than internal vibration because this helps to keep the cables in their exact positions.

The cables were composed of 12 - .196" diameter wires, and after they were prestressed, they were grouted.

It was noticed that it was necessary to secure enough room on the jacking end of the beam to make it possible for the jack to be moved, if deemed necessary, without any difficulty.

In one of the "TV" roof beams, transfer of prestress occured when the concrete had only 3,330 psi compressive strength. This caused the lower anchorage cone at the jackling end to slip in $1\frac{1}{4}$ inches; it sheared off a small piece of the end of the beam. No other similar cases were reported about the other beams. The sheared off beam was patched at the end and tested. It proved to be suitable, so it was taken to the site and used.

Hauling and Erection of Beams and Slabs

No damage happened to any of the beams during hauling or erecting.

The use of lightweight aggregate here reduced the dead weight by 30%, and this was of great advantage in hauling the members.

On these particular jobs, 1/3 more truck round trips of 240 miles would have been required if ordinary concrete had been used.

A Summary of Costs

The cost of a four-inch block slab, cast at the plant, was estimated to be about 65 cents per square foot; and the cost of the six-inch deep blocks was about 85 cents per square foot. City delivery cost was five cents per square foot. Erection cost was from five to ten cents per square foot.

For the beams, it was more difficult to give an estimate but the figures shown below give a fairly good idea about beam costs.

Span Range in Feet	Cost in Dollars per Square Foot of Tributary Area
25 - 30	0.50
30 - 40	0.65
40 - 50	0.80

Other Jobs using Precast Prestressed Lightweight Concrete Members:

(i) The auditorium, music building, and girls' gymnasium, Antioch Unified School District, California (15).

Four beams were used to support the roof. Each of them was: 98 feet long, 5 feet - 3 inches deep, having a clear span of 96 feet. They were plant cast, postensioned and trucked to the site. Each beam contained 32 cubic yards of concrete, having a compressive strength of 5000 psi; each weighed 50 tons. (Expanded shale was the aggregate).

For prestressing, nine cables of twelve .276 inch wires were used in each beam. The initial prestress force was 935,000 pounds on the nine cables.

The importance of these beams is that, so far, they are the largest in California. (1960)

(ii) The American Cyanamid Company Warehouse in Brenster, Florida (16-C).

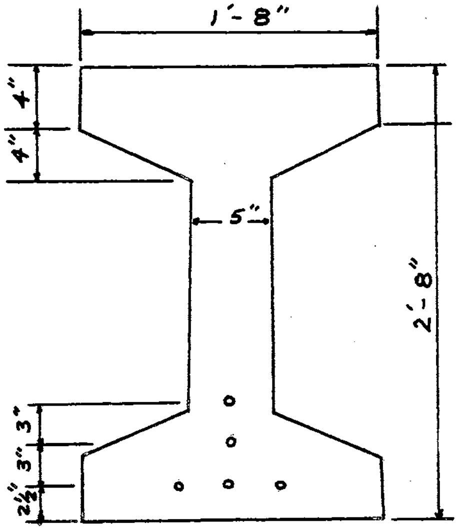
The building is 960 feet long and 100 feet wide. There are no interior columns in the building. All the structural elements are precast and prestressed units.

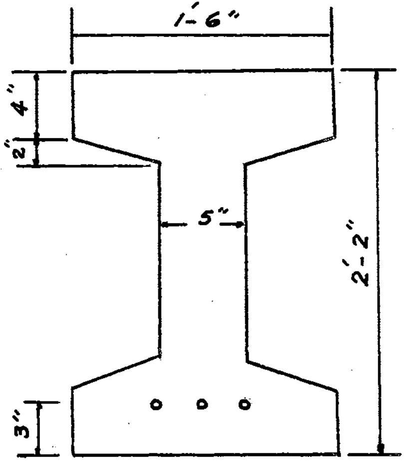
The notable feature in the manufacture of the prestressed concrete girders involves the "one-a-day" schedule for casting the huge roof girders.

Thirty-six cubic yards of pozzolith concrete (lightweight) were used in each girder.

Thirty-three large girders were used in this building each measuring 101 feet - six inches long; 12 feet high at the center, and four feet high at the ends. The top flange was three feet wide; each girder weighed 71 tons.

eam	A_c Sq. In.	I_t $In.^4$	Z_t Cu. In.	Initial Prestress (Lb.)	Final Prestress (Lb.)	Design Moments, Ft. - Lb.						Cracking Safety Faster	Ultimate Safety Faster
						D.L.	Beam Weight	L.L.	Total	Cracking	Ultimate		
e- se: ond or	340	44,228	2770	317,000	254,000	97,400	47,200	263,000	407,600	579,600	985,000	D.L. + 1.65 L.L.	D.L. + 3.2 L.L.
f	260	21,880	1690	192,000	154,000	54,200	37,100	81,500	172,800	308,800	560,000	D.L. + 2.67 L.L.	D.L. + 5.7 L.L.
. tion: f	216	5,830	648	181,000	145,000	29,400	14,700	41,200	85,300	122,500	257,000	D.L. + 1.9 L.L.	D.L. + 5.16 L.L





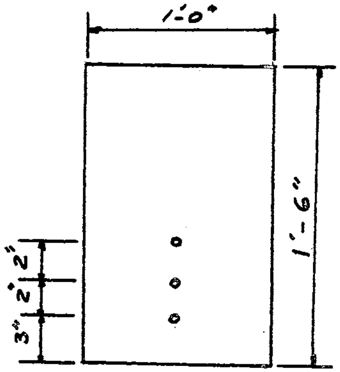


Figure (3) - Beam Sections, Design Constants, Design Moments, and Safety Factors

CHAPTER VI

COMPARISON BETWEEN LIGHTWEIGHT CONCRETE AND CONVENTIONAL CONCRETE UNDER STATIC AND FATIGUE TESTS

The material presented in this chapter is a summary of an article written by Gen Nordby and William Venut (9), after they conducted a two-fold purpose study.

(i) To investigate the use of lightweight aggregate in bonded type prestressed concrete beams.

(ii) To explore the effects of fatigue loading on prestressed concrete beams, made with both conventional stone aggregate and expanded shale aggregate.

Preparation of the Specimens

The materials used in preparing the specimens were: (i) seven-wire uncoated strands of 5/16 inches and 3/8 inches in diameter (these strands were pretensioned). Tests on these strands indicated an ultimate strength of 272,000 psi; and a modulus of elasticity of 28.75×10^6 psi. (ii) Stone Aggregate Concrete; (iii) Expanded Shale Aggregate.

Both of the two aggregates had the same gradation: the sizes ranged from the size of sand up to 3/4 inch particles.

The water cement ratio was 0.41 by weight for both types of concrete.

Steam curing was used to bring the concrete to a strength of 4000 psi rapidly to allow early transfer of prestress.

The average slump of both types of concrete was $\frac{1}{2}$ inch.

Both types of concrete gave approximately equal strengths for equal water cement ratios (the compressive strength was 5000-6000 psi after 28 days).

For shale concrete, the modulus of elasticity was 2.5×10^6 psi and the modulus of rupture was .074 fc'; but for stone concrete the modulus of elasticity was 3.6×10^6 psi, and the modulus of rapture was .109 fc'.

The average unit weight of the shale concrete was 100 pounds per cubic foot, and the average unit weight of the stone concrete was 146 pounds per cubic foot. The specimens were constructed in four different cross sections (see Figure (5)).

Fatigue tests were performed on the A, B, and C beams; while beam D was tested only for static tests.

In all the specimens the initial prestress was 175,000 psi, but after losses, the stress was reduced to 155,000 psi.

The main object of performing the static test on beams was to study the bond theory and the embedment length under various conditions of loading.

Description of the Tests

Tables (5), (6), and (7) contain the results of fatigue and static tests.

As the tables show the design load was approximately 27% to 29% of the ultimate load while the cracking load was approximately 58% of the ultimate load.

The tests were carried out in three phases.

During the first phase, six beams of each of the cross-sections A and B were cast of ordinary concrete. One beam of each set was tested under fatigue load. The other beam of each set was tested under static load. The results of both tests were recorded and compared.

The typical failure under the static test was due to exceeding the ultimate strength of the steel. Only a few beams in this phase failed under fatigue load by fatigue of the steel strands; beam 6A failed under a load of 2.4 times the design load after 136,000 cycles. Most of the failures under fatigue load were due to cracking of the concrete under the fatigue load.

During the second phase, three beams of conventional concrete and three beams of expanded shale concrete were cast for testing.

The compressive strengths and the water cement ratios were approximately equal for both types of concretes in this phase.

The first matched pair of beams - a conventional concrete beam and an expanded shale concrete beam - were loaded by static loads to failure. The shale beam showed a greater deflection than its mate because of the lower modulus of elasticity which the shale aggregate concrete had. The cracking and ultimate loads were identical (see Table (5)).

The second pair was identically loaded by fatigue machine to the design load. No damage occured to either one of these two beams.

Under 80% of the ultimate load in a static test, both beams suffered some cracking; slip of the steel strands occured in both beams.

The last pair was loaded at 2.5 times the design load in a fatigue machine. The shale beam failed in steel fatigue after 842,000 cycles. The ordinary concrete beam did not fail even when subjected to 2,000,000 cycles.

During the third phase, six beams of cross section D - three of each type of concrete - were cast and tested under static load.

Failure in all the beams was due to bond failure, and it was identical in character in the case of all the beams. (See Figure (6) for type of loading).

Discussion of the Results

Loss of Prestress

The contributing factors to the loss in prestress were: the elastic deformation of the concrete under prestress (compression), and shrinkage and creep in the concrete.

The measured loss in prestress due to the elastic deformation of the concrete was only 5.4% below the calculated value based on the modulus of elasticity of the concrete obtained from the cylinder tests.

The creep was measured over a period of (90) days. In one case, ordinary concrete had a greater creep value than shale concrete; in all the other cases the reverse was noticed.

Steel Fatigue Failure

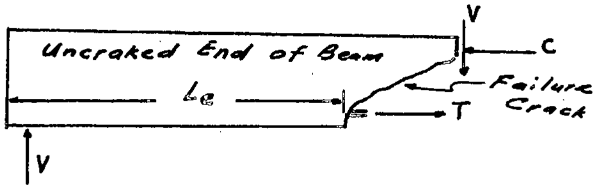
Only three beams (6A, 6B, S6 - see Table (6)) failed by steel fatigue. The fatigue was observed to be across the top plane of the wires resulting in failure across the diagonal plane. These fatigue failures were probably caused by cracking of the concrete under severe loading, which resulted in stress concentration on the wire and abrasion of the wire.

Bond Failures

Nine beams of the second and third phases failed in bond. This type of failure was detected by dials attached to the end of the beam to record slip. The initial slip occured at stress increases in the steel as low as 3020 psi, before any crack was noticed in the beam. The first crack appeared as a vertical flextural crack under one of the loads; after that, all strands slipped .03 to .01 inches. A diagonal tension crack started to develop. Any time after the slip reached .01 inches, failure was due to shear.

When the beams were broken after the test, it was noticed that there was no bond between the concrete and the strands.

It was noticed, after investigation, that the average bond stress decreased rapidly as the length of embedment increased; and this value of bond stress reached an asymptotic value for embedment of six feet or more.



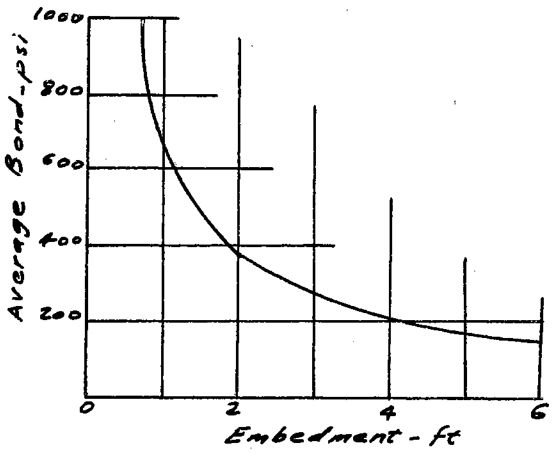


Figure (4) - Average bond stress developed at failure versus embedment length

The test indicated that a six feet length of embedment was required to develop the ultimate strength of the 3/8 inch strand when embeded in concrete having a compressive strength of 6000 psi (ultimate). For the 5/16 inch strand, only a three feet length was required. For strands used with shale concrete, seven feet or eight feet should be recommended.

Conclusion and Remarks

Expanded shale aggregate in concrete produced a concrete with a compressive strength equal to the compressive strength of the concrete made from gravel aggregates, when the water-cement ratio was equal in both kinds of concrete.

The creep and shrinkage values of concrete made from expanded shale aggregate were lower than the maximum values allowed for in the specifications

The modulus of elasticity of the shale aggregate concrete was found to be rather low.

Therefore, there is no objection to the suitability of shale concrete to be used in prestressed concrete structures.

The steel strands proved to be of higher advantage than the smooth wires because the strands had a higher mechanical bond. This mechanical bond in the strands prevented complete loss of prestress even after some slip had occured in the beams.

Fatigue failures did not happen in the beams before they were cracked. So cracks should not be allowed in beams subjected to repeated loading.

It was noticed that slip of strands occured at low bond stress. So no limiting bond stresses should be assumed in design. It is rather the embedment length between the end of the beam and the first possible crack that should be specified.

The test showed that the elastic theory was applicable on concrete made either of lightweight aggregate or heavy aggregate before the beam was cracked.

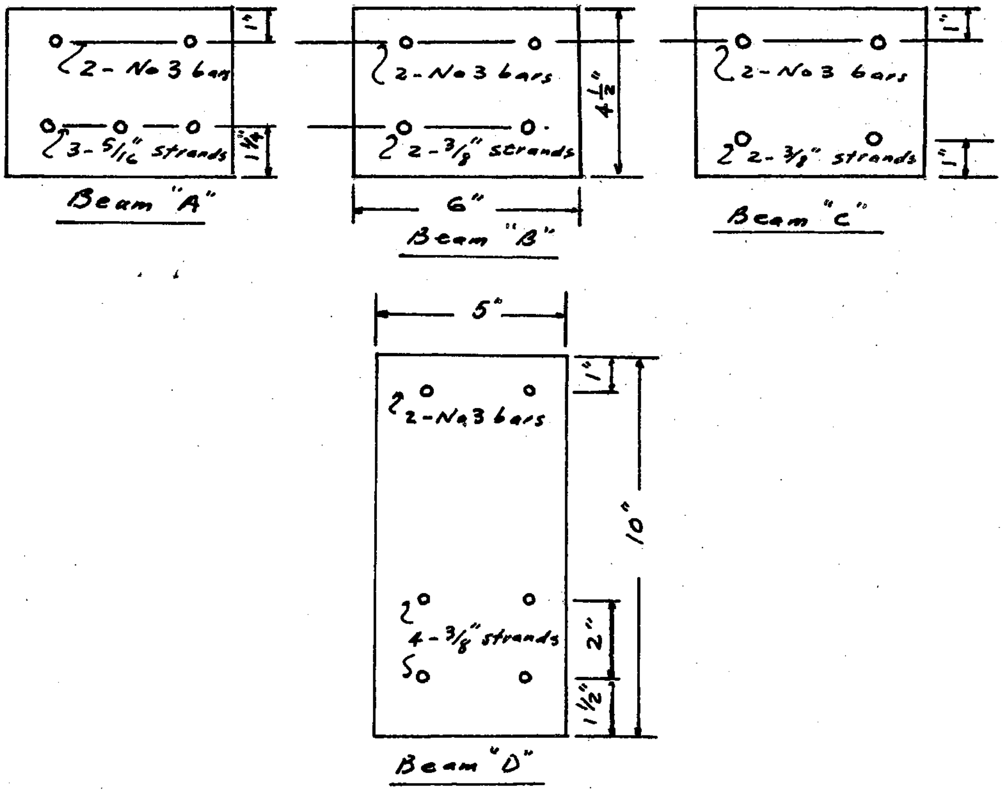


Figure (5) - Section of Beams A, B, C, and D

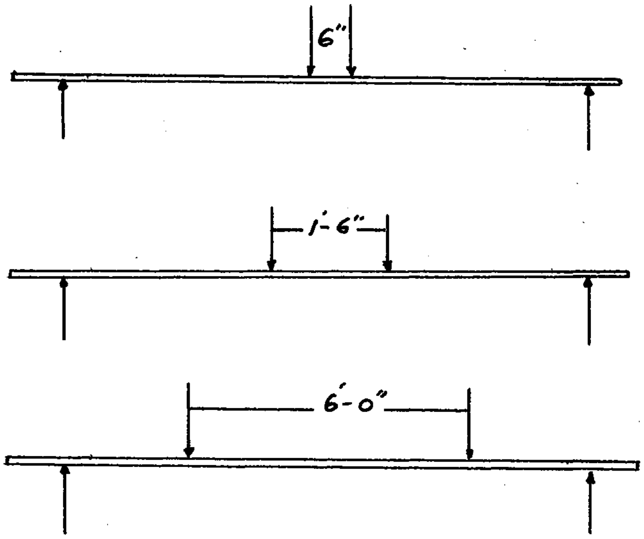


Figure (6) - Loading Spans for Static and Dynamic Tests

TABLE 5 - SUMMARY OF STATIC TESTS

Conventional Concrete

eam No.	Cross Section Fig. (4)	Load Type Fig. (5)	fc' at test	Concrete Prestress top	bottom	Steel Prestress psi	Steel Stress at Failure psi	Cracking Moment K - In.	Ultimate Moment In. - Kips	Bond at Ultimate psi	Type of Failure
A	A	II	6660	-49	1891	160,635	246,070	55.4	114.1	58	Conc. Crushing
B	A	II	6660	-49	1891	160,635	238,290	57.9	114.1	53	" "
A	A	II	6660	-44	1856	158,065	239,890	-	118.1	56	" "
B	A	II	6660	-44	1856	158,065	239,960	60.5	114.9	54	" "
A	A	III	6660	-44	1856	158,060	233,915	57.6	105.3	86	" "
B	A	III	6660	-44	1856	158,065	229,910	57.2	103.5	91	" "
A	B	II	6040	-68	1794	157,320	251,720	61.1	118.1	94	" "
B	B	II	6040	-68	1794	157,320	275,160	-	129.1	118	" "
A	B	II	6040	-66	1784	156,480	251,720	-	118.1	95	" "
B	B	II	6040	-66	1782	156,390	258,970	-	121.2	103	" "
A	B	II	6040	-66	1784	156,480	-	-	76.9	-	Steel Fatigue
B	B	II	6040	-66	1786	156,560	-	-	76.9	-	" "

Shale and Conventional Concrete

eam No.	Cross Section Fig. (4)	Load Type Fig. (5)	fc' at test	Concrete Prestress top	bottom	Steel Prestress psi	Steel Stress at Failure psi	Cracking Moment K - In.	Ultimate Moment In. - Kips	Bond at Ultimate psi	Type of Failure
1	B	II	6400	-121	1737	154,190	236,600	-	113.4	68	Conc. Crushing
2	B	II	6400	-121	1737	154,190	239,900	-	114.9	71	" "
3	B	II	6780	-115	1697	150,190	246,400	-	118.1	78	" "
4	C	III	5840	-360	1910	148,600	204,900	55.8	81.0	69	Bond
4	C	III	5430	-300	1950	151,600	217,700	47.5	83.7	69	"
5	C	III	5480	-350	1890	147,300	-	-	79.8	-	"
5	C	III	5430	-300	1950	151,300	-	-	83.7	-	"
5	C	II	5600	-370	1940	150,700	-	60.7	76.9	-	Steel Fatigue
5	C	II	6345	-310	1980	153,600	244,900	59.2	123.8	74	Concrete Crushing

TABLE 5 (Cont'd) - SUMMARY OF STATIC TESTS

Shale and Conventional Concrete

eam No.	Cross Section Fig. (4)	Load Type Fig. (5)	fc' at test	Concrete Prestress		Steel Prestress psi	Steel Stress at Failure psi	Cracking Moment K - In.	Ultimate Moment In. - Kips	Bond at Ultimate psi	Type of Failure
				top	bottom						
3	D	I	5820	-310	2080	152,100	214,600	310.5	427.8	46.5	Bond
3	D	I	5390	-310	2120	152,500	272,000	308.7	552.2	93	Conc. Crushing
1	D	II	5535	-340	2150	157,100	206,000	275.6	406.3	41	Bond
1	D	II	4956	-320	2180	156,900	216,400	264.6	346.5	39	"
2	D	III	5840	-340	2120	154,900	190,400	260.1	378.0	55	"
2	D	III	5470	-320	2160	154,900	174,300	229.5	306.0	26	"

Note: S = Shale Concrete

G = Gravel Concrete

P_1, P_2, P_3 = Shale Concrete (Pilot Beams)

TABLE 6 - SUMMARY OF FATIGUE TESTS

Beam No.	Repetitions of Loading	Fatigue Load		Fatigue Moment Ft.-Kips	Fatigue Shear (Lbs.)	Steel Range of Stress psi	Concrete Range of Stress psi	Bond Due to Flexure Range of Stress psi
		% of Ultim Load	% of Design Load					
1A	1,014,100	29.4	100	33.7	535	5000	1560	3.4
2A	1,000,000	65.0	240	76.9	1220	21,340	6208	14.7
3A	2,000,000	73.0	240	76.9	2135	11,485	-	13.6
4A	9,653,000	27.6	100	32.7	520	4540	1530	3.8
5A	1,108,700	41.5	150	49.1	780	6800	2295	3.7
5B	1,100,000	54.0	200	65.5	1040	-	-	-
6A	136,000	67.0	240	76.9	1220	23,890	-	20.7
6B	186,000	67.0	240	76.9	1220	23,890	-	20.7
P2	1,100,000	28.5	100	32.7	520	6600	1470	3.9
P3	2,168,000	55.5	200	65.5	1040	-	-	-
S5	1,000,000	28.0	100	36.0	1000	8900	1551	1.3
G5	1,000,000	28.0	100	36.0	1000	7300	1620	1.3
S6	842,000	62.0	240	76.9	1220	29,400	2110	24.0
G6	2,000,000	62.0	240	76.9	1220	24,400	2360	23.4

TABLE 7 - MEASURED STEEL STRESS RISE AND BOND STRESSES AT SLIP AND FAILURE

Beam No.	Stress Rise of 1st Slip, psi				Stress Rise at Ultimate Load		Bond at 1st Slip psi	Bond at Ultimate psi
	Lower Strand		Upper Strand					
	No.1	No.2	No.3	No.4	Lower	Upper		
S1	N.G.	N.G.	3020	8000	46,900	28,600	3.5	167
G1	11,780	11,780	N.S.	4025	59,500	16,700	4.7	175
S2	18,250	18,250	2703	6320	35,500	20,270	6.6	330
G2	13,650	13,650	6470	6470	19,400	10,400	-	280
S3	N.S.	62,530	25,580	15,525	62,530	25,580	-	158
G3	N.S.	N.S.	N.S.	N.S.	119,500	80,648	-	199
S4	33,640	33,640	-	-	66,100	-	-	264
G4	11,070	11,070	-	-	56,350	-	-	272
S5	-	-	-	-	75,890	-	-	206
G5	-	-	-	-	71,730	-	-	-

Note: N.G. = No Gage
N.S. = No Slip

CONCLUSIONS

I. Experiments and tests made on lightweight aggregate in prestressed concrete elements, on a pilot scale and on a full scale, indicated that lightweight aggregates can be used advantageously in prestressed concrete members.

II. The several structures which have been constructed during the last decade, using lightweight aggregate in prestressed concrete members were the best proof of the validity of the results of the experiments, which have been carried out on lightweight aggregates in prestressed concrete before the construction of these structures.

III. Lightweight Aggregates in concrete structures have many advantages over heavy aggregates such as: lighter weight, and hence lighter dead load; better insulating qualities against heat and sound; and, more economical hauling distance and erection.

IV. On the other hand, lightweight aggregates in prestressed concrete structures have some deficiencies which make these aggregates inferior to heavy aggregates in some respects. The most serious deficiency that faces lightweight prestressed concrete members is the low modulus of elasticity of the concrete. But this low value is not serious enough to over-throw the suitability of this aggregate for use in prestressed concrete members. There are other minor deficiencies such as that more care and control are needed in proportioning, mixing, and casting concrete made of lightweight aggregates.

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