JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

(ACI PROCEEDINGS Vol. 41)

Vol. 16 — January 1945 — No. 3

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Vol. 16—No. 3 January 1945 (Proceedings Vol. 41)

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

Published by the American Concrete Institute. The Institute was founded 1905; incorporated in the District of Columbia in 1906 as The National Association of Cement Users; the name changed 1913 by charter amendment. The Journal is issued six times yearly in the months of January, February, April, June, September and November under the authority of the

Papers and other contributions prepared with a view to Journal publication should be submitted in triplicate, addressed: Secretary, Publications Committee, American Concrete Institute, 7400 Second Boulevard, Detroit 2, Michigan. Publication of a contribution does not imply the acquiescence of critics (whose advice is sought by the Committee, prior to acceptance) or of the Institute in the opinions which it expresses nor the approval of data or practice which the contribution records. Institute authority attaches only to Standards formally adopted as provided in the By-Laws. A committee report implies merely the joint contribution of an appointed group.

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Publication address: 7400 Second Boulevard Detroit 2, Michigan. Printed in U. S. A. Entered at the Post Office at Detroit, Michigan, as mail of the second class under provisions of the Act of March 3, 1879.

the ACI JOURNAL

It is the policy of the American Concrete Institute to encourage participation by its members and others in the work of extending the knowledge of concrete and reinforced concrete as a basis for improved products and structures.

To this end the Board of Direction has assigned to the Publications Committee the responsibility of selecting for publication such papers, committee reports, discussions and other contributions or parts of such contributions, as in the judgment of the Committee, seem to offer most of value in attaining Institute objectives within space requirements, consistent with budget limitations.

AMERICAN CONCRETE INSTITUTE
NEW CENTER BUILDING
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● The New York Convention, February 1945 will include none of the usual technical sessions—a luncheon-business meeting only at noon, February 16, with meetings of Board of Direction, and of the Advisory and Publications Committees, February 14 and 15 and 16. (See News Letter p. 1)

● The American Concrete Institute has announced the inauguration of the ACI Construction-Practice Award, to be given for a paper of outstanding merit on concrete construction practice. This award is established to honor the construction man—the man whose resourcefulness comes in between the paper conception and the solid fact of a completed structure.

The token of the award is a suitable Certificate of Award accompanied by $300 (maturity value) of United States War Bonds Series E. It is hoped to enrich the literature of concrete construction-practice.

● Five cash awards are also announced for contributions to the Job Problems and Practice pages September 1944 to June 1945. For the contribution judged by an ACI Committee to rank as the best of the volume year (V. 41)—$50.00; next best $25.00; to each of the three next best $10.00. See announcement of these awards, June JOURNAL or ask for a leaflet.

● Discussion of report and papers in this issue closes April 1, 1945.

A limited supply of separate prints of all papers and reports becomes available with the publication of each JOURNAL issue (usually 25 cents each).

A part of PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE Vol. 41

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

Vol. 16 No. 3 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN January 1945

Concrete Operations in the Concrete Ship Program*

By LEWIS H. TUTHILL†
Member American Concrete Institute

SYNOPSIS

This paper describes only briefly the hulls constructed in the concrete ship program of the U. S. Maritime Commission but goes into more detail in connection with problems encountered and their solution in the course of these concrete operations. Construction joint procedure, lightweight aggregate concrete and mix control, handling, placing and vibration practice, curing, testing and repair problems are described in the belief that much of this information is applicable to any concrete work of high standard. Design of hulls is not discussed except as construction is affected. The ships have not been in service sufficiently long to justify much discussion of their performance or durability.

INTRODUCTION

In the spring and summer of 1941, when demand for ship tonnage began to increase and there were indications that production of steel plate might not be sufficient for all demands, consideration was given by the U. S. Maritime Commission to the use of other materials for ships. After considerable investigation it was decided to inaugurate a program of reinforced concrete vessels. Inasmuch as such vessels require very little steel plate, their production would make use of a type of material and a class of labor which would conflict very little with the demands of steel ship construction. Construction of facilities, and later of hulls, began during 1942. While considerable difficulty was encountered during the first year of the program, due to lack of knowledge and experience, it was gradually overcome, and deliveries of concrete vessels started in 1943. The increasing number of vessels delivered in each succeeding quarter of 1943 (1, 4, 11, and 17, respectively) is evidence of the soundness of the conclusion that such ships could be built. Their performance so far, as reported, justifies the decision to build them.

*Submitted to the Institute August 22, 1944; approved for publication by U. S. Maritime Commission.
†Senior Engineer, Bureau of Reclamation, Denver, Colorado; Chief, Concrete Control Sub-Section, Technical Division, U. S. Maritime Commission from March 1943 to July 1944

During the writer's association with the program there were five concrete shipyards in operation, and five different designs of vessels built. Table 1 outlines the principal features of each design and the production of each yard:

TABLE 1—OUTLINE OF DATA ON CONCRETE HULLS

Yard	Number built	Lgth. BP ft.	Beam ft.	Depth ft.	Displacement with cargo, long tons	Cargo, long tons	Concrete cu. yd.	Steel, short tons
Savannah	7	350	54	35	10930	5430	2730	1525
Houston	4	350	54	35	10930	5430	2730	1520
Tampa	24	350	54	35	10930	5200	3100	1250
San Francisco	20	350	54	35	10930	5730	2600	1100
National City	22	360	56	38	12750	6375	3200	1655
National City	27*	265	48	17½	4000	1600	1125	490

*These lighters were an extension of the initial program.

Steel vessels of similar type and cargo capacity would require from 20 to 40 percent more steel, and practically all the steel used would be plate, instead of the relatively plentiful steel bars of the concrete vessels. While the intricate design and difficulties of construction ran the costs of the early concrete hulls very high, indications are that a properly designed concrete hull, on a production basis in well-equipped yards, could be built for no greater cost, and possibly less, than a corresponding steel hull. A good reinforced concrete vessel should have a considerably longer life and lower maintenance costs, particularly as a tanker or for other specialized cargoes, although the weight of the hull is appreciably greater. Considering all factors, it is possible that the cost per ton mile of transportation in concrete ships or barges would be no greater in some services and might even be somewhat less, than in steel ships, despite the higher cost of propelling the heavier concrete vessels, either by self-propulsion or by towing.

GENERAL DESCRIPTION OF HULLS

All vessels built during this program except the Tampa vessels were barges or lighters, with no propulsion machinery and were designed for towing, either singly or in tandem. At Tampa, self-propelled, dry-cargo vessels were built, officially called the C1-S-D1 design. The Savannah, Houston, and the first National City vessels were oil barges, composed of separate cargo holds or tanks, each of which had to be "bottle tight" on every face under a hydrostatic test with water 8 feet above the deck. Dry cargo hulls had to be practically as watertight under test, but the tests were either at lower head or made with hose pressure. The shells of the vessels ran from 4¼ in. to 6 in. thick (except the group of lighters built at National City), contained from 3 to 5 "layers" of closely-spaced steel bars, and were designed for a coverage over the steel of ¾-in. on the exterior or outboard face, and ½-in. on the interior or inboard face throughout. It might be mentioned that our investigation of concrete vessels built during the World War I period indicated that where the coverage over the steel was ⅜ in. or more, the steel rarely rusted or became exposed by spalling of the concrete.

Fig. 1—One of four concrete oil barges built at Houston. Seven similar tankers were built at Savannah.

The oil barges built at Savannah and Houston (Fig. 1) were identical in design. These hulls had two longitudinal bulkheads and ten transverse bulkheads, the latter on 32-ft. centers, forming in the midship or parallel body section center tanks 18 ft. 4½ in. by 32 ft., and wing tanks 17 ft. 9¾ in. by 32 ft. Transverse rib framing was spaced at 10 ft. 8-in. centers and supported a system of horizontal beams at approximately 4-ft. centers which in turn supported the 4½-in. shell and 4-in. bulkhead slabs, as shown in Fig. 2 (and in Fig. 40). The bottom slab was 5 in. thick, the deck 4 in. Reinforcing details are in Fig. 3.

The oil barges built at National City, California, (Fig. 4), had only one longitudinal bulkhead at the centerline. This fact and the use of rib frames at 5 ft. 5½ in. centers as shown later in Fig. 39, without horizontal beams for the side shell (except behind fenders) resulted in an appreciably simpler structure, easier to construct than the other oil barges of the program. Transverse bulkheads were spaced at 27 ft. 2½-inch centers and were carried on a system of vertical ribs held by tie beams or struts at midheight shown later in Fig. 28. Bulkhead and sideshell slabs were 4½ to 5 in. thick, bottoms 5 in., and decks 4¾ in. Fig. 5 shows the reinforcing of these slabs.

The dry cargo, self-propelled ship built at Tampa (Fig. 6), had transverse bulkheads at 32 ft. centers with no longitudinal bulkheads except in one bay where there were two, which form two 17 ft. 5½-in. by 32-ft. wing ballast tanks with a 19 ft. 1 in. by 32-ft. void space between them.

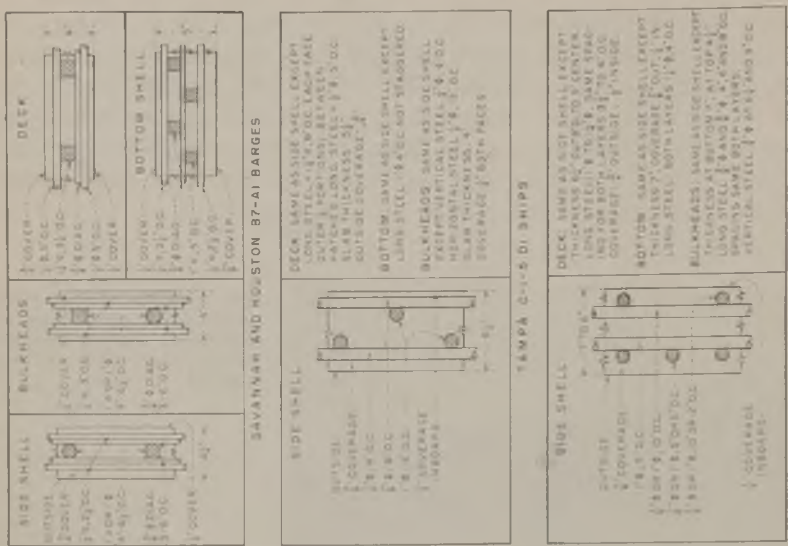


Fig. 3—Hull reinforcement details.

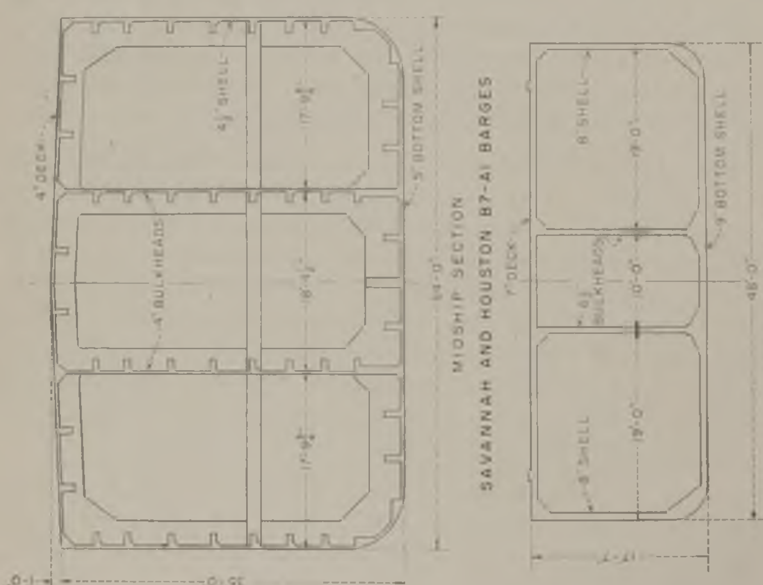


Fig. 2—Comparison of complicated and simplified design.

Fig. 4—One of 22 concrete oil barges built at National City, Calif.

Fig. 6—One of 24 self-propelled dry cargo concrete ships built at Tampa.

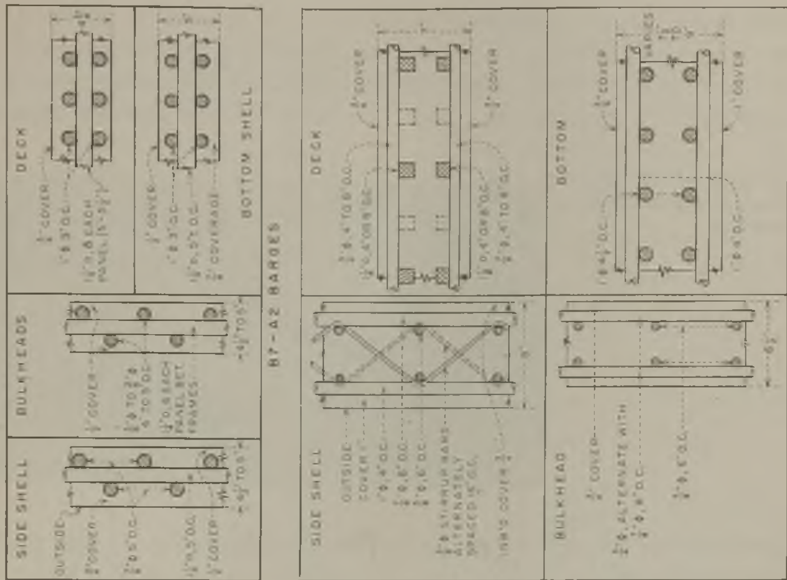


Fig. 5—Reinforcing details—National City hulls.

In these hulls, ribs on shell and bulkheads were spaced on 10 ft. 8-in. centers and carried horizontal beams at about 4-ft. centers which supported the 6½-in. shell and 4-in. bulkhead slabs. Bottom slabs were 6½ in. thick and the decks 5½ in. thick. See Fig. 3 for reinforcing details.

Fig. 7—One of 20 dry cargo towing ships built at San Francisco.

The dry cargo barge built at South San Francisco (Fig. 7) was probably the easiest hull to build in the initial program, but as Fig. 8 shows, it was by no means simple. It had no horizontal beams in the bulkheads or shell except opposite fenders on the latter, the slab of tapering thickness being supported directly on ribs at 6 ft. 4-in. centers. There were no longitudinal bulkheads. Transverse bulkheads were spaced at 32 ft. Slab thicknesses were 7 to 4½ in. bottom to top in the transverse bulkheads 7 to 6 in. bottom to top in the shell, 7 in. in the bottom, 5 to 6¼ in. in the deck. See Fig. 3 for details of reinforcing.

In frames and beams for all four of these hulls, bars up to 1¼-in. square were used in various amounts and patterns with closely spaced stirrups as shown in Fig. 9, 10 and 11. The general detail of steel in the shell, bulkhead, and deck slabs for all hulls is shown in Fig. 2 and 3. The density of steel in the concrete is realized when it is noted that about ½ ton of steel was embedded in each cubic yard.

Of entirely different design were the 25 lighters built at the National City yard in 1944. Two longitudinal bulkheads 10 ft. apart provided void spaces on the centerline as shown in Fig. 12. Six transverse bulkheads separated remaining space into 12 cargo holds approximately 19 by 48 ft. and the void space at the fore peak. This hull was unencumbered with ribs, beams, frames, columns, pilasters or struts as shown in Fig. 2. Shell sides, bottom, deck and bulkheads were flat slabs completely unrelieved except for haunches at the connections and corners. Bulkhead slabs were 6½ in. thick; all were precast (the transverse bulkheads in one piece, the longitudinals in lengths between transverse bulkheads); and in some hulls all 20 pieces, most of them weighing

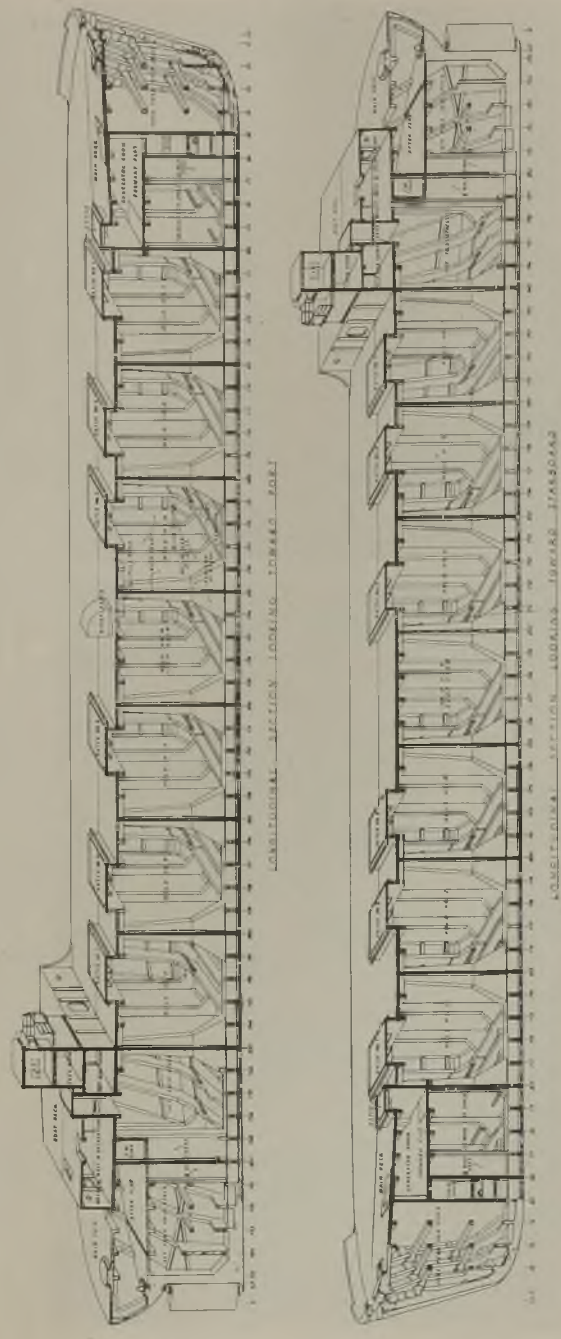


Fig. 8—Isometric longitudinal sections of B7-DI dry cargo hulls built at San Francisco.

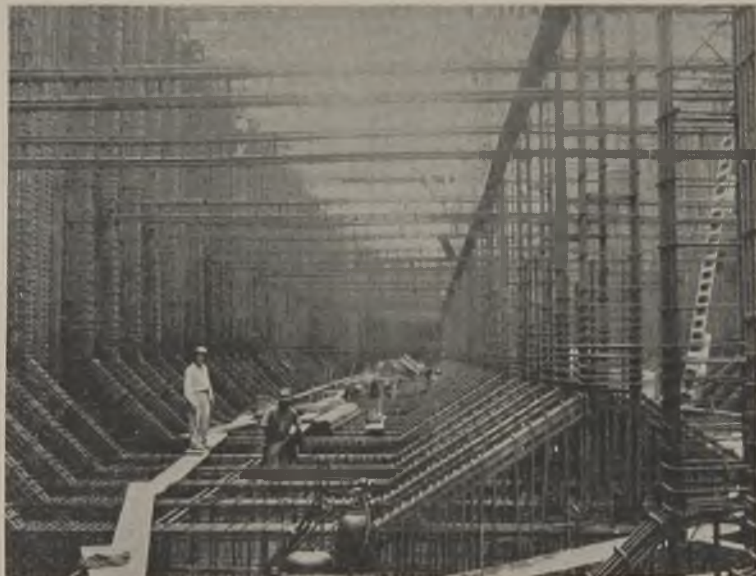


Fig. 9 (left)—Beam, haunch, and rib reinforcing in place prior to setting forms and placing first concrete in National City hull and beams to near top of haunches.

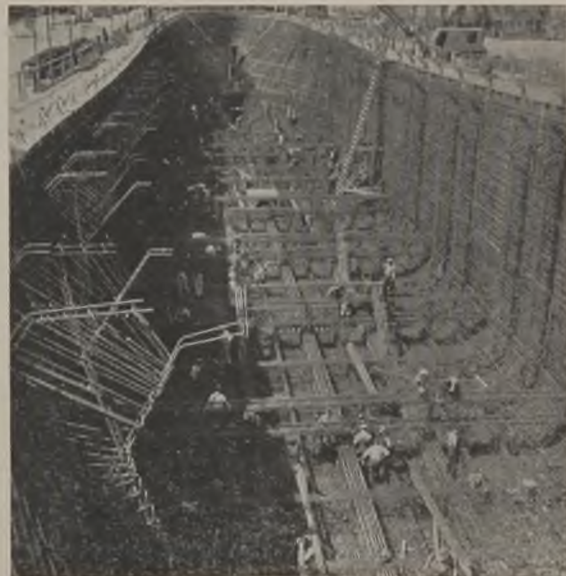
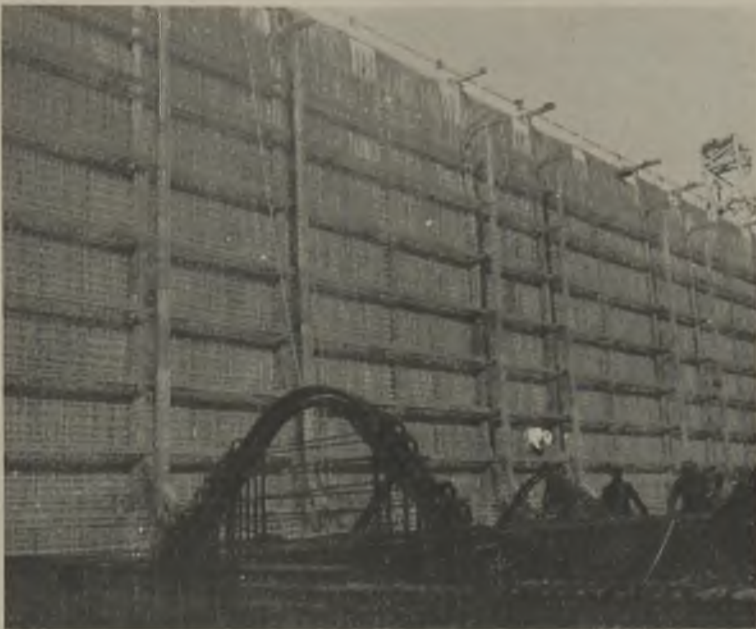


Fig. 10 (right)—Beam, bulkhead, and rib reinforcing ready for setting forms and placing first concrete in Tampa hull and beams to top of bilge.



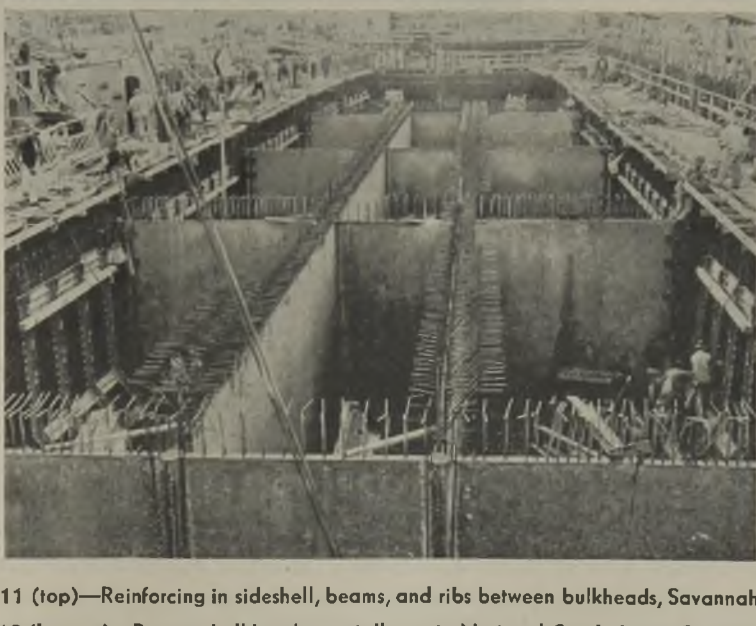


Fig. 11 (top)—Reinforcing in sideshell, beams, and ribs between bulkheads, Savannah hull.

Fig. 12 (bottom)—Precast bulkheads partially set in National City lighter. Concrete cast in bottom, sides, and deck and at intersections welded structure into a reinforced monolithic unit. One was built in 6½ days.

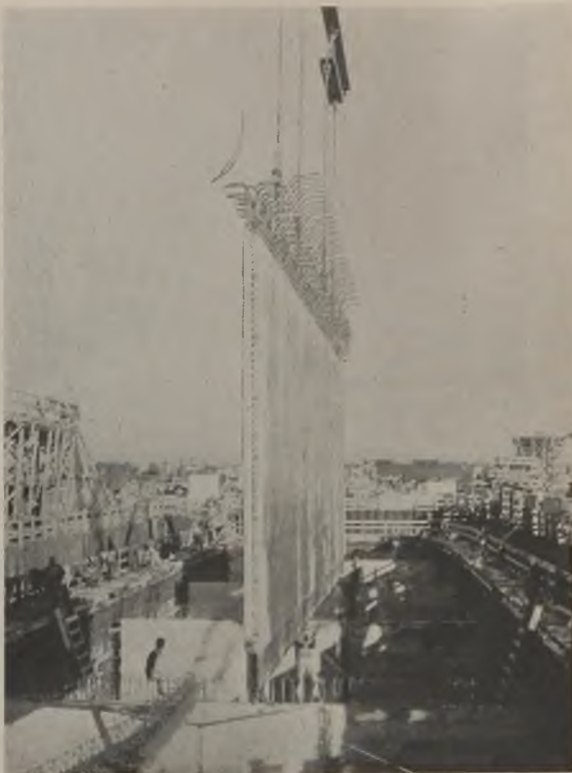


Fig. 13—Setting 22-ton precast length of longitudinal bulkhead in National City lighter. All precast bulkheads were set in this simplified hull in 3 hours.

22 to 24 tons each, were set in less than three hours (Fig. 13). There was no welding of bottom and side connections as protruding bar detail was shaped to become interlocked with steel in the bottom, side shell and deck. Longitudinal steel, ⅝ in. round at 6-in. centers, protruded from both ends of the precast lengths of longitudinal bulkhead and was welded to similar bars which pierced the transverse bulkhead at the intersections. 15-in. slots where this welding was done were concreted separately before setting the deck forms. The bottom slab was 7½ to 9 in. thick, the sides 8 in., and the deck 7 in.

With straight slab construction of these thicknesses, absence of ribs and beams, and with the reinforcing steel spaced to allow general access for the internal vibrators, these hulls were designed for efficient concrete construction. One of them was actually built and launched in 6½ days. Because of the thicker walls and surer placing of the 2 to 3-in. slump concrete, no cracks or leaks developed in the shells during hydrostatic testing. This radically new design was far better suited to reinforced concrete than any of the earlier designs, which consisted largely of the substitution of a reinforced concrete member for the corresponding member of a steel ship, and eliminated almost entirely the complications in steel setting, form construction, and concrete placing which caused relatively slow construction and high costs in the earlier vessels. It is a matter of regret that the idea of flat slab design without frames and beams could not have been carried out in a larger vessel, similar to the oil barges, while the program was still under way, and it is definitely the writer's opinion that any future concrete ship program should start with this type of design—or not start.

CONCRETE MATERIALS

In hulls containing about 3000 cu. yd. of concrete, nearly 40 tons of dead weight could be converted to cargo capacity by each reduction of one pound in the unit weight of the concrete. Since this multiplied to very appreciable tonnage when the 144-lb.-per-cu.-ft., natural aggregate mix was reduced 20 to 40 pounds in weight by the use of various types, amounts and conditions of light-weight aggregate, favorable consideration was given to the use of light-weight material. Tests were made to determine whether physical properties of light-weight aggregate concrete, such as flexural and compressive strength, diagonal tension, shrinkage, toughness, bond and impermeability, and other properties less clearly evaluated, were of sufficient quality to warrant acceptance of such concrete as a safe practical substitute for concrete made of natural aggregate, also whether its strength was sufficient to meet the rigid specification requirement of 5000 psi compressive strength at 28 days for hull concrete.

It appeared from these tests that light-weight aggregates could be used with confidence, but, as an additional factor of safety and to create slightly greater toughness, it was agreed to include in the mix a minimum of 15 percent natural sand. In the National City yard from the start (where light-weight fine aggregate was difficult to obtain, and expensive) and later with St. Louis coarse Haydite at Savannah and Tampa, all natural fine aggregate was used. There has been no evidence that this large percent of natural sand resulted in any measurable or observable benefit to the hulls and it did add about 400 tons extra weight. At both Savannah and Tampa no difference could be found between the hulls with the different amounts of natural sand that could be positively correlated with the difference in natural sand content.

Typical average physical properties of the various aggregates used, including the natural sands, are shown in Table 2.

Haydite, a manufactured light-weight fine and coarse aggregate, constituted the largest volume of light-weight material used. At one time or another Haydite was used in all yards and came from three plants—

TABLE 2—AVERAGE PHYSICAL PROPERTIES OF AGGREGATES USED AT THE FIVE CONCRETE SHIPYARDS

Yard	Aggregate	Aggregate Size Designation	Weight Per Cu. Ft. (dry loose) lb/c.f.	Absorption %	Bulk Specific Gravity (dry)	Percent Retained Each Size, By Weight										
						¾"	½"	⅜"	No. 4	8	16	30	50	100	P	F. M.
Savannah	E. St. Louis Haydite	½"	40	23	1.09		8	31	46	10					5	6.27
	E. St. Louis Haydite	⅜"	41	21	1.14			7	59	28					6	5.67
	E. St. Louis Haydite	No. 8	50	20	1.24					8	34	24	11	8	15	2.78
	Natural Sand	No. 8	99	1	2.62				1	3	14	31	29	16	4	2.44
Houston	Kansas City Haydite	⅜"	42	24	1.25			10	61	20	5	1	0	1	2	5.60
	Kansas City Haydite	No. 8	51	22	1.43					15	33	24	15	6	7	3.15
	Natural Sand	No. 4	101	1	2.60				9	16	13	14	36	11	1	3.11
Tampa	E. St. Louis Haydite	½"	35	12	1.12		14	30	46	5	2	0	1	0	2	6.30
	E. St. Louis Haydite	⅜"	37	15	1.14			6	60	28	4	0	0	0	2	5.58
	E. St. Louis Haydite	No. 8	46	19	1.50					7	31	23	15	11	13	2.69
	Nodulite	½"	42	8	1.29		9	41	44	2	0	0	0	1	3	6.34
	Nodulite	⅜"	46	8	1.42				44	50	2	0	1	1	2	5.25
	Nodulite	No. 4	61	7	1.86				1	15	31	17	12	11	13	2.91
	Natural Sand	No. 16	101	1	2.62						3	25	45	25	2	2.02
S. San Francisco	Kansas City Haydite	No. 8	55	21	1.49					16	38	21	11	8	6	3.25
	California Haydite	½"	46	13	1.50		13	48	33						5	6.68
	California Haydite	⅜"	47	12	1.51			2	75	18					5	5.74
	California Haydite	No. 8	65	12	1.74					29	32	17	8	7	7	3.47
	Natural Sand	No. 8	95	2	2.57				0	13	24	24	18	15	6	2.84
National City	Kansas City Haydite	½"	43	17	1.32			13	65	23						5.91
	Kansas City Haydite	⅜"	47	16	1.39			1	32	40	27					5.07
	Kansas City Haydite	No. 8	50	11	1.55					18	35	23	13	6	5	3.31
	California Haydite	¾"	44	8	1.34	1	46	34	17	2						7.27
	California Haydite	½"	48	8	1.50		7	49	42	2						6.61
	California Haydite	⅜"	49	8	1.60				79	19	2					5.77
	Rocklite	¾"	37	16	1.19	2	75	21	2							7.76
	Rocklite	½"	42	19	1.21		14	62	23	1						6.89
	Rocklite	⅜"	44	19	1.25			5	89	5	1					5.98
	Airox	¾"	39	5	1.10	3	49	38	10							7.45
	Natural Sand	No. 8	93	0.7	2.63					3	19	27	36	12	3	2.56

San Rafael, Calif., Kansas City, Mo., and East St. Louis, Ill. It is a crushed product, rough, sharp and angular. Briefly, it is made by crushing and screening the clinkered product of burning suitable shale in a rotary kiln at a temperature of about 2100 to 2000 F. until the degree of vesiculation necessary to produce material of the desired unit weight and strength is obtained.

Rocklite was manufactured in a small new plant at Ventura, Calif., and the entire output was used at the National City yard. It was produced as an individual, nearly spherical, particle with a thin shell and vesiculated interior in coarse sizes only. It was made by crushing, screening, and burning appropriate sizes in a rotary kiln at a temperature of about 2170 F. Despite the irregular shape of the particles as they enter the kiln, the processing and expansion during burning results in a remarkably well-rounded material.

Nodulite was made in a large new plant at Ellenton, Fla., and was used only at the Tampa yard. It is a coated, vesiculated, spherical particle produced by burning in a rotary kiln at about 2050 F. The nodules were prepared for kiln feed by a "nodulizing" process in which pulverized and dried Fuller's earth was fed into a large revolving drum containing adjustable water sprays and came out rather hard damp balls ranging in size from 1/16 to 1 in., which shrink considerably in burning. The nodules were dusted with fine silica sand to prevent them sticking together when burning. After burning, all oversizes and some excess intermediate size was crushed and rolled, and blended in screening with the kiln-run fine nodulite aggregate.

Type II modified cement was selected for the hull concrete because of its moderate heat of hardening (important because of the massive ribs and beams in relation to thin shell structure), its expected durability, and its better resistance to sulfate waters. For these reasons the manufacturers were encouraged to supply cements that were as far toward low heat and sulfate resisting compositions as practicable and yet give adequate strength at 10 and 28 days. Table 3 lists the general properties of cements used in the various yards. More than one column of data means that more than one brand of cement was used in that yard. Moderately high fineness was considered desirable because of the importance of reducing bleeding for the enhancement of water tightness and the benefit of workability.

MIXES AND PROPERTIES OF HULL CONCRETE

The principal mixes and their average properties are shown in Table 4. During the investigations of light weight aggregates by separate laboratories working for the Commission and for the builders, trial mixes were made and proportions arrived at for use in the yards. To secure an ample margin of strength over the required 5000 psi at 28 days and high values of tensile strength and watertightness, about 9 sacks of cement per cu. yd. were used in most mixes. With this much cement and a maximum aggregate size of 3/8 to 1/2 in. there were few if any mixes (at a slump from 2 to 6 in.) that were not highly workable and readily and solidly placed by the minimum amount of vibration essential to the placing of any of the mixes. To one whose experience has been mostly with relatively lean mixes and aggregates graded to comparatively large maximum sizes, all the mixes looked like mortar and extremely workable.

As is usual on a concrete job, when something goes wrong and placing results sometimes are not all they should be, even these mortar mixes would come under fire. There were solemn discussions of the significance of quarter-bag changes in cement content (3 per cent), changes of a few percent in the amount of sand or fines in the sand, minor shifts here and there in the grading curve, quarter or half-inch differences in slump. Actually there were very few imperfections, other than those due to

TABLE 3—AVERAGE CHEMICAL AND PHYSICAL PROPERTIES OF TYPE II PORTLAND CEMENT USED AT THE FIVE CONCRETE SHIPBUILDING YARDS

Yard	Savannah			Houston	Tampa	S. San Francisco			National City
Portland Cement	A	B	C	D	E	F	G	H	I
Amount Tested, Bbls.	11,500	43,000	11,500	26,720	172,000*	24,000	138,000	54,500	168,500*
Chemical Analysis									
CaO	62.0	64.6	63.6	64.9	64.5	65.0	65.0	64.5	64.2
SiO_2	21.3	22.3	21.8	22.1	22.1	21.4	22.7	23.0	22.9
Al_2O_3	4.7	4.5	5.3	5.4	4.8	5.4	4.6	4.3	4.4
Fe_2O_3	4.5	3.6	4.7	4.0	4.3	4.3	3.3	3.4	2.6
MgO	4.4	1.4	1.6	0.8	1.1	1.6	1.4	1.7	2.7
SO_3	1.8	1.7	1.7	1.5	1.5	1.7	1.5	1.5	1.8
Loss on Ignition	0.8	1.2	0.6	0.7	1.2	0.6	0.6	0.8	0.8
Insoluble Residue	0.1	0.2	0.1	0.2	0.2	0.1	0.1	0.1	0.1
Al_2O_3/Fe_2O_3	1.1	1.3	1.1	1.3	1.1	1.2	1.4	1.3	1.7
3CS	46.1	49.3	46	48.6	52	55	52	48	49.
3CA	5	6	6	7.5	6	7	6.5	5.5	7
Total Alkalies Expressed as Na_2O		.16	.62	.65	.31	1.06	.57	.47	.57
Physical Tests									
Fineness Specific Surface Sq. Cm. per Gram	1850	1695	1885	1770	1860	1785	1880	1935	1855
% Passing No. 325 Sieve	87.0	83.3	91.4	92.7	87.0		92.3	93.6	93.5
Mortar Strengths									
Tensile, 1" Briquets									
3 day	290	255	275	305	275	245	300	275	270
7 day	365	355	355	390	360	350	375	370	360
Compression, 2" Cubes									
7 day		1830	2120	2775	2395	2790	2710	2680	2695
Time of Set									
Initial	3 hr. 40 min.	3 hr. 15 min.	3 hr. 25 min.	O.K.	3 hr. 20 min.	O.K.	O.K.	O.K.	4 hr. 5 min.
Final	7 hr.	7 hr.	6 hr. 40 min.	O.K.	7 hr.	O.K.	O.K.	O.K.	7 hr. 55 min.
Normal Consistency, %	23	23	23.2	24.9	23.8				
Soundness, Autoclave, %	.16	.01	.01	.19	.02	.07	.15	.04	.16

*Estimated.

TABLE 4—SUMMARY AVERAGES OF TEST RESULTS OF PRIMARY CONCRETE MIXES AT FIVE CONCRETE SHIPBUILDING YARDS

Yard	Mix No.	Portland Cement, Type II	Lightweight Aggregate: Fine	Lightweight Aggregate: Coarse	Mix Proportions, % By Solid Volume: Ntrl. Sand	Lightweight Aggregates: Fines	Lightweight Aggregates: 3/8"	Lightweight Aggregates: 1/2"	Lightweight Aggregates: 3/4"	Admixture % by Wt. of Cement	No. of Cylinder Sets	Cement Content—sk/c.y.	Water Cement Ratio by Wt.	Slump—in.	Mixing Time—Min.	Unit Wt. of Fresh Concrete	Modulus of Elasticity at 28 days—10^6 psi	Compressive Strength, 6"x12" Cyl., lb. per sq. inch: 7 day	28 day	60 day	365 day	Combined Sieve Anal. by Solid Vl. Percent Retained On Each Size: 1/2"	3/8"	4	8	16	30	50	100	P	F. M.
Savannah	A-1	B	E. St. Louis Haydite	E. St. Louis Haydite	15	35	50			0.5	99	9.9	.47	5.3	11	109	2.410	3760	5610	6165	7290	0	3	29	18	18	14	6	6	6	4.17
	B-1	B	" " "	" " "	25	25	50			0.5	18	9.7	.44	3.6	14	112	2.650	4590	5610	6190	7330	0	2	31	19	12	16	8	8	4	4.15
	C-1	B	———	" " "	50	—	50			0.5	14	9.5	.44	3.6	9	122	3.685	4185	6440	7215	8050	0	2	31	20	8	13	17	7	2	4.12
	C-7	C	———	" " "	50	—	30	20		0.5	31	9.5	.41	4.2	13	122	3.612	5060	6920	7380	7890[c]	2	8	28	11	9	12	20	8	2	4.15
Houston	No. 1	D	Kansas City Haydite	Kansas City Haydite	15	45	40			0.5	85	9.6	.43	5.9	10	115	2.567	4040	5630	6235	7470[b]	0	5	28	16	18	13	12	4	4	4.22
Tampa	H-1	E	E. St. Louis Haydite	E. St. Louis Haydite	15	35	50			0.5	76	9.7	.49	6.2	7	108	2.24	4085	5875	6460	6725	0	3	26	14	12	14	13	12	6	3.75
	H-2	E	———	" " "	50	—	50			0.5	7	9.5	.46	6.4	7	120	2.62	4560	6330	6800	7150	0	4	30	13	4	13	24	10	1	3.92
	HN-1	E	———	*See Note	40	—	25	35		0.5	60	9.2	.42	4.1	25	117		4460	5655	6080	6585[d]	3	10	27	16	2	11	20	9	2	4.26
	N-3	E	Nodulite	Nodulite	15	35	20	30		0.5	47	9.2	.50	4.5	25	116	2.42	4205	5645	6355	7085[e]	2	14	21	15	13	10	12	7	6	4.28
S. San Francisco	AN9A	F	California Haydite	California Haydite	45	—	20	35		0.5	15	9.0	.43	6.1	3	125	3.25	3975	5850	6685	7595	1	15	24	12	14	12	9	9	4	4.36
	A	G	"	"	20	30	20	30		0.375	86	9.2	.40	4.0	3	120	3.123	4525	6355	7050	8155[f]	1	20	23	15	14	10	6	7	4	4.66
	B	G	"	"	30	20	20	30		0.375	41	9.2	.38	3.2	3	123	3.00	4485	6400	6955		1	18	26	15	14	9	8	5	4	4.69
	C	H	Kansas City Haydite	California Haydite	15	35	20	30		0.375	80	9.2	.40	4.1	3	118	2.845	4395	6140	6825		1	21	24	10	18	12	7	4	3	4.75
National City	YL-3	I	———	"	50	—	15	35			36	9.3	.46	3.1	3	126	3.62	3285	5085	5815	7280	2	17	29	4	12	12	15	5	4	4.48
	EYL-3	I	———	"	50	—	15	35		0.375	46	9.3	.46	3.5	3	128	3.55	4675	5970	6570	7350	3	19	25	5	9	11	17	6	5	4.41
	EK-3	I	———	Kansas City Haydite	52	—	14	34		0.375	193	9.75	.44	3.5	3	127	3.30	4900	6370	7095	7600	0	4	28	13	12	16	17	6	4	3.97
	ER-3	I	———	Rocklite	48	—	16	36		0.375	71	9.3	.45	3.5	3	125	3.33	5025	6330	6780	7715[a]	4	23	19	5	8	13	18	6	4	4.45

*Coarse lightweight Aggregate:
3/8" Nodulite
1/2" E. St. Louis Haydite

[a] 365 day average based on 67 cylinders
[b] " " " " " 48 "
[c] 365 day tests reported for 17 cyl. (out of a total of 31) to date

[d] 365 day tests reported for 21 cyl. (out of a total of 60) to date
[e] " " " " " 30 " " " " " 47 " "
[f] " " " " " 44 " " " " " 86 " "

leakage or failure of forms, that a little more well-timed vibration at that spot would not have eliminated regardless of the mix or the slump.

The question of consistency was much discussed in the concrete ship program. Correctly recognizing that vibration would have to be applied to every shovelful of concrete in a hull if reliable results were to be obtained, early investigators of the mixes assumed that, since such thorough vibration would obviously be necessary and, in this day of vibrated concrete, would not be impracticable of attainment, advantage could be taken of the vibration to use concrete having a slump of 2 in. Although desireable, this did not sufficiently recognize the amount of slump loss (sometimes none, often ¾ to 1 in., sometimes as high as 2 in.) that occurred from the time concrete was delivered from the mixers until it was all placed in the form, nor the human equation in obtaining complete vibration. Nevertheless, the first hull in the program was cast at National City with an average slump of 3 in. and no admixture. As in most things, the first attempt left some things to be desired and in the next hulls, average slump was increased to 4 in. where it remained for the side shell concrete but was returned after a few hulls to 3 in. for the bottom and deck concrete and excellent results were attained.

Before South San Francisco, Savannah, Houston and Tampa yards were ready to place their first concrete and prior to the writer's connection with the program, decision had been made to use a retardant admixture because some were fearful of cold joints. (Actually, reactivation of the previous lift is inconsequential, as was amply demonstrated at the construction joints. The main thing in preventing cold joints, regardless of age or hardness of previous lift, is to see that the bottom few inches of next lift is vibrated adequately.) This admixture also increased 28-day strength about 5 percent which sometimes was needed, and increased the slump from 4 to about 6 in. with the same water content. (It did not decrease slump loss however and slightly increased bleeding, even at the same slump as concrete without it.) Consequently first concrete in these yards was mixed at the 6-in. slump on the theory they needed all the slump they could get and after all, with the admixture, they were using only as much water as would be required for a 4-in. slump.

However, with the influence of the National City contractor, who fully understood and appreciated the importance of well placed medium-low-slump concrete, and who did not increase his slump when the admixture was added, and with gradual improvement of concrete control and experience in the other yards, slumps were brought down gradually until practical minimum medium slumps were generally in use throughout the program. Final average slumps were seldom much over 4 in. except when conditions were such that slump loss was consistently an inch or more. Fig. 14 shows maximum, minimum, and average slumps for the sideshell concrete for each hull in each yard. Slumps for bottom and deck concrete averaged up to an inch lower. The South San Francisco yard was particularly successful in getting down to low slumps (Fig. 15) and at the same time getting excellent, well-placed concrete. It takes high class performance to get such results with 3-in. slump concrete as delivered at the hull or a net slump of 2¼ in. at the forms after an average loss of ¾ in.

With aging and drying of the earlier hulls and due to the various construction, testing, and natural loadings, fine hairline shrinkage and deflection cracks appeared in various beams and slabs. There was nothing abnormal or indicative of failure in these cracks. In fact, they were abnormally inconspicuous as concrete work goes. All were less than one hundredth of an inch in width; most were barely visible, probably not over two thousandths inches wide. Although of a character that would go unnoticed in an ordinary structure and the whole be rated

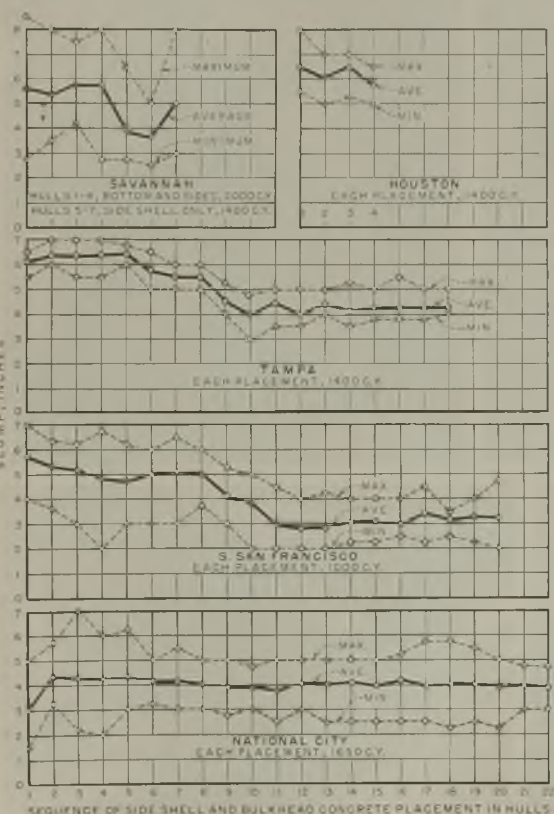


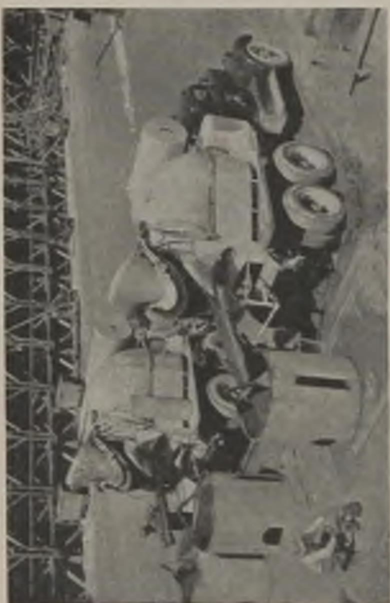
Fig. 14—Slump record of sideshell and bulkhead concrete.

Fig. 15 (top left)—Excellent results were obtained with medium-low-slump concrete when it became generally recognized that good work depended on very thorough vibration regardless of slump. See figure 14. This concrete produced excellent results in the San Francisco hulls where best concrete work in program was done.

Fig. 16 (top right)—At Tampa a slump test was made of each $3\frac{1}{2}$-yd. truck load. Note medium low slump.

Fig. 17 (bottom left)—Outside forms at all yards were arranged to pull back from the hull in full-height units. This detail was used on the San Francisco hulls.

Fig. 18 (bottom right)—Opening panel at the top of improved Tampa wall form (below horizontal beams) served to guide concrete for lower portion before closing to form upper portion.



near perfection as far as cracking goes, some of the cracking of this ordinarily negligible character caused difficulty in concrete ship delivery because in certain prescribed hydrostatic tests some of these cracks in thin panels (often due to deflection from the high load of the test pressure) permitted a slight showing of moisture and caused rejection by the ship classification society. Repeated repairing and retesting was necessary with resultant delays and the inevitable questioning of the concrete mix. Particularly the tensile strength of concrete containing Haydite light-weight, fine aggregate was questioned despite the fact that the tensile strength in concrete structural design has quite universally been regarded as zero. Consequently, since the hull mixes were little more than mortars, many briquett tensile strength tests were made and are reported in Table 5. It is considered of interest that, except for those without curing, all are substantial high values regardless of the character and amount of fine light-weight aggregate.

CONCRETE PRODUCTION AND MIX CONTROL

Aggregate was received in open cars or barges and was rehandled to stock piles and then to batcher bins by clamshell buckets or belt conveyors. The latter were less desireable and resulted in considerable separation because they necessarily deposited the material mostly at one point and there was considerable undersize material in the light-weight aggregates, especially the Haydite.

All aggregates were manually batched by weight. Most yards used bulk cement and batched it by weight. Water was measured by volume, usually in wobbly-disc water meters of standard make. There was nothing unusual about any of this equipment necessitated by use of light-weight aggregate. Some was new and some had been used previously. Automatic batching would have been preferable.

Mixing equipment varied. At the South San Francisco yard concrete was centrally mixed for 3 minutes in two 2-yd. double-cone tilting mixers and conveyed to the hull side in 4-yd. truck mixers. At National City one-yard batches were mixed 3 minutes in paving mixers at the hull. At the other three yards the concrete was entirely mixed in truck mixers a minimum of 6 to 7 minutes.

Although hourly moisture samples (and in some yards, specific gravity samples) were tested and used as a basis for batch weight and water adjustment, the basis of mix control was slump and unit weight. Regardless of aggregate tests, water was adjusted to produce the desired slump. Regardless of specific gravity tests, batch weights were adjusted on the basis of unit weight tests to produce concrete of the proper yield and cement content. Regardless of yield and computations from unit

TABLE 5—TENSILE STRENGTH, HULL CONCRETES, PSI*

Percent Natural Sand	Moist Cured		Cured 7 days, then in lab. air until	Lab. air
	28 days**	28 days	28 days old	28 days
St. Louis Haydite concrete, Savannah mixes				
All Haydite	410	455	420	—
15% Natural sand	445	430	395	—
25% " "	470	500	450	—
50% " "	475	515	440	—
St. Louis Haydite concrete, Tampa mixes				
All Haydite	—	490	400	350
15% Natural sand	—	455	380	320
25% " "	—	600	530	430
50% " "	—	590	400	390
Kansas City Haydite concrete, Houston mixes				
All Haydite	—	505	515	310
15% Natural sand	525	550	560	350
25% " "	560	565	550	360
50% " "	545	580	560	370
San Rafael Haydite concrete, San Francisco mixes				
All Haydite	—	550	530	450
15% Natural sand	—	550	545	375
25% " "	500	540	485	475
50% " "	470	525	525	220
Nodulite concrete, Tampa mixes				
All Nodulite	430	480	433	329
15% Natural sand	435	483	428	356
25% " "	445	578	450	362
50% " "	475	529	418	362

*Each value is the average of 4 to 6 briquette tests. Sixty tensile strength tests of 20 mixes in 4½-inch dia. specimens showed an average strength 11% greater than corresponding briquette tests.
**These data from series of check tests.

weight, whole batches were measured for volume in calibrated containers of appropriate size. By means of such tests and such procedure, the mix was generally closely controlled within the limits desired. Net W/C was usually less than .50, often less than .46 by weight. Although W/C was the basis of the mix for concrete quality it was not the means but the result of mix production control.

At the beginning of concrete operations it was taken for granted that uncontrollable variations in slump and yield would result unless the highly absorptive light-weight materials were completely saturated.

When concreting started, materials had been on hand for some time and were well soaked by having been sprinkled for days previously. Naturally no difficulty was experienced with control. As concrete production increased and new supplies of light-weight aggregate arrived, there was not always time to saturate the material completely before it had to be used, as it required several days for complete saturation of the material regardless of how wet it was kept. The result was that aggregates of varying degree of saturation were used but no particular additional difficulty was noticed in maintaining slump and yield by the methods of control described above and slump loss was no more than usual.

It was then suspected that the original idea that light-weight aggregate had to be saturated for uniform results was not only incorrect but that better control and better concrete (at least lighter weight concrete) could be made if the aggregates were used in the driest practicable condition. Enterprising concrete engineers for the builder at South San Francisco completed an important series of tests of ship concretes using saturated and dry coarse aggregate and showed that compressive strength was increased slightly and unit weight was decreased about 3 lb. per cu. ft. (and remained so in wet storage for 365 days) by use of the dry materials. Contemporary investigators* found that concrete using material soaked only 30 minutes (as it would be if used dry in concrete) was at least twice as resistant to freezing and thawing as concrete using saturated material. Earlier tests† showed that drying shrinkage after 2 years was no different for concrete made of dry Haydite than for concrete made of saturated Haydite.

As this information became known in the other yards there was a marked decrease in efforts to saturate the material. Control of slump and yield on the job proved to be no more difficult than for the saturated material. Reason for this is that all the absorption that takes place at a rapid rate was completed prior to discharge of the concrete from the mixer. Consequently, when sufficient water was added to the batch to provide for the immediate absorption of the aggregate, and concrete was mixed to the proper slump before it was discharged, there was no more than usual slump loss as a result of using dry or only partially saturated material. When the material was rained on it was only partially saturated and this merely changed the amount of extra water which had to be provided for immediate absorption. From laboratory data on the absorption capacity of the material and the water content when batched, the effective W/C was computed.

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*Carlson, R. W. and Forbrich, L. R.

†F. E. Richart and J. E. Keranen, "Shrinkage of Haydite and Sand-Gravel Concrete", 1936 report of A. S. T. M. Committee C-9 (Appendix IV).

Slump and unit weight tests were frequently made for the record and as a basis of mix adjustment. At Tampa, a slump test was made on each $3\frac{1}{2}$-yd. batch (Fig. 16, p. 154) but not before its discharge was begun if it appeared to be about the right slump. Sets of 6 by 12-in. cylinder specimens were made for tests at various ages. Each set represented about 250 cu. yd. of concrete. Simultaneous aggregate samples were taken from materials measured for the batch from which concrete specimens were made. The tested batch was not selected at random but was spaced at stated intervals to avoid preferential selections and permit the previous taking of aggregate samples. Typical average results are shown in Table 4.

FORMS

Except for the use of steel forms for the interior of the mid-section at National City and Tampa, forms were of wood. Sheathing of wood forms was generally $\frac{3}{4}$-in. plywood except for the bottoms of the Tampa hulls where 1 -in. T and G cypress of random widths was used.

Since all concrete placing was from inside the hulls there were no openings in the outside forms (except that in later hulls at Savannah and Tampa, there were slots at construction joints and in all bottoms for clean out). Outside forms for the entire hull were completed before any reinforcing or inside forms were placed in any of the yards. In some yards these forms were built in full height sections on towers which could be rolled back from the hull for launching (Fig. 17, p. 154). With relatively minor repair of sheathing, outside forms were reused as many as 6 times in completion of the contract and were still good for more.

Inside forms for the bottom lift were simplest of all, for the most part forming the sides of beams and ribs in which concrete could be placed through the top. Forms for the first lift in the bulkheads and the turn of the bilge usually required a closely spaced series of openings at mid height, about 2 feet above the bottom.

Forms for the second lift, shell and bulkhead walls, varied considerably for the different hulls, the principal cause of difference being the many horizontal beams in the hulls in which the transverse ribs were spaced 10 ft. 8 in. Although the open tops of these beams were admirably arranged to admit the placing and vibration of concrete, being usually about 4 feet apart, there was some concern as to the reliability of concrete placing results when attempting to place the concrete in the thin panels between beams in a 4 foot lift. This question was not answered to the satisfaction of all when forms were stripped since, in the similar hulls at Houston and at Savannah there was no great difference in results, none being perfect without certain repairs, yet at Houston the concrete was placed beam to beam while at Savannah it was placed in shallow lifts through closely spaced openings between the beams as well as through the beams.

Inside forms for the Tampa ship, the other hull having horizontal beams, were built with the top half of the slab form between beams made in long removable panels which were placed in position only after the concrete in the lower half of the shell or bulkhead slab between two horizontal beams had been placed and vibrated. The panel was then set in and the concrete brought up to beam level (center of the beam or lower) placing through the beam. Because these forms were insecure under form vibration and resulted in bulges and overrun yardage in the hulls, they were changed to a double panel held in the center by a vertical waler bolted through the shell at each beam (Fig. 18, p. 154). In the top half of each panel was a full sized panel which opened out at the top to form a wide chute to direct the concrete into the forms to fill the lower half of the lift from beam to beam. These forms were a great improvement and reduced bulging considerably.

Fig. 19—Shallow-lift steel forms were placed one lift at a time as the concrete was placed at National City and provided needed accessibility for placing concrete.

From the standpoint of construction advantages, hulls with closer ribs and fewer horizontal beams were distinctly superior. In the National City hull, the only forms (steel) in place in the mid section at the start of the second lift of concrete placing were those anchored on the faces of the ribs and the first series of steel panels (12 to 15 in. high depending on location) around the bottom of the lift just above the construction joint. Successive series of U-shaped steel panels were installed after each previous series had been filled with concrete (Fig. 19). At the San Francisco yard a shutter-type wooden form was used with excellent results. The feature of this form was a tall double frame forming the panel between ribs. In the tall rectangular openings of these frames were sliding shutters, 12 in. high, each originally mounted a foot above its final position (Fig. 20). After each 12-in. layer of con-

Fig. 20 — Drop - shutter forms at San Francisco permitted 12-inch lifts of concrete with good accessibility for placing shell and bulkhead concrete. Box panels (right) were used similarly in warped and tapered surfaces.

crete was placed, another round of shutters was dropped into place and concrete raised another foot. Both types of forms afforded splendid accessibility for placing and vibrating the concrete. In this, as in all except the National City forms, frequent openings in the sides of the rib forms were provided for introduction and vibration of the concrete.

Although forms were generally as satisfactory for placing concrete as hull design would permit, and changes were made to improve their security, over-run in concrete placed (from 1 up to 10 percent of the computed volume) was never eliminated. This was a serious addition to the deadweight of the hulls and presents a challenging problem to form designers at the outset in any further construction of this kind.

HANDLING AND PLACING THE CONCRETE

There was nothing unusual about methods of transporting and handling the concrete from mixer to the forms. At Tampa mixer trucks discharged into bottom dump buckets which were swung by crawler-type cranes to buggy-loading hoppers on a runway at proper elevation for each of the 3 major placements. At the other yards, mixers discharged into skip buckets in elevators at the side of the hull and these were emptied into buggy loading hoppers at deck level and concrete was dropped through drop chutes from the buggies to the proper level or from the buggies into position in the deck. Exceptions to the latter were for the decks at National City and South San Francisco where gantries or crawler cranes swung 1-yard buckets of concrete from the mixers to position in the deck and the concrete was dumped directly in place with some spreading from the bucket. At these two yards, location of the construction joint high, so that very little shell and bulkhead had to be placed with the deck and relatively less congestion of steel in the deck beams at these yards, made such procedure practicable where it would have resulted in seriously unsuccessful placing in the more difficult deck structures at the three eastern yards.

At Savannah, all concrete for all except the beam and slab portion of the deck was dumped from the buggies into shallow batch boxes from which it was passed by the bucketful into place, even into the bottom slab. At other yards, except in tight spots where it was impracticable to do so, the discharge of drop chutes was directed into the forms by means of small wooden chutes as the buggies were slowly dumped above in accordance with signals from a semafore at the hopper which was actuated from below. In the tight spots concrete was dumped into a batch box and shoveled or bucketed into the forms. In practically all cases, it was necessary to run an immersion type vibrator in the concrete as it entered the forms to keep it flowing out of the way from the chute or bucket.

There were many special problems in placing hull concrete that had to be recognized but their solution was largely a matter of application of basic principles of concrete placing. For instance, in early hulls poor results were obtained when the bottom concrete was started in one or both of the peaks and developed down the slope toward the mid-section. Naturally, when vibration was transmitted back through steel and forms, this concrete settled and here and there came apart and required later repair because it was not tight. This was corrected by requiring all placing to begin at the lowest point and work up the slope, thus letting transmitted vibration further compact the concrete. Even so, sometimes crews would place concrete in the bilge beams and slabs in the peaks before placing concrete below them in the keelson and adjacent flats and beams in succession toward the bilge. Trouble would result in the bottom placement when concrete was placed higher in one member, such as a transverse bulkhead, than the top of intersecting members, such as longitudinal beams, before the latter were placed and vibrated. Invariably, this resulted in pulling concrete down in the bulkhead and a repair would be necessary. This was corrected when no concrete was placed above a specified level until all adjacent placing and influencing vibration in that area was completed up to that level.

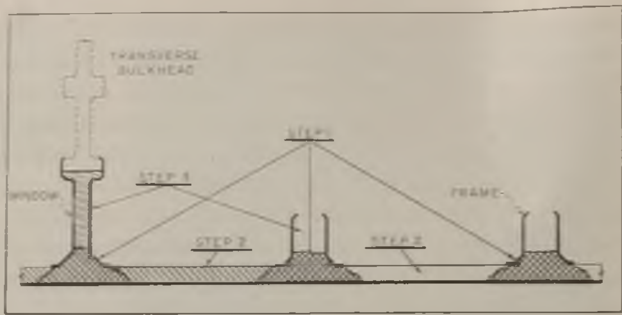

Fig. 21 — Sequence of concrete placing in hull bottoms.

The question of whether the first concrete in the bottom should be placed in the slabs or in the bottoms of beams and bulkheads was a perennial that was good for a debate anytime. Although good results could be and were produced in some cases, placing the first concrete in the slab, some reconstruction had to be done following this procedure. It was required that first concrete be placed and vibrated directly in the bottom of frame, keelson, and large longitudinal beams (but not low narrow ones which filled reliably at the bottom from the slab), and in bulkheads sufficient to fill completely the portion of the slab under such members with as little additional concrete in the members at that time as possible. See Fig. 21. This was required on the theory that good results would be best assured if the joining between slab concrete and beam concrete was made in the open rather than out of sight, under the toe plates of the form or splays, and results were good.

Height of lift came in for considerable discussion. This was not a matter for an arbitrarily selected dimension. Matters of relative accessibility for placing and vibrating the concrete and the sufficiency of vibration, required consideration as did those of form construction and the shape of the structure. There was no question but that all things being equal, a shallow lift of a foot or so (as used in the California yards) was ideal. However, the horizontal beams in the other hulls made such a lift prohibitive in form detailing. Houston elected to place the concrete beam to beam with vigorous form vibration on the panel. Savannah, in the same thin-shelled hull, cut closely spaced pockets between the beams and placed 2 or 3 shallow lifts through the pockets and 2 or 3 through the beams with the idea that frequent shallow lifts would avoid cold joints. Under the conditions, the beam-to-beam lift with good form vibration produced a tighter hull than the slow, bucket-by-bucket placing of the very shallow lifts with only such vibration as could be obtained from flexible shaft vibrators pressed intermittently against the steel and against the forms. At Tampa, the forms were built to open widely above the bottom half of each panel between beams; the top half was placed through the beams (Fig. 18). When form vibrators were used during the placing of each of these lifts, results were excellent but, even with the thicker shell than at Houston and Savannah, the immersion-type vibrators alone could not be used to sufficient advantage to get reliable results.

VIBRATION

The success of concrete placing in all parts of all hulls depended on the thoroughness of vibration. Variations within practical limits of slump or mix were far less significant in determining the outcome of a placing operation than whether or not vibration had been adequately comprehensive and sufficiently sustained for full consolidation of the concrete. There were very few imperfections, other than due to form leakage, that a little more local vibration at the right time would not have eliminated. With reinforcing as close and congested as it was and with the inherent sluggishness of light weight aggregate concrete, 7-in. slump required as thorough (though possibly slightly less sustained) vibration as 2-in. slump concrete. Not a shovelful of concrete at any slump could be reliably placed without vibration. As these facts became more generally recognized, there was a wider recognition of the desirability of taking full advantage of this necessarily meticulous vibration regardless of slump to reduce water content of the concrete and derive the well-known improvements in concrete properties to be gained by so doing (Fig. 15, 22, 23 and 26). This is borne out by the trend of the slump test records shown in Fig. 14 and it was of interest to note that the low minimum slumps recorded resulted in no imperfections which could be attributed to them.

The question of the propriety of vibrating the reinforcing steel was often raised and never settled for some, never existed for others. At National City a device (Fig. 24) was used to attach form vibrators firmly to the steel to insure its thorough vibration in the side shell. At Savannah in their last hull, No. 7, every reasonable effort was made to vibrate only the forms and avoid vibrating the steel. Between these extremes, there was a variety of conditions in which both were vibrated but the steel received considerable vibration in all cases except in the Savannah hull 7. No difference could be found in the strength and water-tightness of the compartments of the National City hull under hydro-static test whether or not the steel was positively and directly vibrated. Somewhat lesser leakage in hull 7 than in former hulls could not be attributed entirely to avoidance of steel vibration as workmanship during placing was noticeably improved as a result of intensive instruction before concreting started. Evidently the amplitude of steel vibration

Fig. 22 (left)—Newly deposited concrete in bottom flat, Tampa (step 2, Fig. 21). Note medium-low-slump, small-maximum-size, nodulite concrete.

Fig. 23 (right)—Medium low-slump concrete, Fig. 22, responds fully to action of immersion-type vibrator, extensively used in placing hull concrete.

Fig. 24—Attachment used at National City with good results to apply form vibration directly to reinforcement during concrete placing.

was dampened and reduced so much, as it penetrated into lower lifts of partly set concrete that might be incapable of responding to vibration and again becoming plastic, that no damage to the embedment and bond of the steel resulted.

Occasionally, and usually in the vicinity of a repair needed as a result of insufficient vibration, a loose vertical bar was found. Often incomplete vibration of any kind, or wet concrete and bleeding, resulted in lack of

Fig. 25 (left)—Form vibrators were used effectively at Tampa on inside forms to supplement immersion-type vibrators which could not enter panels. Note method of direct placing from concrete buggies above.

Fig. 26 (right)—Medium-low-slump concrete placed in gunwhale beam at Tampa. Temporary pipe spreader, set with steel, maintained bar spacings for full penetration of vibrator.

tight closure under horizontal bars, particularly those of square sections. Although such bars were always tightly held by the concrete at the sides and over the bar, this condition under the bar reduced bond strength and often contributed to leakage. For this reason only round (but effectively deformed) bars should be used in horizontal positions in such highly stressed water-tight concrete construction. It can be stated unquestionably that vibration of the steel by one means and another greatly aided placing of the concrete and caused the elimination of far more imperfections than it could possibly have caused.

Best vibration, as in ordinary concrete work, was obtained from the immersion-type vibrator when it could be gotten into the concrete (Fig. 23 and 26). Because often it could not, form vibrators were used considerably. These were found difficult to reliably coordinate with placing operations when they were not used on the same side of the forms from which concrete was placed (Fig. 25). For this reason, the practice of using them outside the hulls gradually lost favor as no concrete was placed from outside the hulls.

The difficulty of concrete placing would have been much less if at the start the importance had been realized of detailing reinforcement with occasional spaces where concrete could have flowed through readily but mainly so that immersion-type vibrators could be inserted to the depth of the new concrete. As an after-thought the simplest of such adjustments were worked out in the field (and the plans accordingly revised and approved) and the benefits in simplifying, placing and vibrating operations at these points were outstanding. But there were many other spots, mainly at intersections and in the peaks, too involved to adjust after work had started which could and should be so refined at the outset if such construction is again attempted.

CONSTRUCTION JOINTS

It was the general practice in the yards to cast the hulls in three major lifts with two construction joints, one just above the turn of the bilge, the other just under the rib haunches to the deck beams. The only exception to this was at Savannah where on the first four hulls the contractor elected to eliminate the lower of these joints but later changed to the general practice of the two joints mentioned. The lighters at National City, being so much shallower than the other hulls, were cast with only one construction joint just below the deck haunches.

At first some were concerned about the idea of having construction joints in concrete hulls but since there was ample evidence in current tests and experience that this was practicable without impairment of strength or watertightness, the practice was permitted and the problems of construction support of steel and forms were greatly simplified. In the first hull at each yard there were some imperfect lengths of construction joint which required repair. The builders quickly got this operation under control with generally excellent results.

It was soon learned that the first step in securing a good construction joint was to leave the surface of the concrete when placed well compacted and fairly even. Curing of the joint surface is important and this was continued with water until the next concrete was placed. As late as practicable after the concrete had hardened, the joint surface was treated to a full-scale sand blast of sufficient intensity to remove completely the film of surface material and fully reveal the aggregate over the entire area of the joint. Standard sand blast equipment was used. In all hulls, construction joints were made at the center of horizontal beams so that the joints would be wide enough to receive thorough treatment and not be merely the narrow width of shell thickness. Immediately prior to concreting, sand blast sand and construction debris were vigorously washed from the joint (and forms and steel above) with jets of various character from firehoses to air-water jets at 100 lb. pressure. There was an increasing appreciation of the importance of ample washout openings in the forms and some yards developed a nearly continuous opening at the joint in the outside forms.

Just ahead of concreting, the joint was coated with a soft mortar similar to that in the concrete and, as far as accessible, this was rubbed

SUCTION AND NOZZLE SIZES FOR VARIOUS USES OF SUCTION GUN

USE TO BE MADE OF GUN	1½"-45°Y SUCTION	1½"-45°Y NOZZLE	2"-45°Y SUCTION	2"-45°Y NOZZLE	NOTES
AIR WATER JET	¾"	¾"x 24"	¾"	1¼"x 24"	SIZES ARE STANDARD PIPE SIZES. STANDARD HOSE SIZES FOR SUCTION FIT NIPPLES OF THESE PIPE SIZES FAIRLY TIGHT. SUCTION HOSE SHOULD BE OF CONVENIENT LENGTH (6 TO 10 FT). NOZZLE LENGTHS SHOWN HAVE WORKED WELL. OTHER LENGTHS MAY BE SUITED BETTER FOR OTHER CONDITIONS. THE NOZZLE MAY BE FITTED WITH ELBOW FOR BUTTERING PERIMETER OF POURED PATCH
DRY SAND BLAST	¾"	¾" OR 1"x 24"	¾"	1¼"x 24"	
WET SAND BLAST	¾" OR 1"	¾" OR 1"x 24"	¾"	1¼"x 24"	
PLASTIC MORT. GUN (MIX 1:1.5 PLASTIC)	¾"	¾"x 10"	¾"	1¼"x 10"	
DRY MORTAR GUN (MIX 1:4.5 DAMP)	1"	1"x 10"	2"	1½"x 10"	
VACUUM CLEANER	1½"	1¼" HOSE OR TO 1½" WASTE	2"	1½" HOSE OR TO 2" WASTE	

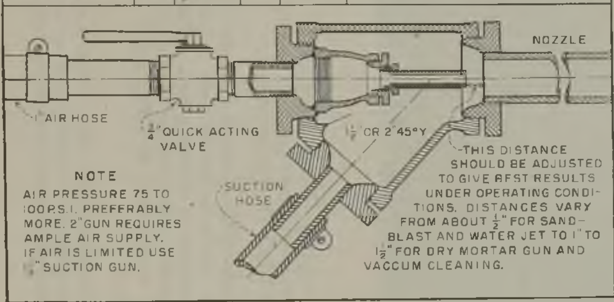

Fig. 27—Air - suction gun for dry sand blasting, washing, mortar application, and vacuum cleaning.

into the joint with various suitable implements. In some yards an air jet was used to spread this mortar. At Savannah an excellent job of coating the construction joint of the last hull built was obtained by shooting the plastic mortar on the joint with the air suction gun shown in Fig. 27.

Imperfections have been due mainly to failure to remove loose material on the joint or the failure in proper vibration of the new concrete immediately above the joint, both human failures eliminated by better supervision and inspection of these operations.

CURING

Water curing, unlimited except for the vagaries of construction and when it conflicted with essential operations, was standard practice. Except for deck surfaces all early curing was from inside the hulls as in Fig. 28, as outside forms were not removed until sometime after the deck concrete was placed.

Because most published* and unpublished tests of concrete made with light weight aggregate indicated great benefit from prolonged water curing, especially in modulus of rupture and tensile strength, it was decided early in the program to extend water curing to a minimum of 30 days and, wherever practicable, maintain it until the hulls were either launched or painted.

*"The Effect of Curing Conditions of the Strength of Burnt Clay Aggregates", by W. F. Kellermann, *Public Roads*, May, 1937

Fig. 28 (top left)—Water curing of hull interiors commenced as soon as forms were stripped. Structural details of interior National City oil barge are shown.

Fig. 29 (top right)—Canvas irrigation hose arranged along gunwhale commencing to wet outside of hull. Water quickly spreads over entire area. This was a particularly advantageous and reliable means of water curing when arranged along tops of walls, hatch coamings, and other high points.

Fig. 30 (bottom left)—Dry cargo hulls above water line were hose tested. Appearance of moisture on opposite face necessitated repair operations and retest.

Emphasis was placed on prompt commencement of curing as soon as surfaces were sufficiently hardened or were exposed by form stripping. Early stripping was urged and done where practicable as forms in place are a poor substitute for early water curing. The greatest delay came in stripping under deck surfaces because forming for another lift was not contingent on its prompt removal. Unfortunately some under deck forms were designed so that slab and sides of beams could not be stripped without removing supports from beams and this required delay. Others were designed for removal of all except the beam soffits and this permitted an earlier start of curing.

Men, and in one yard, women, with hoses were the principal means of wetting the concrete, especially the bottom lift and under the deck. Cotton "soil-soaker" hose was used to good advantage along the tops of the walls after the second major lift before deck construction progressed to the point where the hose could not be left any longer on the construction joint. Arranged around the hatches and along the edge of the deck as in Fig. 29, this type of hose was the best means used to keep deck and outside shell surfaces wet. Cotton mats were used in the southern yards for early deck curing and were excellent as long as they were kept in place. Perforated pipe was also used in some locations.

Sealing compounds were often advocated as a substitute for water curing but were not approved because (1) they were not a suitable primer for the paints and coatings to be used, (2) they might interfere with revelation of a leak under the hydrostatic tests, and (3) for the thin slab, structural concrete in hulls they would provide a curing medium too inferior to water curing to be considered favorably, particularly in view of the special need for thorough water curing of light-weight aggregate concrete.

TESTING AND REPAIR OF IMPERFECTIONS

In maritime construction, it is necessary for ships to receive a satisfactory classification by a classifying agency in order to secure favorable insurance rates and proper crew. The classifying agency designated in the Merchant Marine Act of 1920 has rules and standards based on experience with steel ships, which were the basis of its requirements in connection with design, construction and testing operations of the concrete ships. Extreme testing requirements caused a great deal of very expensive delay in deliveries. Unfortunately, there were not established, before commencing the program, classification rules which recognized any essential differences between steel plate and a thin slab of reinforced concrete.

The steel ship rules require that hull shells, liquid cargo and ballast tank bulkheads, decks, chain lockers and fuel and water tanks be absolutely watertight under hydrostatic pressures of as much as 8 ft. above the deck for periods of at least four hours. Dry-cargo hulls must be tested by examination for leakage inside with the hull immersed 18 to 20 ft. and by hose test above that water line (Fig. 30). In the hose test a fire hose with a pressure at a nozzle of 30 psi is played for an hour on each 100 sq. ft. of hull surface from a distance of 10 feet and no moisture may show on the opposite side for the hull to be acceptable.

Fig. 31—Typical minor leaks resulting from high hydrostatic test pressures with hull tanks filled with water under head 8 ft. above the deck. Even leaks like small damp spot at left of the larger spot were required by the classifying agency to be repaired, supposedly through the shell.

The classification rules make no distinction as to what is a leak requiring repair and what is not. Pressure was held on each tank a minimum of 4 hours and any appearance of water was marked as a leak and repair was required by the classifying agency even though it was only a small damp spot as in Fig. 31. It was believed by many that the classification rules placed an unnecessary handicap on construction of concrete vessels because of the requirement that such leaks be repaired through and through, necessitating as much work to make them tight and dry as leaks obviously requiring repair. Insignificant and inconsequential leaks in concrete would never get larger but would probably get smaller and seal off altogether. On the other hand, in steel, for which the rules were written, such leaks often could be stopped with one or two blows of a caulking tool and should be stopped, because in steel, experience shows they would never get smaller but probably get larger due to working of the hull. Thousands of man hours were spent and many days of hull service were lost working on scores of small damp spots which, all together, as one workman said "Would not make a gallon of water all the way to Australia, and most of them would be dried up before it got there." Bilge pumps serve only as standby equipment on a concrete ship and are never used to pump water leaking into the hulls except as a result of accident. The hulls are dry and dusty inside. Sweating is almost entirely absent, in marked contrast to reported sweating in steel hulls under some conditions sufficient to start the bilge pumps.

As the program progressed and workmanship became better in placing the concrete, testing difficulties decreased. It became more generally recognized that well-vibrated concrete of medium-low slump was more nearly watertight than wet-placed concrete, probably because reduced bleeding and less stratification resulted in a continuity of solids that meant positive watertightness even in the 4½-in. walls under a 40-foot hydrostatic test head. Furthermore, after a seemingly endless exhibition of individualism in all the tanker yards relative to repair methods standardized practice for repairs was established which was based on methods that were sound in principle and well proved in practice on the job.

A large item in repair procedure was the "poured patch" for reconstruction of areas either unfilled originally or so poorly consolidated as to require replacement to secure watertightness. Many a spot showing slight permeability was adequate and sound structurally but, under the severe hydrostatic test and watertightness standards, it proved cheaper and quicker to remove and replace such areas as soon as they were located than attempt to seal them by grouting methods, often unsuccessful and always very time consuming.

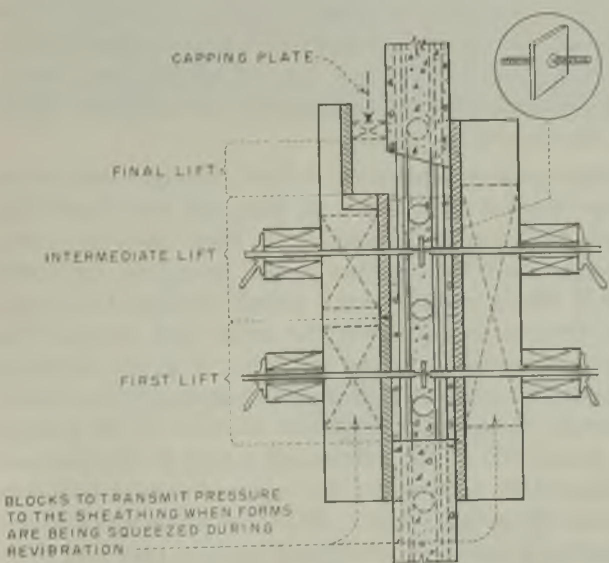

Fig. 32—Forms and fastenings for concrete replacement.

Fig. 32 shows a squeeze-type form which invariably produced tight replacements when established procedure was carefully followed. Briefly, the hole was trimmed out with air-driven chipping tools making sure that all concrete that would leak was removed, that there were no sharp corners but that edges were squarely cut except for the slope to the access chimney at the top, that steel around the perimeter of the hole was ½ in. clear of the concrete, that the contact surface of the concrete was kept wet at least overnight and sandblasted (see Fig. 27, 35 and 36) and thoroughly washed just before the form and new concrete were placed. The front form for larger holes was made in one-foot lifts and the edges of the holes to be covered in each lift were buttered with soft mortar of the same mix as in the concrete, well rubbed in just before the form and concrete were put in place. Concrete was the same as the hull mix without admixture but as dry as could be vibrated solidly into the forms. It was found that 3-in. slump was about right for the first lift and that for succeeding lifts, by the time mortar and forms had been placed, concrete from the same batch, though less in slump, was still readily vibrated solidly in place. Such premixed concrete, though of very low slump by the time the top of the replacement was made, could with care be fully molded into place with vibration and had little or no settlement. As late as the concrete would still respond to vibration, the concrete in place was vibrated through the forms while simultaneously the top center form bolt and the chimney cap wedges were tightened (Fig. 33). Vibration stopped when the tightenings stopped. The complete enclosed and restraining forms of these recast areas presented an ideal opportunity to take advantage of a small amount of aluminum powder admixture to prevent settlement before setting and insure watertightness of the upper perimeter of the repair, but approval of its use was withheld by the classifying agency.

There was an understandable reluctance to cut through and recast an area containing an isolated damp spot or leak but the classifying agency required repair that would be continuous from inlet to outlet, regardless of how insignificant the channel might be. This naturally led to grouting, little of which was successful mainly because the percolation channels were too small to receive the grout and because the cement did not stay in suspension in the majority of grout mixtures used. Where the channels were not too small, grouting made them tight; where they were too small, those made tight were made so by the surface treatment (hereafter described) at the source and outlet of the channel. With water on one side of the wall a 1-in. deep 1-in. diameter hole was drilled with a star drill at the point of issue. When the water was lowered an air pressure testing bell was placed over these holes in turn and their entrance points located by bubbles in a soap solution applied to the opposite side of the wall. A 1-in. hole was similarly drilled at these spots. By means of an impact grout gun (see Fig. 34) several shots of grout were applied at each of these holes both sides of the wall and after proper cleaning the holes were solidly dry packed. Some were made dry by driving a tight wooden peg in the hole after filling it with soft cement paste. Many a time the tiny hole was plugged by the caulking action during drilling the 1-in. grout hole.

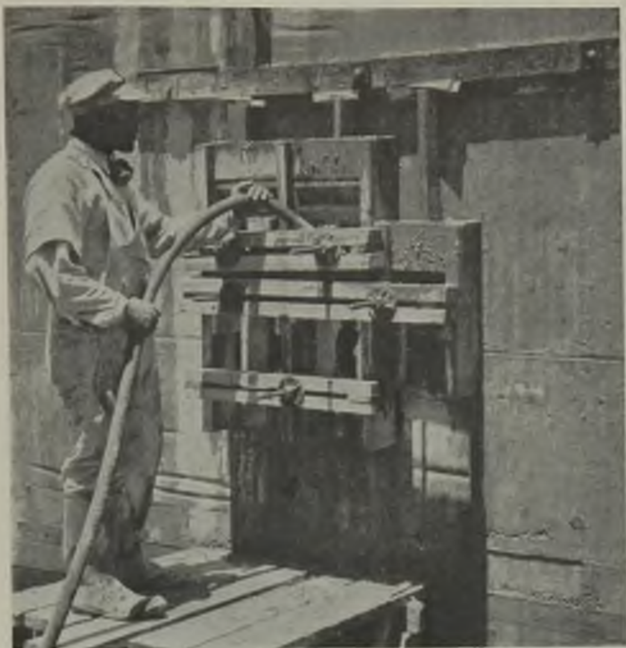

Fig. 33—After completion of concrete replacement, forms were revibrated before the concrete set and form bolts and pressure cap wedges were tightened. See Fig. 32.

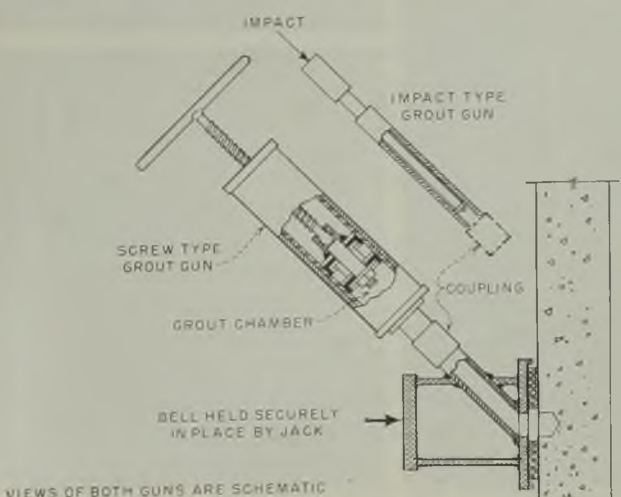

Fig. 34—Screw-type and impact-type grout guns.

Fig. 35 (top left)—The air-suction gun (Fig. 27) was used for wet or dry sand-blast (depending on condition of sand in bucket) cleaning of cutouts for repairs.

Fig. 36 (top right)—Following sandblasting, cut-out surfaces were washed with an air-water jet using the air-suction gun (Fig. 27).

Fig. 37 (bottom left)—Pre-mixed dry-pack mortar, shot in place in shallow imperfections with the air-suction gun (Fig. 27), made well-bonded sound repairs.

Fig. 38 (bottom right)—Repairs were best cured by fastening wet burlap mats against them.

Late in the program at the Savannah yard a screw type grout gun (see Fig. 34) was placed in operation and better suspension of cement in the grout was obtained by first dissolving one percent of a bentonite product in the grout mixing water. Using 2.5 to 3.0 parts of this mixing water to 1 part of cement by weight, there was a marked improvement in results from grouting operations. In most cases water from the grout appeared at the other end of the channels, giving good assurance of penetration and filling well into these small passages. Occasionally, recognizable grout would come through. Areas so treated had few damp spots on refilling the tanks. The thoroughness of this type of grouting for individual wet spots resulted in tacit approval of grouting repair work from only one side with appreciable saving in time and cost. Even this improved grouting was unsatisfactory for repair of cracks and wet perimeters of poorly constructed concrete replacements, as wet spots commonly occurred on retest between the grout holes which were only 4 to 6 in. apart.

Testing pressures severely loaded the hulls and the weight of testing water severely loaded the slightly yielding ways. Result was that deflections caused hairline cracks to appear which showed some dampness. Repair was required by the classification agency despite the fact (proved in many instances) that, with the extreme test load removed, the cracks were closed tight, sealed themselves, and showed not the slightest dampness under ordinary service-head conditions. These were repaired by cutting a deep, narrow slot along the crack on each side of the wall using a saw-tooth bit in the chipping gun and, after an overnight period of moistening and a thorough sand blasting and washing, solidly filling them with dry packed material slightly wetter than usual but placed in half-inch layers with an interval between to avoid a "rubbery" condition that resulted in a loosened bond.

There were various shallow areas requiring repair where no leak or damp spot was involved. After proper trimming, wetting and cleaning, these were replaced either with premixed mortar, layer by layer, with a stiffening period between each layer, or with the mortar guns (see Fig. 27) which applied with excellent bond and density a ready mixed 1:4.5 natural or light weight sand mortar of a very dry consistency but little wetter than for dry pack. See Fig. 35, 36, 37, 38.

In the latter part of the program this mortar gun replaced trowelled repairs and dry pack including work on damp deflection cracks and damp spots. For replacement with the mortar gun it was necessary to flare the cuts to avoid inclusion of rebound. To keep rebound minimum and avoid an over-rich replacement it was necessary to avoid too dry a replacement. Plugging of the gun prevented the material from being placed too wet. Two-layer work and careful trimming later, before it set, prevented damage to bond. Finishing by rubbing with a rag without adding water was advocated in preference to wetting and trowelling.

Gunite was tried prior to adoption of the mortar gun and found unsatisfactory for this work mainly because it was too large a facility for the purpose. Neat work could not be done in the small areas involved; extensive, expensive cleanup of surrounding surfaces was required later because the cleanup was seldom promptly done; attempts made to use gunite for through holes and to stop leaks resulted in a low percentage of successful repairs.

PAINTING

All concrete hulls were required to be painted above the light load waterline, with the standard "low-visibility gray" color selected by the Maritime Commission for war-time operation. Concrete is somewhat similar to this in color, but not sufficiently close to satisfy the requirements. It was decided that hull bottoms need not be painted, as the concrete is resistant to seawater, and the steel was well covered. No anti-fouling paint was required as information at hand indicated somewhat less barnacle formation on concrete than on steel, and the long period of service before repainting would be possible, would make the original application of little benefit. Some contractors elected to paint the bottoms, however, and were permitted to do so.

Bottom surfaces and sides below the light load line were faired by bevelling offsets 1 inch for each ⅛-in. thickness of the offset. Airholes were filled by sacking, on both painted and unpainted concrete, on the hull exteriors, but unpainted concrete surfaces on interiors, such as dry cargo holds, ballast tanks, and storage flats, received no surface treatment after form removal and necessary repairs had been made.

Interiors of holds designed for oil cargo were coated, either with vinylite resin paint (Fig. 39) or with an emulsion of Thiokol latex reinforced with Osnaberg fabric (Fig. 40). Where gasoline was carried this was necessary to protect the gasoline from contact with the concrete, since the alkaline nature of the concrete causes a reaction which breaks down the gum-inhibitors in the gasoline.

Clean, formed surfaces required little or no preparation for application of the Thiokol. Only a clean, dust-free, hard concrete surface is required. Offsets must be roughly bevelled, but no sacking or stoning is required, even for pinholes. No acid etching is required. Where laitance had formed, or cement washes had been used, sandblast proved the most efficient means of removal. The Thiokol latex coating seemed the only surface treatment of the concrete which was sufficiently elastic to bridge and stop leakage of any slight deformation cracking.

Fig. 39 (left)—Vinylite resin paint (black) was used on interior of cargo tanks in the National City oil barges.

Fig. 40 (above)—Thiokol latex (white) reinforced with Osnaberg fabric was used on interior of cargo tanks in the Houston and Savannah oil barges.

Other paints, both for exterior and interior, required surfaces which had been made smooth by a flexible-disc, rotary, electric sander, and then sacked to fill air bubbles and irregularities. It was necessary to etch the surfaces with a 20 percent or stronger solution of muriatic acid. Where the acid was applied by spray instead of brush, a 50 percent solution was found necessary. After brooming or agitating, the acid wash was removed with water so that there would be no powdery bloom when the surfaces dried. For all coatings except the Thiokol the concrete surface had to be dry.

Paints of the vinylite resin base type have given the best service to date of the available materials, and were used for both exterior and interior surfaces, including gasoline cargo holds. The Thiokol latex coating has apparently performed well also.

The writer is indebted to A. D. Kahn, Senior Materials Engineer, for assistance in reviewing and editing this paper, and to Pell Kangas, Associate Materials Engineer, for preparing the drawings and assembling the tables. Messrs. Kahn and Kangas were employed in the Concrete Control Section of the U. S. Maritime Commission during the construction of the concrete vessels.

Discussion of this paper should reach the ACI Secretary in triplicate by April 1, 1945 for publication in the JOURNAL for June 1945.

A part of PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE Vol. 41

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

Vol. 16 No. 3 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN January 1945

Fully and Partly Prestressed Reinforced Concrete*

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Member American Concrete Institute

SYNOPSIS

In this paper it is intended to show the distinction between fully and effectively partly pre-stressed concrete, employing high strength steel. The various systems and methods are described, "pre-stretching" and "post-stretching" being distinguished. The losses of the initial pre-stress are discussed and formula are derived for the factor of safety against cracking and for the minimum stretching force to ensure "full" prestressing, to be reduced for "partial" prestressing to such extent that dangerous cracks are avoided. Published comparative test results of prestressed and conventionally reinforced beams are discussed and some new data are presented regarding preliminary comparative tests on beams reinforced with high tensile wire. It is shown that a partly prestressed beam, having a reinforcement of about one fifth of that required for mild steel, behaves similarly to a conventionally reinforced beam, when under working load, and similarly to a fully prestressed beam (i.e. remaining crackless), when the load is removed.

1. INTRODUCTION

In the design of reinforced concrete it is a standard rule to base the design on a "cracked" section independent of whether cracks occur in the construction or not. The width of cracks varies with the percentage of reinforcement and the working stress of the steel, depending on the bond effected. In order to avoid the danger of corrosion to the reinforcement, which can be done if the width of cracks is limited to, say, 0.01 in.[(1),(2)]‡, the working stresses are limited to 25,000 to 34,000 psi.

Prestressing, generally, denotes that the reinforcement is tensioned *before* the *load* is applied, the stretching force being transmitted as compression to the concrete after the latter has attained sufficient strength to take up the stresses occurring at this stage. By this process stresses are imparted to the structure of opposite sign to those occurring under load.

*Received by the Institute April 17, 1944.
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‡See references at end of paper.

Pre-stretching and *post-stretching* indicate whether the tensioning is carried out *before* or *after hardening* of the concrete. With pre-stretching the products have to remain in the molds until the stretching, produced by tensioning the reinforcement against anchorages at its ends, can safely be transmitted to the concrete, which is done mainly by bond. There is one exception only, when prestressed reinforcing units with self-contained compression members according to H. Schorer(3),(4) are used. In this case the molds can be removed before the concrete has hardened, the connections and compression members of the reinforcement being withdrawn after the concrete has attained the required strength. The self-contained compression is thus transmitted to the concrete by virtue of the bond.

Post-stretching is carried out against the hardened body of the concrete. The molds can be removed soon after production but special provisions at the ends of the reinforcement are necessary for the transmission of the compression to the concrete, there being no bond between the reinforcement and the concrete. There is an exception when the steel is electrically prestressed due to the increase of temperature* according to K. P. Billner(5), in which case the bond is restored afterwards.

With pre-stretching at the release of the stretching force on the concrete, the initial prestress is immediately reduced owing to the elastic deformation of the concrete and to shrinkage, which losses gradually increase later by further shrinkage and plastic flow of the concrete. With post-stretching no immediate losses, owing to the elastic deformation of the concrete and to the first part of shrinkage, occur.

2. THE VARIOUS SYSTEMS OF PRESTRESSING

Table 1 contains a summary, distinguishing various systems on the basis of their first publication either in articles or in letters patent and referring to the progressively different purposes for which the systems were evolved.

Already at the very beginning of reinforced concrete, prestressing was used to strengthen the structure by tightening the reinforcement but without considering the degree of prestress. In his 50 year-old Patent "Constructions of Artificial Stone and Concrete Pavements", P. H. Jackson, of San Francisco, (1)† described many methods for performing the stretching of ties, provided in the footings (impost beams) of arches along their length by skewbacks, turnbuckles, screws and nuts, wedges etc. In the year 1888 another patent was applied for, which might be related to prestressing.‡ Short members of triangular section for use as

*The method of tensioning by heat has already been suggested by the Czechoslovakian Boh. Ruml (U. S. Patent No. 2,061,105).

†The numbers in brackets relate to the single cases in Table 1.

‡German Patent No. 53,548 granted 1890 to C.F.W. Doehring.

fireproof protection of timber floors are manufactured from mortar and tensioned wires.* The method suggested for stretching a number of wires simultaneously appears rather interesting.

The idea of counteracting the straining due to loading by prestressing was for the first time clearly expressed by the Austrian J. Mandl (2*a*), who wanted as early as 1896 to utilize the strength of the concrete as much as possible by reducing the concrete tensile stresses under load. A formula for the magnitude of the stretching force in accordance with this proposal was derived by the German M. Koenen (2*b*) in 1907 to the end that the concrete tensile stress in a triangular straight line stress distribution, obtained for working load, should be limited to a certain magnitude. The Norwegian J. G. F. Lund (2*c*) suggested the production of straight vaults, consisting of rows of prefabricated "brittle" blocks, jointed in mortar, with prestressed tie rods arranged between the rows in a wide mortar joint. The compression is transmitted to the blocks by washer plates at the ends, and the bond is destroyed at stretching. Similarly the American G. R. Steiner (2*d*) proposed to tighten the reinforcing rods first against the green concrete, thus destroying the bond, and to increase the tension after hardening of the concrete. The last two proposals thus represent the first steps towards effective "post-stretching." However, in all these cases the losses owing to shrinkage and plastic flow were not considered; the stretching force was therefore too small and tests did not, in consequence, give satisfactory results. It can safely be assumed that the initial prestress in all these propositions was less than 18,000 psi. in order to meet the existing regulations regarding permissible steel stresses.

A further development of prestressing led to the demand for guaranteed absence of cracks, which can be ensured reliably by "*full*" prestressing; in this case the stretching force has to be of such a magnitude that in the sections of the structure, when under working load, even after all possible shrinkage and plastic flow has taken place, concrete tensile stresses are eliminated. R. H. Dill, of Alexandria Nebr., (3*a*), appears to have been the first, in 1923-25, to make such a proposal to be carried out by "post-stretching." In this system a bonding between concrete and reinforcement is prevented by coating the latter with a yielding substance, and the stretching process is carried out after a great part of the shrinkage has taken place, thus largely avoiding the losses of the prestresses, which rendered the proposals described above, ineffective. Dill's idea was a process "that recognizes the difference in qualities of concrete and steel and combines the two materials in a scientific manner",

*The explanation given by Doehring does not, however, appear to meet the point by stating that if an extensible material such as wire under a primar tension is combined with a second, inextensible material, such as mortar, both materials, when under tension due to load, will be strained together nearly to the same extent up to their ultimate strength, the breaking of both materials occurring simultaneously; nothing is mentioned of tightening, counter-action or pre-compression.

TABLE 1.—THE VARIOUS SYSTEMS OF PRESTRESSING FOR STRUCTURES, STRAINED BY BENDING.

No.	Purpose of Prestressing	Process Suggested		Name of Proposer	First Publication		Pre-	Post-	Way of Achievement of the Purpose
					Year	(Place—time of issue)	Stretching		
1	Strengthening the structure	Tightening to a not determinate degree.		P. H. Jackson San Francisco	1888	U. S. Pat. No. 375,999 (1888)	yes	no	Many methods for stretching the reinforcement.
2	Reduction of the concrete tensile stress to a given limit and reduction of cracking	Counteraction due to stretching the tensile reinforcement, being not or only partly effective since initial prestress too small and losses not or only partly considered.	a	J. Mandl Vienna	1896	Journal Austr, Ass. Eng. & Arch. (Z.d.Oe.I.&A.V.)	yes	no	Stretching before concreting.
			b	M. Koenen Berlin	1907	Central-Journal Germ. Bldg. Ass. (Zentral Bl. d. D. Bau Verw.)			Stretching by hydraulic jacks before concreting.
			c	J. G. E. Lund Bjorn	1907	U.S. Pat. No. 1,020,578 (1912)	no	yes	Rods having threads tightened by nuts between prefabr. blocks.
			d	C. R. Steiner Gridley-Cel., Cal.	1908	U.S. Pat. No. 903,909 (1908)			Rods having threads tightened by nuts against green concr. and afterwards stretched again.
3	Guaranteed cracklessness.	Counteraction by full pre-stressing, the stretching force being of such magnitude that no tensile stress occurs when under working load, considering the greatest possible losses of the prestress.	a	Rich, H. Dill Alexandria Nebr.	1923 1925	U.S. Pat. No. 1,684,663 (1928)	no	yes	Destruction of bond by coating.
			b	W. H. Hewett Minneapolis Min.	1927	U.S. Pat. No. 1,818,254 (1931)			Similar to Dill's proposition.
			c	E. Freyssinet Neuilly-sur Seine, France	1928	U.S. Pat. No. 2,080,074 (1937)	yes	no	High strength steel or wire stretched, reinforcement substantially reduced.

			d	Thom. E. Nichols Hornell, N. Y.	1931	U.S. Pat. No. 2,035,977 (1936)			Tensile reinforcement in excess of usual requirements.
			e	F. O. Anderegg, Newark, Ohio	1934	U.S. Pat. No. 2,075,633 (1937)	no	yes	Tensioned tie rods extending through perforated ceramic blocks.
4	Extended applicability (Increase of span).	Tensioned ties in combination with normal reinforced concrete.	a	F. Dischinger Berlin	1934	Brit. Pat. No. 464,361 (1937)	no	yes	Ties, hanging in curved lines, engage externally the reinforced concrete elements.
			b	U. Finsterwalder, Berlin	1936	U.S. Pat. No. 2,155,121 (1939)			
5	Reduction of cracking (similar to 2 but effective).	Partial counteraction by combination of an effectively stretched and an unstretched tensile reinforcement.		F. Emperger Vienna	1939	U.S. Pat. No. 2,255,022 (1941)	yes	no	Unstretched main reinforcement in usual manner and bonded additional prestressed rods of superior strength.
6	Saving Steel	a Partial prestressing.		P. W. Abeles	1940	Brit. Pat. No. 541,835 (1941)	yes	yes	The tensile reinforcement substantially reduced (thus differing from 5) by the use of high strength steel or wire also for the unstretched reinforcement.
		b Partial counteraction by partial pre-stressing high strength steel.		London	1942	Brit. Pat. No. 554,693 (1943)			As before but the whole reinforcement prestressed to the same or to different extent.

permiting a "crackless concrete" which is "free from tension". A further advantage exists according to Dill "in utilizing hard steel or steel of high elastic limit and high ultimate strength" instead of mild steel. As far as the method is concerned "nuts and rods" are suggested as the "cheapest way of stretching." Threading the hard steel would present some difficulty; but Dill does not limit his methods to nuts and refers to any other possibility of stretching.(6) W. H. Hewett, of Minneapolis Min., (3*b*) made suggestions similar to those of Dill, after he had successfully applied his idea to circular tanks in 1922(7), in which latter case the structure was not strained by bending but by tension only.

Independent of these American ideas, the French engineer, E. Freyssinet (3*c*) put forward in 1928 his scheme of creating from concrete a new homogeneous material using high strength steel or wire bonded to the concrete. This allowed such a high prestress that after the losses mentioned the residual stretching force was great enough to exert permanent compressive stresses in the concrete sections of the structure when under working load; moreover, a great saving of steel was achieved.* This idea was further developed in Germany by Hoyer, who introduced the use of super high strength wire (piano or string wire) and suggested the prefabrication of articles in a long continuous run, later cutting them into pieces of the required length, which is only possible with a homogeneous material.(8)

Another proposal for full prestressing is that of the American T. E. Nichols, of Hornell, N. Y., (3*d*) who suggested a bonded reinforcement "substantially, in excess of that warranted by prior practice" (i.e. mild steel). This proposition, however, would not appear to be very suitable (since a substantially greater reinforcement is required than usual), save for the purpose of ensuring absence of cracks, which could be attained by the previously mentioned systems with a great saving of steel.

The quick assembly of high strength ceramic blocks to form a fully prestressed structural unit was suggested by the American, F. O. Anderegg, of Newark, Ohio, (3*e*). He used tie rods, preferably of high strength steel, extending through perforations of the blocks. The interspaces between the prestressed tie rods and blocks are filled with a rather liquid mortar, no safe bond being ensured between the post-stretched tie rods and the blocks.

The idea of post-stretching ties in arched bridges with a suspended carriage originates from the German, F. Dishinger (1934); he (4*a*) and U. Finsterwalder (4*b*) (who had also suggested post-stretching the tension members in lattice girders) applied this idea to beams in order to extend the applicability of reinforced concrete structures to long

*Another suggestion of E. Freyssinet relates to a method of post-stretching by using a jack for tensioning and wedging (U. S. Pat. No. 2,270,240).

girders and bridges, where normal constructions are not so suitable. In this case curved ties engage externally conventionally reinforced concrete elements which may be divided into several sections at the joints. The tensioning is performed either by the dead weight only of the whole structure, without ensuring cracklessness, or by stretching, which may be carried out to such an extent that total absence of cracks is guaranteed.

Another aim is to increase the strength and to reduce the formation of cracks to a desired extent by adding to a conventional main reinforcement, carrying in the main dead and live load, an additional prestressed bonded reinforcement of higher quality steel, the prestress, however, although effective, being moderate. The suggestion of the Austrian engineer, F. V. Emperger (5) is a development of aim (2) of Mandl with the special purpose of permitting the permissible stresses for conventional reinforcement to be increased in view of the increased resistance against cracking, without using high strength steel as the main reinforcement.

Saving steel is the main aim of the writer's proposals. In one of them (6*a*), similar to the suggestion of Emperger, a combination of prestressed and unstretched reinforcement is employed, but the whole reinforcement is substantially reduced by using also for the unstretched reinforcement high strength steel, preferably wire. Both pre-stretching and post-stretching are applicable. In the other suggestion (6*b*) the whole of the high strength reinforcement is tensioned to the same or a different extent, either the whole or a part of the reinforcement being prestretched.

3. FULL AND PARTIAL PRESTRESSING

Whereas systems 1 and 2 in Table 1, constituting ineffective methods of prestressing, are mainly of historical interest only, system 3 relates to full prestressing and systems 5 and 6 to partial prestressing, system 4 allowing of each of these processes. The use of high strength steel reinforcement is suggested in systems 3a, 3c, 3e and 6, with the possibility of a substantial saving of steel. With system 3 of "full" prestressing the conventional methods of calculation can be dispensed with, since the sections remain under permanent compression. No objections can therefore be made against the use of high strength steel tensioned to a multiple of the usual permissible stresses. However, to fulfil the condition of full prestressing a great stretching force is required and the whole required tensile reinforcement has to be stretched. Moreover, the maximum straining of the structure at the release of the precompression on the concrete is greater than that under working load, which necessitates a great compressive strength at the early stage and is inconvenient for the transport and handling of precast products, since any additional straining has to be avoided.

By contrast, in effective partial prestressing a considerably smaller stretching force is applied with resulting economy, when only a part of the reinforcement is stretched. The reduction of the stretching force itself may not effect a saving in cost, if the stretching process is carried out by a machine, but the reduction of the number of rods to be stretched is of importance since by a simultaneous tensioning each individual rod has to be gripped at its ends and an equal straining in each rod has to be ensured before stretching the rods. Due to the reduction of the stretching force the initial maximum straining is greatly reduced as against full prestressing; however, cracklessness is not guaranteed. In spite of the substantial reduction of the reinforcement, which may be decreased to one-fifth or even less, according to the aim of partial prestressing, a straining is obtained under working load similar to that in a conventionally reinforced structure. The theoretical concrete tensile-bending stresses in a straight stress distribution serve as a basis of comparison. In a cracked section under working load there appear theoretical steel stresses which are a multiple of the usual permissible stresses. However, tests have proved that dangerous cracks which would occur if the whole reinforcement were unstretched, are avoided, as long as the reinforcement or at least a considerable part of it, is bonded to the concrete. The high steel stresses are developed only, where the bond is destroyed, in a short length in the neighborhood of each crack, especially when high strength concrete and thin wires, ensuring a good bond, are used.

A partly prestressed structure represents in its behavior an intermediate case between a fully prestressed homogeneous structure with very small deformations under working load and an unstretched structure in which great deformations and heavy cracking are developed, when high strength steel is used. The design has, of course, to ensure the same factor of safety against breaking both for fully and partly prestressed concrete, as discussed in Chapter 4. A comparison of a fully and partly prestressed design is shown in Table 3 and described more in detail in Chapter 9.

4. BEHAVIOR OF PRESTRESSED REINFORCED CONCRETE

In reinforced concrete, at cracking, a transformation of the behavior takes place, as can be seen from the break in the deformation diagram, there being in principle no difference between conventionally designed and prestressed reinforced concrete. In a cracked section of a prestressed structure, the pre-compression which is produced mainly in the normal tensile zone is gradually reduced or totally interrupted. As long as the cracks do not widen too much and consequently no substantial permanent elongation of the steel remains, the cracks close at the removal of the load, and the pre-compression is transmitted between the adjacent parts. Thus the stress distribution before cracking is as with a homogeneous material of some plasticity, as concrete, the pre-compression and the counter-bending being taken into account. These counteractions gradually disappear when wide cracks occur and at failure apparently only a conventionally cracked section—without any pre-compression and counter-bending—has to be considered.

Since with the use of high strength steel the percentage of the reinforcement is greatly reduced, there is always a state of under-reinforcement in such prestressed concrete—assuming the latter is of high strength, which is the normal case—and thus only the steel primarily influences failure. "It is agreed" on the basis of long experience in prestressing, according to Freyssinet's system, "that prestressed concrete presented no advantage as regards the point of ultimate failure, no claim has been advanced in that respect."(9)

This has also been proved in tests by Hoyer(8) on four series of beams of the same cross section, reinforced with wire of a strength of 369,000 psi., but differing in initial prestress (14,220, 56,880, 113,760 and 170,640 psi).* The ultimate load was in each case nearly the same. The two beams of each series were investigated simultaneously. In one of the beams with the highest prestress the wire broke; the other beam, however, remained intact and yielded laterally apparently effecting an unequal load distribution. At failure, five of the other beams crushed in the compression zone, the sixth beam remaining intact by laterally yielding. In all beams the theoretical steel stress at failure, calculated for a cracked section in simple bending, was 330,000 psi. i.e. 89.4 per cent of the ultimate strength of the wire. In view of the relatively small difference between the theoretical steel stress and the strength, which is only a small fraction of the initial prestress, and in view of other test results available, conclusions cannot be drawn from the fact that the wire broke only in one of the beams with the highest prestress.

Reference may also be made to tests conducted by Emperger(10) relating to four series of beams, one reinforced with mild steel of a yieldpoint of 38,350 psi. and the others with twisted round bars of special shape (Torstahl) of an equivalent yieldpoint of 57,000 psi., all having an additional bonded wire reinforcement (equivalent yield point 170,400 and ultimate strength 213,000 psi.), these wires, employed in couples, being twisted and threaded with cross bars to increase the bond. In all these series the ultimate loads obtained by tests agree surprisingly well with the theoretical loads calculated for a cracked bending section, as

*See Table 3 in (8) and Tables IV and VII in the writer's contribution to (9).

TABLE 2. THE TESTS OF EMPERGER

Series		1	2	3	4
Shape of beam		Rectangle (1)	T-profile (2)	Rectangle (1)	T-profile (2)
Percentage of reinforcement %		1.07	0.248(3)	0.325	0.113(3)
Main Reinforcement	Mild steel %	95	—	—	—
	Tor steel %	—	81	64	58
Additional Wire %		5	19	36	42
Theoretical steel stress in wire at failure		f'_s	f_y	f_y	f'_s
State of wire at failure		broken	unbroken (4)		broken
Ultimate load / Theoretical maximum load		1.04	0.987	1.01-1.04	0.99

(1) $b = 30$ cm., $D = 25$ cm., $d = 24$ cm.
(2) $B = 38$ cm., $b = 16$ cm., $D = 33$ cm., $d = 31$ cm., $d_s = 11$ cm.
(3) related to $B.d$.
(4) In some of the beams series 3, first the main reinforcement and afterwards also the wires broke.

can be seen from Table 2. With series 2 and 3 the additional wire did not break as with series 1 and 4. Emperger says that with some of the beams 3 (6 beams of this series and 2 beams of each other series were tested), the main reinforcement of a relatively small ultimate elongation (Torstahl) broke. Since further details are not disclosed, it is impossible to find out why with some of these beams the main reinforcement broke, although the ultimate load agrees very well with the equivalent yield-point stress. It is also not clear, why the wire broke in series 4 and not in series 2 and 3, although in all three series the same main reinforcement (Torstahl) was used. This matter therefore requires further investigations. It may be added that in all four series the initial prestress was in accordance with Emperger's proposal "moderate low" (57,000 psi.) and low-strength concrete was employed (Cube strength 3200-3550 psi. corresponding to f'_c of 2400 to 2660 psi.).

Since the exact straining at failure in a conventionally designed structure has not yet been known—especially in under-reinforced structures with well bonded high strength steel reinforcement[11]—, a full clarification cannot be expected for prestressed concrete. In view of the above considerations and test results discussed, it can be assumed that the ultimate tensile resistance is not increased by prestressing, and the same permissible steel stresses, related to ultimate steel stress and ensuring the same factor of safety, have therefore to be taken into account for full and partial prestressing. However, this is quite correct only as far as prestressed concrete with bonded reinforcement is concerned. When there is no bond between the prestressed reinforcement and the concrete, the permanent deformation of the steel is much greater, since it occurs not only in the neighborhood of cracks but along the whole length; consequently the cracks are wider and the structure does not behave like a conventional reinforced concrete beam but like a two hinged arch having a tie[12]. It will depend on further investigations whether a prestressed concrete beam without any bond is able to carry at failure the same loads as an equivalent beam with bonded reinforcement, if high strength steel is used. Of course, all these considerations relate only to a design where failure owing to shear or slip of the reinforcement is prevented. On the whole, with full prestressing the shear resistance is greatly enhanced as described more in detail in[4] [3] and.[13]

5. A NEW METHOD OF PRESTRESSING

The new methods of Schorer[3],[4] and Billner[5] appear especially useful for full prestressing (3*b*) and partial prestressing (6*a*). The superiority of these new methods compared with methods formerly known has been stressed in Chapter (1).

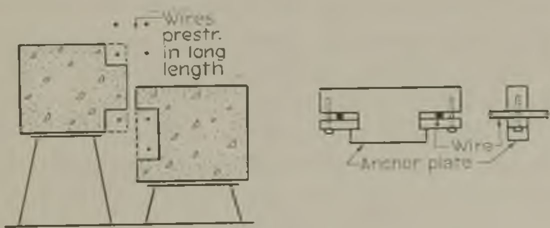


Fig. 1 and 2 (at the right)

A further new method relating to post-stretching, suggested by the writer,* may also be mentioned. The concrete products or structures cast in place have recesses provided along the tensile surface, as shown in Fig. 1. It is thus possible to manufacture them by mass production or in the usual manner in molds. After the concrete has hardened and attained sufficient strength, the stretched reinforcement is placed in these recesses and the pre-compression transmitted to the concrete. The stretching may be carried out direct against the concrete, special anchor plates being arranged at the ends which ensure the correct positioning of the reinforcement and avoid its touching the concrete in the

*Brit. Pat. No. 556,572.

recesses (friction would prevent uniform tensioning). If the reinforcement consists of thin wire, two recesses may be formed at the two edges (see Fig. 1 left part), the wire wound round the whole product, which may also consist of an assembly of single blocks), and the stretching process be carried out by twisting the wires, preferably by machine, the prestress being ascertained by the tone. For constructions cast in place the tensioning may be carried out by winding wires around a reel anchored in the hardened concrete. There is, however, another possibility of simplification in the manufacture of precast products i.e. to stretch the wire in long length against anchor blocks at its ends. The hardened products are brought into position, as shown in Fig. 1. At the ends of each product simple anchor devices may be used, having jaws to grip the tensioned wire according to Fig. 2, relating to Fig. 1 left beam.

It is possible to fill these recesses completely with concrete or cement mortar and thus to obtain an additional bond. This method allows therefore of a combination of the advantages of post-stretching and pre-stretching similar to Billner's method, as pointed out in 1. It is, however, also possible to fill the recesses with a plastic cohesive filler, thus protecting the reinforcement and allowing of a later readjustment of the tensioning, which might be of some advantage for cast in place constructions.

An idea similar to the latter was already suggested by Walter C. Parmley, of Upper Montclair, New Jersey, in 1927* for fully prestressed concrete pipes. According to this suggestion, steel hoops are placed in grooves, provided in such pipes, and are tensioned, the grooves afterwards being filled with a non-adhesive material. However, by post-stretching of the hoops it is impossible to ensure a uniform straining along the circumference in view of the friction between hoop and concrete. With circular reinforcements, however, an accuracy of tensioning is attained by use of a new wire winding machine(14), the wire being tensioned (e.g. to about 150,000 psi.) and wound around concrete shells for prestressed tanks.

6. LOSSES OF THE INITIAL PRESTRESS

The initial prestress p_i† is reduced, as mentioned in part 1, in the case of pre-stretching by the losses owing to elastic deformation Δp_e and to shrinkage Δp_s and in the case of "post-stretching" by the loss, owing to that part of shrinkage Δp_{s2}, which takes place after the pre-compression is transmitted to the hardened concrete ($\Delta p_{s2} = \Delta p_s - \Delta_{s1}$, Δp_{s1} denoting the loss, owing to that part of shrinkage, occurring before transmission).

*U. S. Pat. No. 1,781,699 (Patented 1930).
†Notation see Appendix.

In both cases also the loss, owing to plastic flow, Δp_p has to be considered. Hence the remaining prestress at working load:

Pre-stretching: $p_w = p_i - \Delta p_e - p_s - \Delta p_p$ .... (1a)

Post-stretching: $p_w = p_i - \Delta p_{s2} - \Delta p_p$ .... (1b)

On the transfer of the pre-compression to the concrete, only a part of the total loss due to shrinkage Δp_{s1} has to be taken into account in the case of pre-stretching and no loss occurs with post-stretching. Hence the remaining pre-stress p_t at the transmission of the compression to the concrete:

Pre-stretching: $p_t = p_i - \Delta p_e - \Delta p'_{s1}$ .... (2a)

Post-stretching: $p_t = p_i$ .... (2b)

When only the bottom reinforcement is prestressed the loss due to the elastic deformation of the concrete in a section according to Fig. 3 amounts to:

$$\Delta p_e = n\left(\frac{P_i}{A_o} + \frac{P_i.e^2_{so}}{I_o}\right) = \frac{n.P_i}{A_o}\left(1 + \frac{e^2_{so}}{g^2}\right) \qquad (3)$$

For a rectangular section (breadth b, total depth D, prestressed bottom reinforcement, as conventional, $A_s = p.b.d.$), equation (3) can be simplified if the influence of the top reinforcement A'_s is neglected

$$\left(g^2 = \frac{1}{12}D^2\right):$$

$$\Delta p_e = n.p.p_i.\frac{d}{D}\left[1 + 3.\left(\frac{2d}{D} - 1\right)^2\right] \qquad (3a)$$

From this equation it is seen that the loss is reduced for the same p_i with a decreasing percentage of the reinforcement p. The relative loss of prestress becomes therefore a minimum with a maximum initial prestress and a minimum tensioned reinforcement. For $n = 8$ and $\frac{D}{d} = 1.1$, the equation (3a) can be transformed to

$$\frac{\Delta p_e}{p_i} = 17.5p \qquad (3b)$$

p being expressed in per cent. For a reinforcement of 0.1 per cent the loss of the initial prestress is only 1.75 per cent and increases to 17.5 per cent for a reinforcement of 1 per cent.

When the top reinforcement is also prestressed according to Fig. 4, P'_i and p_i being the initial stretching force and prestress at the top, the two equations (4a) and (4b) are derived.

$$\Delta p_e = n.\left[\frac{P_i + P'_i}{A_o} + \frac{(P_i.e_{so} - P_i.e'_{so}).e_{so}}{I_o}\right] \qquad (4a)$$

$$\Delta p'_e = n.\left[\frac{P_i + P_i}{A_o} - \frac{(P_i.e_{so} - P'_i.e'_{so}).e'_{s}}{I_o}\right] \quad \text{(4b)}$$

The shrinkage coefficient varies between 0.00025 and 0.0005. The loss owing to plastic flow depends on the magnitude of the concrete stresses developed and on their duration. The maximum may be considreed to be not greater than the greatest loss due to shrinkage. To be on the safe side it suffices to consider the greatest possible losses only, which amount on the above simplified assumption to $0.001E_s$ i.e. to 25,000 to 30,000 psi. for a Modulus of Elasticity of 25 to 30 x 10^6 for high strength steel.

From all these considerations it is seen that for a larger reinforcement and a smaller initial prestress a much greater initial stretching force P_i has to be applied than for a smaller reinforcement and a higher initial prestress to ensure the same stretching force P_w remaining at working load conditions.

7. THE CONDITIONS FOR FULL PRESTRESSING

a. Bottom reinforcement only prestressed (Fig. 3)

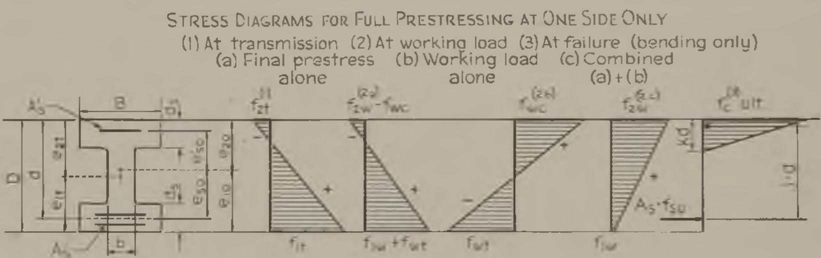


Fig. 3

The main equation for the design of a fully prestressed section can be derived from the condition that the stress f_{1w} must not be a tensile stress, the limit being $f_{1w} = 0$.

The stresses at working load are given in the following equations (5a) and (5b), M_w denoting the bending moment for working load, and the area A_o and the moments of inertia I_o and I_t computed as follows:

$$A_o = b.D. + 2.(B\text{-}b).d_s - A_s + (n\text{-}1).A'_s, \text{ and } A_t = A_o + n.A_s$$

$$I_o = \frac{1}{12}\left[B.D^3 - (B\text{-}b).(D - 2.d_s)^3\right] - A_s.e^2_{so} + (n\text{-}1).A'_s.e'^2_{so}, \text{ and}$$

$I_t = I_o + A_o.(e_{1o} - e_{1t})^2$. $A_o, I_o, e_{so}, e'_{se}, e_{1o}$ and e_{2o} relate to the net area of the concrete and to the unstretched area, whereas A_t, I_t, e_{1t} and e_{2t} relate to the total area including the prestressed reinforcement, there being the following relation: $n.A_s.e_{so} = A_t.(e_{1o} - e_{1t})$.

$$f_{1w} = \frac{P_w}{A_o} + \frac{P_w.e_{so}.e_{1o}}{I_o} - \frac{M_w.e_{1t}}{I_t} \qquad (5a)$$

$$f_{2w} = \frac{P_w}{A_o} - \frac{P_w.e_{so}.e_{2o}}{I_o} + \frac{M_w.e_{2t}}{I_t} \qquad (5b)$$

Equation (5a) can be transformed for the limit $f_{1w} = 0$ to

$$p_w = \frac{P_w}{A_s} = \frac{M_w}{\left(\dfrac{I_t}{A_o.e_{1t}} + e_{so}.\dfrac{I_t.e_{1o}}{I_o.e_{1t}}\right).A_s} = \frac{M_w}{\left(\dfrac{S_{1t}}{A_o} + e_{so}.\dfrac{S_{1t}}{S_{1o}}\right).A_s} \qquad (6)$$

hence the initial stretching force, considering the losses Δp according to equation (1a) or (1b)

$$P_i = A_s.p_i = A_s.(p_w + \Delta p) \qquad (7)$$

Equations (6) and (7) are the main conditions for the determination of the required minimum stretching force to ensure full prestressing. When the percentage of reinforcement is small, the center of gravity of the unreinforced concrete section A_c will only be slightly shifted for the sections A_o and A_t, since n is rather small (about 8) in this stage of homogeneity, e_{1o} and e_{1t} may therefore be considered to be approximately equal.

The reinforcement A_s has, of course, to ensure the desired factor of safety s against breaking, as pointed out in Chapter 4, in accordance with equation (8), relating to a cracked section of conventional type, f_{su} being the ultimate steel stress at failure.

$$A_s = \frac{M_w.s}{j.d.f_{su}} \qquad (8)$$

For high strength steel, especially wire, f_{su} may be 85 to 100 per cent of the total strength and mostly exceeds the so called proof stress (corresponding to a permanent elongation of 1 per cent).

The greatest stresses do not occur at working load but at the transmission of the pre-compression to the concrete, as is seen in Fig. 3, in accordance with the following equations (9):

$$f_{1t} = \frac{P_t}{A_o} + \frac{P_t.e_{so}.e_{1o}}{I_o} = \frac{P_t}{A_o} + \frac{P_t.e_{so}}{S_{1o}} \qquad (9a)$$

$$f_{2t} = \frac{P_t}{A_o} - \frac{P_t.e_{so}.e_{2o}}{I_o} = \frac{P_t}{A_o} - \frac{P_t.e_{so}}{S_{2o}} \qquad (9b)$$

These stresses are reduced at the center portions of beams when the dead weight counteracts the straining due to the prestress. The expression $P_t.e_{so}$ in the above equation has for this case to be replaced by $(P_t.e_{so} -$

M_o), M_o denoting the bending moment at the particular sections for which the stresses are calculated due to dead weight in action before the transmission of the pre-compression of the concrete.

b. Top and bottom reinforcement prestressed

The occurrence of tensile stresses f_{2t} (Fig. 3) can be avoided, if the top reinforcement also is prestressed, as illustrated in Fig. 4. In this case the two equations (5a) and (9b) serve together with the equations (4a) and (4b) for the determination of the two forces P and P'.
corresponding to equation (5a):

$$f_{1w} = \frac{P_w + P'_w}{A_o} + \frac{P_w.e_{so} - P'_w.e'_{so}.}{S_{1o}} - \frac{M_w.}{S_{1t}} \quad \ldots\ldots\ldots\ldots (10)$$

corresponding to equ. (9b):

$$f_{2t} = \frac{P_t + P'_t}{A_o} - \frac{P_t.e_{so} - P'_t.e'_{so}.}{I_o} \quad \ldots\ldots\ldots\ldots (11)$$

STRESS DIAGRAMS FOR FULL PRESTRESSING AT BOTH SIDES
(1) At transmission (2) At working load (3) At failure (bending only)
(a) Final prestress alone (b) Working load alone (c) Combined (a) + (b)

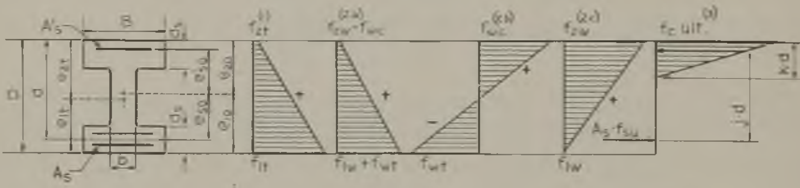


Fig. 4

These equations have to be transformed for the limits $f_{1w} = 0$ and $f_{2t} = 0$.

8. SAFETY AGAINST CRACKING

The factor of safety against first cracking* s_r is given by the equation:

$$s_r = \frac{f_{1w} + f_{wt} + f_r}{f_{wt}} \quad \ldots\ldots\ldots\ldots (12)$$

f_{wt} is the tensile stress, due to working load, in a homogeneous section with unstretched reinforcement. The expression ($f_{1w} + f_{wt}$) represents the remaining prestress at the tensile fibre after the greatest possible losses have taken place (Fig. 3 and 4). In a fully prestressed structure, f_{1w} is a compressive stress with the limit zero. The stress f_r, occurring at first cracking (rupture), corresponds to the flexural strength f'_{cf} of unreinforced concrete, which is also called "Modulus of rupture." This is only

*Under "cracking" it is understood in the following that state where the first cracks become visible.

an arbitrary value, obtained for a straight line stress distribution and incapable of measurement. But in practice, the stress distribution in the tensile zone of a reinforced concrete section before cracking, corresponding to failure in an unreinforced section, is either curved or rectangular, due to the plasticity of the material, the greatest stress being equal to the tensile strength of the concrete f'_{ct} (see [4] page 528-5 Fig. B and C). The Modulus of rupture varies greatly for different shapes and dimensions of the cross section and for different plasticities, even if the tensile strength is the same, and amounts to about 1.5 times to twice the tensile strength. The latter is, however, "not appreciably affected by either length or size" of specimens, as tests have proved.[15] The concrete tensile strength can be obtained from briquettes or, preferably, cylinders. It is known that the stress distribution in briquettes is not uniform, owing to the reduction of the cross section[16]; however, some tests, discussed in the following, have proved quite satisfactory. Several formulas are known for the relation between f'_{ct} and f'_c, mainly on the basis . . . . . . $f'_{ct} = a.f'^{\,b}_c$, e.g. $f'_{ct} = 5\sqrt{f'_c}$ according to Abrams ([4] formula (1)). If generally $f'_{ct} = \frac{f'_c}{k}$, the value k is according to new extensive investigations [17] for a (1:4.5) mix concrete on the average 8 for $f'_c = 2000$ psi. and 15 for $f'_c = 8000$ psi. Thus the following relation is obtained for intermediate values $k = 8 + \frac{f'_c - 2000}{857}$, which gives a good approximation.

When f'_{ct} is known, f_r can be replaced by 1.5 times to twice f'_{ct}. It appears to be advisable, generally, to consider only the smaller value *i. e.* 1.5 f'_{ct} in order to include all cases of plasticities and various shapes and dimensions.

When considering the factor of safety against cracking, necessary to guarantee complete absence of cracks, the effect of repeated loading has to be taken into account. According to various tests, cracking in a reinforced concrete structure of conventional design can only be avoided, if the working load is less than half the load which results in first cracking. [18] It means that the concrete tensile stress f_{rr} at repeated loading, and still avoiding cracking, must not be greater than $\frac{1}{2} f_r$ or $0.75 f'_{ct}$. With repeated loading, however, only the live load and not the working load has to be considered. The relation $f_{rr} = 0.75\, f'_{ct}$ is therefore only correct when the dead weight is negligibly small compared with the total working load. Assuming the dead weight is half the working load, the reduction of the stress f_r at first cracking is not $\frac{1}{2}$ but $\frac{1}{1.5}$, hence $f_{rr} = f'_{ct}$, which may be considered as a general relation, giving the equation:

$$s_{rr} = \frac{f_{1w} + f_{wt} + f'_{ct}}{f_{wt}} \quad \text{(12a)}$$

When the live load is ¾ or ¼ of the total working load, the equation $f_{rr} = f'_{ct}$ has to be replaced by $f_{rr} = \frac{1.5}{1.75} f'_{ct} = 0.86 f'_{ct}$ or $f_{rr} = \frac{1.5}{1.25} f'_{ct} = 1.2 f'_{ct}$ respectively. Since with prestressing the dead weight is greatly reduced, it is more likely that the live load is at least ½ of the total working load with the exception of roofs, where the dead weight is of main influence. When the live load is more than half the total working load, f'_{ct} in equation (12a) should be replaced by a smaller value, say $0.8 f'_{ct}$. (limit $0.75 f'_{ct}$).

9. PARTIAL PRESTRESSING

For the design of this kind, equation (8) remains unaltered for the reinforcement $A_s = A_{sp} + A_{su}$, if the same quality of steel is used for each part. If the properties of the prestressed and unstretched reinforcement are different, equation (8) has to be replaced by (8a), in accordance with the tests of Emperger[10], f_y denoting the yield point stress or its equivalent for the unstretched reinforcement:

$$A_{sp} + \frac{f_y}{f_{su}} A_{su} = \frac{M_w \cdot s}{j \cdot d \cdot f_{su}} \quad \text{(8a)}$$

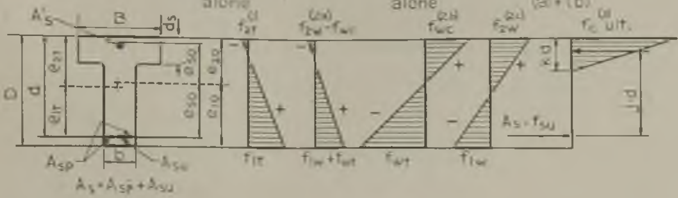
Fig. 5 shows the stress distribution for a partly pre-stressed T-section. The stresses occurring at the release of the stretching force are much below those with full prestressing and do not require a specially increased compression portion at the normal tensile zone (which is the case in an I-section).*

The resultant tensile stress f_{1w} may serve as a basis of comparison with conventional practice. It is not usual to compute this stress, since the design is carried out for a cracked section only. In order to have some idea of the magnitude of straining in conventional structures, designed for a permissible steel stress f_s = 18,000 psi., the concrete tensile stress f_{wt} at working load for a homogeneous section (n = 8) is shown in Fig. 6 (both for a rectangle $\frac{D}{d}$ = 1.1 and a thin slab of $\frac{D}{d}$ = 1.4) in relation

*In Fig. 5 a reduction of the stresses due to the pre-compression is indicated. Such a reduction would occur, if the cross section were the same as in Fig 3 and 4. However for a T-section according to Fig. 5 the compressive stress at transmission f_{1t} would, in view of the smaller concrete portion, not be reduceed as compared with the equivalent stress for full prestressing in an I-section.

STRESS DIAGRAMS FOR PARTLY PRESTRESSING **Fig. 5**

(1) At transmission (2) At working load (3) At failure (bending only)
(a) Final prestressing alone (b) Working load alone (c) Combined (a) + (b)



to the percentage of the reinforcement p and to the compressive stress f_c (in a cracked section, n = 15). The stresses f_{wt} vary between 51.5 psi. for a thin slab of 0.1 per cent reinforcement and 1080 psi. for a rectangle (designed for the limit $f_c = 1250$ psi), which latter tensile stress increases when a compressive reinforcement is provided. Even higher stresses are obtained for T-beams in which case the centre of gravity is nearer to the upper fibre and the stress f_{wt} may increase to 1500 psi. and even more for structures designed in accordance with the existing regulations.

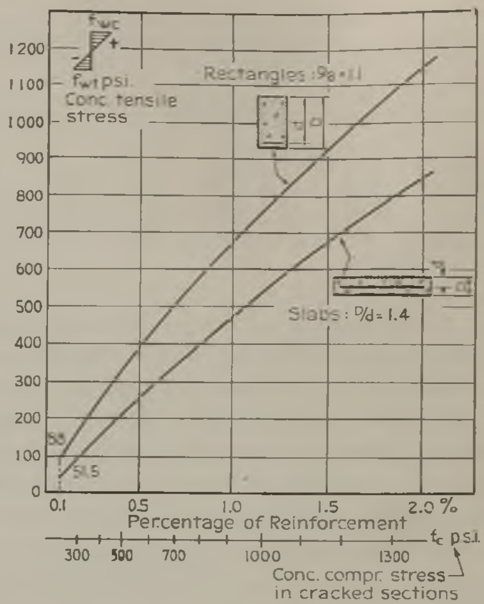
It is necessary to determine the limit of the stress f_{1w} for guaranteed absence of cracks. As can be seen from the foregoing chapter, total absence of cracks is ensured when the tensile stresses f_{1w} are considerably less than the stress f_r corresponding to first cracking. From equation (12a) it is seen that cracklessness cannot generally be guaranteed when the stress f_{1w} is greater than the concrete tensile strength f'_{ct} which limit has to be reduced to 0.75 f'_{ct} for an increased ratio of live load to total working load.

The reason why dangerous cracks do not develop in partly prestressed structures-although the theoretical steel stress in a cracked section is 100,000 psi. and more—has been discussed generally in Chapter 3. In a structure in which a part of the tensile reinforcement is unstretched, the pre-compression of the untensioned reinforcement is also of favorable influence.*

A reduction of the stretching force, derived in equation (6) for full prestressing, to about 40 per cent has proved sufficient in a special instance, as will be seen in Chapter 11. Further tests are required to prove that, generally, ¼ to ½ of the stretching force for full prestressing suffices to avoid dangerous cracks. In Table 3, three beams of the same cross section and reinforcement (Fig. 7) are compared for (a) unstretched, (b)

* Mr. Schorer states in his closure to (4), page 528-7 that "the unstretched reinforcement will in time be subjected to considerable compressive stresses which require closely spaced stirrups, lateral ties or spirals, as in columns, in order to counteract possible buckling tendencies of the longitudinal bars." This is only correct for a reinforcement in the compressive zone of a beam, where the straining is increased with increasing load. But the unstretched reinforcement in the tensile zone of a prestressed beam is pre-compressed only to a relatively small extent, say to 10,000—15,000 psi. At the release of the stretching force this stress remains only at the ends of the reinforcement and is transformed into a tensile stress in the greatest part of the beam, the maximum tensile stress, in a cracked section, being a multiple of the original compressive stress. No special provisions against buckling are therefore necessary.

Fig. 6



partly prestressed and (c) fully prestressed wires. It is interesting to note that the stress f_{1w} for the partly prestressed structure, reinforced with high strength wire, is about the same as the stress f_{wt} computed for a beam of the same carrying capacity but reinforced with mild steel.

10. DISCUSSION OF SOME PUBLISHED COMPARATIVE TESTS

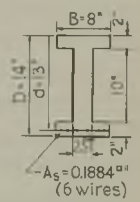
Comparative tests on prestressed beams with post-stretched reinforcement of mild steel and conventionally reinforced beams have been carried out by W. K. Hatt [19] and R. E. Mills and W. B. Miller.[20] The first relate to 4 beams 8 in. by 14 in., 13 ft. 6 in. long of 12 ft. span, reinforced with 2 rods (p = 1.04 per cent), 3 beams prestressed to different extents and one conventional beam, whereas the second tests were carried out on 3 I-shaped joists 3 in. by 8 in., 12 ft. long of 11 ft. span, reinforced with one rod of ¾ in. diam. (p = 2.2 per cent), 2 joists of Haydite concrete (one prestressed and one conventional) and one joist of segmental synthetic stone. The test results do not allow of comparison of the behavior up to the ultimate load, since the first tests relate only to stresses in the conventionally reinforced beam up to 26,000 psi., whereas in the second group of tests the joists with bonded unstretched reinforcement failed in shear (at about 2/3 of the ultimate load attained with the prestressed joist of the same material). Both groups of tests have shown that, as long as no cracks develop, the behavior of the prestressed article agrees very well with that expected according to calculation for a homogeneous

TABLE 3—COMPARISON OF 3 I-SHAPED BEAMS (SEE FIG. 7) OF THE SAME CARRYING CAPACITY

Designed for high strength steel reinforcement: f_{su} = 200,000 psi., s = $2 \cdot j.d$ = 12 in.; M_{ult} = 0.1884 × 12 × 200,000 = 452,160 lb. in. hence M_{perm} = 226,000 lb. in. corresponding to a uniformly distributed load of 377 lb. per ft. for a simply supported beam of 20 ft. span (dead weight: 57 lb. per ft.), greatest shear stress: 126 psi. (sufficient shear reinforcement provided). — Calculated with the approximate design values: $A_o = A_t = A_c$ = 57 in.^{2}, $e_{1o} = e_{2o} = e_{1t} = e_{2t} = \frac{D}{2}$ = 7 in., e_{se} = 6 in. and $S_{1o} = S_{2o} = S_{1t} = S_{2t}$ = 196 in.^{3} (No. 8).

			(a) Reinf. unstretched	(b)+ Partly prestr. (1/3)	(c) Fully prestressed
P_i for p_i = 170,000 p.s.i.		lb.	–	10,667	32,000
Δp_e according to equ. (3)		p.s.i.	–	3,733	11,200
$p_t = p_i - \Delta p_e - \Delta p_{sl}$ ($\Delta p_{sl} \sim 6300$ p.s.i.)			–	160,000	152,500
P_t		lb.		10,000	28,700
Stress distribution at transmission of the pre-compression		psi	–	f_{2t} = −130, f_{1t} = +482	f_{2t} = −376, f_{1t} = +1386
Deformation of the beams at transmission of the pre-compr. (before loading)					
$p_w = p_t - 24{,}000$ (assumed $\Delta p_{s2} + \Delta p_P \sim 30{,}000 - \Delta p_{sl}$)		p.s.i.	–	136,000	128,500
P_w		lb	–	8,540	24,200
Concrete stresses in a straight line stress distribution under working loads		p.s.i.	f_{wc} = 1155, f_{wt} = 1155	f_{2w} = +1044, f_{1w} = −743	f_{2w} = +838, f_{1w} = +13
Deformations of the beams when under working load					
Safety against cracking	s_r (equ. 12)++	f_{cf} = 750 p.s.i.	0.65	1.07	1.66
		f_{cf} = 400 p.s.i.	0.35	0.70	1.36
	s_{rr} (equ. 12a)++	f_{ct} = 500 p.s.i.	0.43	0.79	1.44
		f'_{ct} = 250 p.s.i.	0.22	0.57	1.23

Fig. 7



NOTES: +The steel consumption of the tensile reinforcement for the partly prestressed beam can be reduced by 19% as compared with the fully prestressed beam, which needs, say, 1.05 A_s (5% for overlength) as against $1.05 \frac{A_s}{3} + \frac{2.3}{3.4} A_s = 0.85\ A_s$ (assuming the unstretched reinforcement (2/3, A_s) could be reduced on the average, to ¾ of the length in accordance with the moment distribution).

*Heavy cracking must be expected considering the high tensile stress f_{wt} and the small percentage (p = 0.18%). A conventionally reinforced beam would require a mild steel reinforcement A_s = 1.04 in. (f_s = 18,000 psi.), in which case stresses f_{wc} = 1078 and f_{wt} = 873 psi. in a straight line stress distribution would be obtained i. e. similar to the stresses f_{2w} and f_{1w} of beam (b).

++It is assumed f'_{cf} = 750 and f'_{cf} = 400 to correspond to f'_{ct} = 500 and 250 psi respectively. For beam (a) the effective prestress $f_{1w} + f_{wt}$ = 0.

material. From Chart 7 in the publication [20], however, it is seen that in the Haydite joist with destroyed bond, after a crack has developed, the deflection increased to a much greater extent than in the conventional joist.

The tests of Hoyer [6] and Emperger [10], with respect to the behavior at failure, have already been discussed in Chapter (4). As to the behavior before failure they agree very well with the investigations of R. H. Evans [12], [13], discussed more closely in the following. Beams 4 in. by 10 in. of 11 ft. length (10 ft. span) were alternately reinforced with high tensile steel (one rod 1 in. diam. at the bottom in the tensile zone, p = 2.4 per cent and one rod 5/8 in. diam. at the top) and with high tensile wire (tensile reinforcement of 44 wires: $A_s = 0.221$ in.^{2} at the bottom, p = 0.68 per cent, and 13 wires: $A'_s = 0.0654$ in.^{2} at the top); 76 per cent of the stretching force was applied to the bottom reinforcement and 24 per cent to the top reinforcement in order to avoid tensile stresses at the release of the stretching force to the concrete.

Comparative tests were carried out for rod and wire reinforcement for the following initial stretching forces: 28,000, 33,600, 47,000 and 56,000 lb. as well as for unstretched reinforcement. A further test was made for a rod reinforcement with a stretching force of 56,000 lb. but destroyed bond. Fig. 8 shows the results on rod beams (including a tests with a initial stretching force of 40,000 lb.) on a chart relating to the greatest central deflection for various loadings, wherein are plotted also the theoretical values for a homogeneous section (n = 8.75) and for a cracked section. It is very striking that the deflections of the prestressed beam with destroyed bond are much greater than both those in the bonded type and the theoretical values for the conventional type, even before cracks have developed. Evans has derived formulas for the steel and concrete stresses in such beams on the assumption that the total strain in the ties must equal the total strain in the concrete surrounding it [12]. The values thus obtained agree very well with the measurements. When cracks have developed, reinforced concrete beams with destroyed bond behave as two-hinged arches, the deflection immediately after cracking greatly increasing.

As far as the difference between rod and wire reinforcements is concerned, the losses of the initial prestress were in accordance with the equation (3) much greater for the rod beams than for the wire beams, and relatively greater for smaller initial stretching forces than for greater. Cracking thus occurred much earlier in the rod beams than in the wire beams. Fig. 9 shows a comparison of the behavior of rod and wire beams similar to Fig. 8, the original charts for the respective beams being brought to the same basis of comparison by taking into account the different steel areas in the moments of inertia. From Fig. 9 it is seen that it is possible, between the two limits of beams with unstretched reinforcement and fully prestressed beams (stretching force of 56,000 lb. for wire beams), to find any convenient intermediate case where the deformation and cracking is limited to a definite extent. This can be done either by tensioning

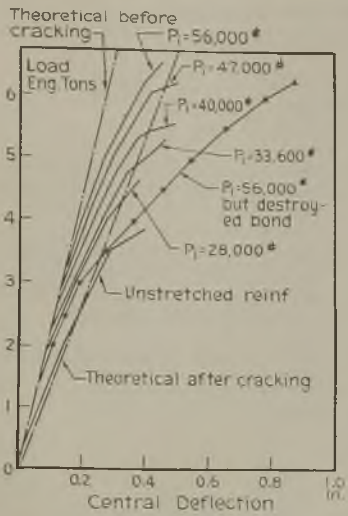


Fig. 8

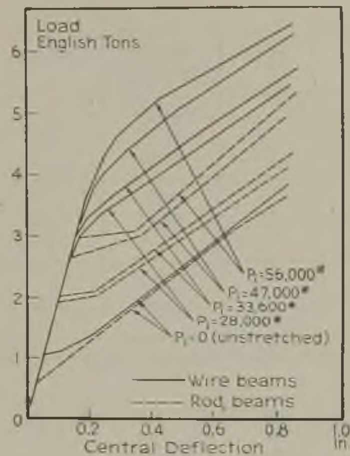


Fig. 9

the whole reinforcement to a lesser degree than necessary for full prestressing, as done in the tests by Evans, or by tensioning a part of the reinforcement only, but to a high initial prestress, a part remaining unstretched and being pre-compressed at the release of the stretching force on the concrete; the latter case is more advantageous.

11. NEW COMPARATIVE TESTS

Preliminary tests on beams reinforced with high tensile wire were made at R. H. Harry Stanger's Testing Institute, London, and conducted by the writer in collaboration with K. Hajnal-Konyi.* It was intended to compare the behavior of three different types of beams, having the same section, being of the same concrete strength, and reinforced with the same amount of high tensile wire, which is (a) not tensioned, (b) to a part (40 per cent) prestressed, the stretching force being 40 per cent of that required for full prestressing, the rest remaining unstretched, and (c) fully prestressed. According to the program two beams for each type were to be tested and two beams were to be manufactured simultaneously. The cross section of the beams and a view, showing the arrangement of loading, are shown in Fig. 10 and 11 respectively. The stress-strain diagram of the wire is shown in Fig. 12. Table 4 contains mix and strength properties of 3 series of concrete.

Unfortunately, it was not possible to use the beams of series 1 in accordance with the program, since the initial film of grease left on the

*Chartered Struct. Eng., London, M.A.C.I.

TABLE 4. STRENGTH OF CONCRETE

Series	Age days	Cube* strength psi.		Tensile strength † psi.		Notes
		Single Values	Average	Single Values	Average	
1	6	5400 5140	5270	469 448 436	451	Concrete Mix: 112 lb. Portland Cement.
2	10	6670 6240	6455	480 467 427	458	1 cu. ft. Stone Court Sand 2.5 cu. ft. Ham River shingle
3	11	5910 5800	5855	382 356 347	362	0.34 water-cement ratio

*cubes 6 in. † briquettes, section 1.5 in. by 1.5 in.

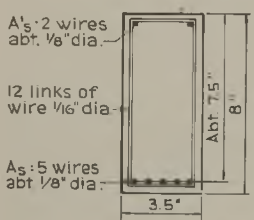
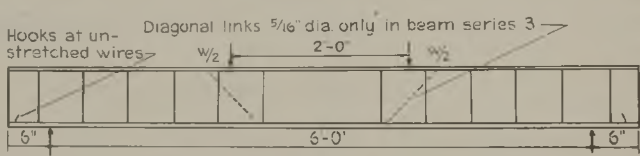


Fig. 10

Fig. 11



wire as protection against corrosion was apparently not wholly removed, by wiping with a clean rag, thus causing an early failure due to slip, owing to insufficient bond, in beams (a) and (c). To avoid bond failure the wires in series 2 and 3 were subjected to a pickling treatment, consisting of degreasing by double treatment with trichlorethylene and etching by immersion in 10 per cent nitric acid for 10 minutes. This treatment proved successful, however, it resulted in a reduction of about 0.004 in. of the diameter of the wires. In series 2 and 3 the time of hardening was extended from 6 days, as in series 1, to 10 and 11 days respectively. Owing to difficulties due to war conditions another brand of cement had to be used for concrete series 3. It was therefore not possible to comply with the main condition of comparing beams of the same concrete strength. As can be seen from Table 4, the strength properties of concrete series 2 and 3 were quite different, the tensile strength of concrete series 3 being greatly reduced as compared with series 2.

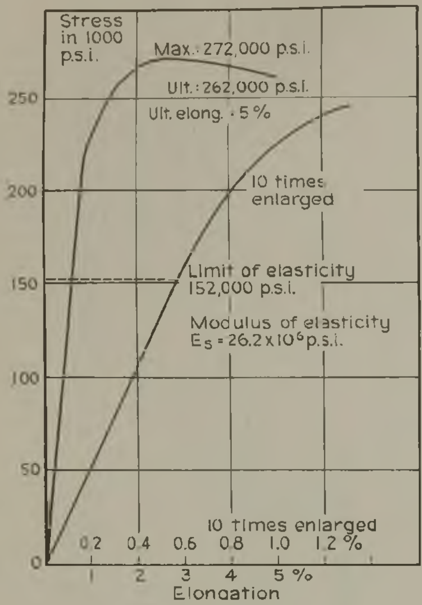


Fig. 12

Test Procedure—Method of prestressing

The wires were tensioned by means of a jack and a cross head to which the single wires were anchored, in such a manner that the stress in each individual wire could be varied by means of screw-thread. First a calibrated jack was used, giving the requisite force by a direct load reading, the single wires being adjusted so that by tuning each wire appeared equally stressed. After the elongation of the wires was recorded, the load was released and the calibrated jack replaced by a positive screw jack, the load applied being sufficient to stretch the wires to the same extent as previously recorded. The reinforcement was kept stretched up to about two hours prior to testing the beams, when the force was released and the contraction of the wires was measured. From this the loss of the initial prestress was ascertained in the case of beam (*b*) of series 3. The data regarding prestressing for beams (*b*) and (*c*) are seen in Table 6.

Method of testing

The deflection was measured at mid-span with a dial micrometer, reading in thousandths of an inch. The formation of cracks was observed and the maximum widths and depths of the cracks were measured under various loads.

Results of the tests

The dimensions of the 4 specimens and the designing values computed therewith are contained in Table 5, whereas in Table 7 there are seen the

TABLE 5—DIMENSIONS OF SPECIMENS AND DESIGN DATA

Concr. ser.	Type	b	D	d	A_s*	p	k.d.**	j.d.**	A_o	e_{so}	e_{1o}	e_{1t}	S_{1o}	S_{2o}	S_{1t}	S_{2t}
		in.			sq. in.	%	in.		sq. in.	in.			cu. in.			
2	a	3.50	8.10	7.64	0.0605	0.226	1.74	7.06	....	....	....	4.02	....	....	40.5	39.8
	c	3.47	8.10	7.60	0.0609	0.229	1.75	7.02	28.38	3.57	4.07	4.02	38.3	38.6	40.0	39.5
3	a	3.42	8.05	7.72	0.0613	0.232	1.78	7.13	....	....	....	3.99	....	....	39.2	38.5
	b	3.45	8.02	7.52	0.0603†	0.233	1.74	6.94	28.13	3.50	4.00	3.98	38.4	38.2	39.2	39.6

* A'_s = 0.0258 sq. in. for all specimens
† A_{sp} = 0.02388 sq. in. (p = 0.09%)

** $n = 15$
(homogeneous section: $n = 8$)

TABLE 6—PRESTRESS OF TYPES (B) AND (C)

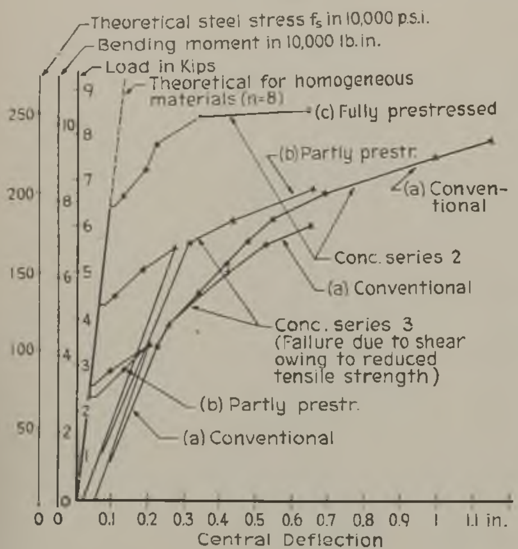
Concr. ser.	Type	P_i	p_i	P_t	p_t	Total Losses of p_i		p_e (equ. 3)	$\frac{\Delta p_e}{p_i}$	f_{1t}	f_{2t}	Notes
		lb.	psi.	lb.	psi.		%	psi.	%	psi.		
2	c	11,300	185,500	9,300	152,500	33,000*	17.8	11,650	7.1	+1180	−556	*Calculated
3	b	4,500	188,500	4,250	178,000	10,500†	5.6	4,100	2.2	+539	−239	†Measured

Note The stress Δp_{s1} = 33,000 − 11,600 = 21,350 psi. for beam 2c is too high (with beam 3b only 10,500 − 4,100 = 6,400 psi.), and may be due to insufficient anchorage, which was avoided in the case of beam series 3, type b.

TABLE 7—BENDING MOMENTS AND STRESSES

Concr. ser.	Type	First Cracking					Failure			M_{crack} / M_{fail}	Notes
		M	f_1	f_2	f_c	f_s	M	f_c	f_s		
			homogeneous		cracked section			cracked section			
		lb. in.	psi.				lb. in.	psi.		%	
2	a	32,400	−799	+814	1500	76,000	92,200	4,270	216,000	35.1	
	c	79,200	−800	+1483	3690	185,400	107,150	5,000	252,000	73.8	
3	a	28,400	−725	+738	1300	65,000	73,600	3,380	168,500	38.6	Failure due to shear
	b	53,700	−830	+1153	2565	128,200	85,700	4,100	204,800	62.6	

bending moments and respective stresses at cracking (computed both for a homogeneous section, taking into account the precompression, and for a cracked section in bending only) and the bending moment and respective stresses at failure (computed for a cracked section in bending only).



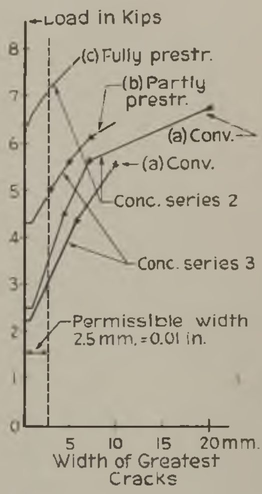


Fig. 13 (left)

Fig. 14 (above)

Although one of the main bases for a comparison was missing, owing to the difference in the concrete strength, it is possible to compare the results and draw conclusions, since the tests of Evans [12], [13] agree very well, in principle, with the results obtained in series 2 relating to beams a and c and in series 3 relating to beams a and b. Fig. 13 shows the deflections and Fig. 14 the greatest widths of cracks of the beams series 2 and 3 in relation to the loading, bending moment and theoretical steel stress in a cracked ($n = 15$). In Fig. 14 the width of 0.01 in. is also plotted, which is considered to be the limit for dangerous cracks. From the chart it is seen that with beams (*a*) this limit was reached before half the ultimate load, which proves that a wire reinforcement of such high strength cannot be used without prestressing. On the other hand, partial prestressing of 40 per cent was sufficient to delay attainment of such a width until a much higher loading was reached. It is interesting to note that in partly prestressed beams (*b*) the cracks, of a width of 0.02 in. completely disappeared after the load, which was up to 80 per cent of the ultimate load, was removed.

In both beams of series 2 the main crack at which failure occurred was at the loading points, looking as if influenced by shear. For this reason

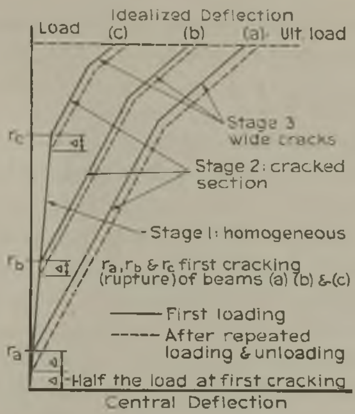


Fig. 15

diagonal closed links were added in series 3 (see Fig. 11). It was expected that this might result in obtaining an even higher ultimate load than in series 2. Since the tensile strength of concrete series 3 was unfortunately much lower than that of 2, the diagonal links prevented only shear cracks at the loading points, but such cracks developed between the latter and the supports, and both beams series 3 failed in shear. The shear stress at failure in beam (*a*) with unstretched wire reinforcement was only 128 psi., which is very low. Further tests are necessary to find the reason.

With reference to the test results shown in Fig. 9 and 13, a comparison of the typical behavior of wire reinforced beams (a), (b) and (c) in idealized form is seen in Fig. 15. Up to cracking the structure behaves as if of homogeneous material ($n = 8$), and after cracking in accordance with conventional practice (cracked section, n about 15), but when the width of the cracks increases, a third stage with greater deformations has to be considered. If sufficient shear resistance is provided for, the ultimate load for all three cases should be about the same, as discussed in Chapter (4).

12. CONCLUSIONS

1. Prestressing generally

The application of this process was of no practical or economical importance until certain improvements were introduced to ensure the effectiveness of the prestress by post-stretching (the losses of the initial prestress, which may exceed with pre-stretching the Limit of elasticity of mild steel, being considerably reduced) and by the use of high tensile steel (the total losses being only a small fraction of the initial high prestress). The ultimate tensile resistance is not increased by prestressing, but both the resistance against cracking and the resilience of the structure after cracking are greatly enhanced, and the economical employment of high strength steel is made possible.

2. Pre-stretching and post-stretching

There is a distinct difference between these two processes regarding effectiveness of the initial prestress and behavior of the structures. With post-stretching a relatively greater stretching force remains effective than with pre-stretching, thus allowing also conventional steel to be used. However, owing to the lack of bond between reinforcement and concrete, the total and permanent deformations and widths of cracks are greatly increased as compared with constructions in which at least a considerable part of the reinforcement is bonded to the concrete. Therefore when there is no bond, high strength steel can be used only if all possibility of cracking is avoided.

3. Safety against cracking

It is not the load causing the first cracks that has to be considered, but that causing cracking after repeated loading. Total absence of cracks can generally only be attained if the resultant tensile stress at working load in a straight line stress distribution is less than the concrete tensile strength to be obtained from special specimens or related to the cylinder strength. If concrete tensile stresses are excluded altogether ("full" prestressing), there is a definite safety against cracking.

4. Full prestressing

By this process a product is obtained, behaving as if of homogeneous material, hence of particular importance for special constructions such as tanks or barges, where guaranteed cracklessness is essential; an increased shear resistance is attained and steel of the highest strength can economically be used. However, a relatively great stretching force has to be applied, and the whole of the required tensile reinforcement has to be stretched; moreover, the greatest stresses at the release of the pre-compression on the concrete are greater than those under working load, necessitating a great strength at the early stage, and being inconvenient for the transport and handling of precast products, since any additional straining has to be avoided.

5. Effective partial prestressing

As with full prestressing the use of high strength steel is possible, the disadvantages mentioned above being avoided and the relative loss of the initial prestress becoming a minimum with a minimum tensioned steel and with a maximum initial prestress. Cracklessness is not guaranteed, but dangerous cracks are excluded (although the theoretical steel stresses, appearing in a cracked section, are a multiple of the working stresses now permitted) because the great elongation of the steel is limited to the immediate neighborhood of the cracks only, where the bond is destroyed, assuming at least a considerable part of the reinforcement is bonded to the concrete. The cracks close totally when the working load is reduced below the cracking load, thus allowing e.g. a construction to be designed which remains crackless under dead load, only fine cracks temporarily occurring under working load. Partly prestressed concrete presents therefore the possibility both of saving steel and of improving the properties. Tests will have to show which part of the minimum stretching force for full prestressing is sufficient to avoid dangerous cracks under working load (presumably ¼ to ½).

6. The performance of prestressing

When whole series are produced a number of wires are tensioned, preferably in long length, against anchorages at the ends. The use of thin wires as stretched and unstretched reinforcement is preferable to that of rods, since it ensures better bond. Except if prestressed reinforcing units according to Schorer [3], [4] are used, with pre-stretching the products have to remain in the mold until the concrete has attained sufficient strength to take up the pre-compression, thus requiring a large working place or a special treatment of the concrete to ensure rapid hardening. This is avoided with post-stretching where there is generally no bond; special mechanical means, however, are necessary for the transmission of the pre-compression to the concrete. There are two methods i.e. that of Billner [5] (electrical prestressing), where the bond is restored later, and the method suggested by the writer, where the stretched reinforcement is placed in recesses of the structure which can later be thoroughly and completely filled with concrete or cement mortar, thus ensuring bond in addition to the direct compression at the ends (the advantages of pre-stretching and post-stretching are thereby combined).

7. Economy of prestressing

Saving of 80 per cent of the tensile reinforcement, required in conventional practice is of some importance when steel is in demand for many other purposes. This saving in cost of material may be 60 to 70 per cent of that of mild steel, which is of great economical importance—at least in Europe. Where labor is of greater influence than the saving of material—as in the United States—the economy of prestressing depends greatly on the method used. The economical superiority of partial prestressing compared with full prestressing can be recognized, when considering the use of prestressed Schorer-units; the amount of this high quality steel is reduced to, say, ½, the other ½ being replaced by unstretched high tensile wire, which is obviously less expensive than the preparation of self-contained units.

8. Prestressed composite structures

It is possible to apply prestressing also to composite structures consisting of an assembly of single blocks, formed from any plastic substance, such as concrete, ceramic, glass, or resinous plastics, and either to thread the reinforcement through perforations or to place it in receses or grooves. In these cases post-stretching is applied and an additional bond may be ensured when the method suggested by the writer is employed.

NOTATIONS

1. General notations

A_s = area of the total tensile reinforcement.
A_{sp} = area of the prestressed tensile reinforcement.
A_{su} = area of the unstretched tensile reinforcement.
A'_s = area of the compressive reinforcement.
E_c = Modulus of elasticity of concrete.
E_s = Modulus of elasticity of reinforcing steel.
f'_c = cylinder strength of concrete, psi.
f'_{cf} = flexural strength of concrete (modulus of rupture), psi.
f'_{ct} = tensile strength of concrete, psi.
f'_s = strength of reinforcement, psi.
f_{su} = theoretical ultimate steel stress (in a cracked section), psi.
f_y = yield point stress or its equivalent, psi.
$n = \frac{E_s}{E_c}$, Modular ratio.
p = percentage of the tensile reinforcement, related to the depth d ($A_s = p.b.d.$).
p' = percentage of the compressive reinforcement.
P_i = initial prestressing force with respect to A_s or A_{sp}, *)
P_t = prestressing force at transmission, *)
P_w = effective prestressing force at working load conditions. *)
p_i = initial prestress in A_s or A_{sp}, psi. *)
p_t = prestress at transmission in A_s or A_{sp}, psi. *)
p_w = effective prestress at working load in A_s or A_{sp}, psi. *)
Δp_e = loss of the initial prestress p_i, owing to elastic deformation of the concrete, psi.
Δp_s = loss of the initial prestress p_i, owing to shrinkage, psi.
Δp_p = loss of the initial prestress p_i, owing to plastic flow, psi.
s = factor of safety against breaking.
s_r = factor of safety against first cracking (rupture).
s_{rr} = factor of safety against cracking after repeated loading.

*) P'_x and p'_x relate to the respective forces and stresses with respect to the reinforcement A'_s (x relating to i, t, w respectively).

2. Cracked section

f_c = concrete compressive stress, psi
f_s = steel stress, psi.
I = moment of inertia.
$j.d$ = lever arm.
$k.d$ = distance of the neutral axis from the compressive fibre.

3. **Straight line stress distribution for a homogeneous material**

A_o = net area of the concrete section increased by n times the areas of the unstretched reinforcements A_{su} and A'_s.
A_c = net area of the concrete section.
A_t = total area of the section, i.e. A_o plus n times the stretched reinforcement A_{sp}.
e_{1o} = distance of outside lower fibre from the centre of gravity of A_o.
e_{2o} = distance of outside upper fibre from the centre of gravity of A_o.
e_{1t} = distance of outside lower fibre from the centre of gravity of A_t.
e_{2t} = distance of outside upper fibre from the centre of gravity of A_t.
f_{1t} = concrete stress *) at the lower outside fibre at transmission of the prestress, psi.
f_{1w} = concrete stress *) at the lower outside fibre at working load, psi.
f_{2t} = concrete stress *) at the upper outside fibre at transmission of the prestress, psi.
f_{2w} = concrete stress *) at the upper outside fibre at working load, psi.
f_{wc} = concrete compressive stress due to working load alone, psi.
f_{wt} = concrete tensile stress due to working load alone, psi.
f_r = concrete tensile stress at first cracking, psi.
f_{rr} = concrete tensile stress, occurring at cracking after repeated loading, psi.
I_o = moment of inertia computed in accordance with A_o.
I_t = moment of inertia computed in accordance with A_t.
S_{1o} = sectional modulus related to fibre e_{1o}.
S_{1t} = sectional modulus related to fibre e_{1t}.
S_{2o} = sectional modulus related to fibre e_{2o}.
S_{2t} = sectional modulus related to fibre e_{2t}.

* + denotes compression, — denotes tension.

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Discussion of this paper should reach the ACI Secretary in triplicate by April 1, 1945 for publication in the JOURNAL for June 1945.

A part of PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE Vol. 41

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

Vol. 16 No. 3 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN January 1945

An Instrument and a Technic for Field Determination of the Modulus of Elasticity, and Flexural Strength, of Concrete (Pavements)*

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SYNOPSIS

An instrument for determination of the dynamic modulus of elasticity of concrete, in situ, is described. Test results are presented which show (a) the comparison of test values of E, obtained by various older methods, with that obtained with the new instrument; and (b) the relationship of such values to the flexural strength of concrete. It is concluded that adoption of the new method and technic is justified; and that widespread use of the new instrument would eliminate the necessity for casting field specimens during construction (except perhaps for day-to-day control purposes) or of removing costly "samples" from completed works. A method for determining the thickness of concrete pavements is briefly discussed. A rather extensive bibliography is included.

INTRODUCTION

The past few years have witnessed considerable advancement in nondestructive testing technics; with the so-called "sonic" (or perhaps more correctly "dynamic") methods of obtaining the modulus of elasticity of concrete, coming into increasingly widespread use. Powers [1] Hornibrook [2], and Obert [3] have employed values so obtained to reveal (a) the comparative strength and quality; (b) the resistance to frost action; and (c) the internal stress condition, of concrete. In virtually all of the work done by these investigators, however, laboratory specimens of comparatively small dimensions have been employed and E has been measured by flexural vibration methods. The authors of the present paper have, likewise, been much interested for a number of

*Submitted to the Institute August 24, 1944. This development was reported briefly by Mr. Long at the Research Session of Committee 115, at the ACI Convention Chicago, Mar. 2, 1944.
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(1) (2) (3) See Bibliography at end of paper.

years in the possibilities of non-destructive testing, and have applied dynamic technics to study of the elastic and plastic behavior of concrete. They have believed, however, that the full potentialities of such methods could never be realized so long as only laboratory specimens could be tested. Some instrument and method must be devised, which would be independent of size and shape of specimen, and would be applicable to actual concrete pavements (or other structures and materials) in the field. Specimens cast during construction, and more often than not, fabricated, cured and treated in a manner totally dissimilar to that of the actual field structure, were considered inadequate to reveal the true characteristics of the concrete pavement or structure supposedly represented.

The instrument and technic described in this paper, and the work which lead to their present development; constitute the authors' answer to the challenge presented by this situation; and it is hoped that the test results included will be helpful in clarifying the widespread speculation and misunderstanding concerning the practical value of "Dynamic *E*" determinations, in the field of engineering design and structural sufficiency evaluations.

METHODS OF OBTAINING DYNAMIC "E"

The most commonly employed method of obtaining dynamic E at the moment, is probably the flexural (resonance) vibration method used by Powers, Hornibrook, Obert and Duval, the authors, and many others. It has been amply described in the literature and will not be further expanded herein. Its limitation lies in that it is applicable only to specimens (usually prisms) of comparatively small dimensions.

Another method consists in determining the longitudinal frequency of vibration of similar specimens. In this case the limitation imposed by the method is the same.

Still another method (mentioned by Powers in a paper published in 1937, and since used by the authors, Obert, and perhaps others) consists in obtaining E by means of measuring the velocity of vibration (wave) transmission through the concrete. Since the velocity is nearly independent of size and shape of specimen, this latter method is not subject to the limitations imposed by the other methods mentioned, and hence has been the principal subject of experimentation of the authors and constitutes the principal matter of the present paper.

EXPERIMENTAL METHODS EMPLOYED

Three possible methods of measuring velocity have been investigated; involving two distinct approaches to the problem. These methods are

(a) the use of a source of sustained vibration of constant frequency to initiate waves in the concrete; and of analyzing such waves by means of a vibration pickup and cathode ray oscillograph. (b) the use of a single impact (hammer blow) to initiate a longitudinal, or compressional wave in the concrete, and measuring its velocity by means of a seismograph-type, time-recording instrument. (c) the use of a single impulse (as in (b)), but measuring its velocity by means of an electronic device consisting of two pickups, two thyratron tubes, and a triode-ballisitc galvanometer circuit. The first and last of these methods are illustrated by Fig. 1 to 4.

Selection of Present Method—(Electronic Interval Timer)

The method of (b) was abandoned because of its inadaptability to determination of wave velocity over short distances, and because of the cumbersomeness of the required film, and lack of precision in adequately recording short time intervals. The method of (a) has been superseded (at least for the present) by the simpler and more direct method of (c), yet it appears to be of sufficient importance, from the point of view of determining pavement thickness, as well as purely theoretical considerations of the wave phenomena involved, to warrant inclusion herein. A brief description of its employment to determine pavement thickness, in conjunction with the method of (c) is contained in a later section hereof.

Wave Types Involved in Test Methods

Theoretically, in the interior of an isotropic elastic solid, only two types of waves are possible [11] (a) a longitudinal (compressional) wave, in which the particles vibrate in a direction parallel with the direction of wave progression; and (b) a transverse (shear) wave, in which the particles vibrate in a direction perpendicular to the direction of wave progression. The velocities of these two types of waves are expressed as follows, using in general the notation of Timoshenko:

$$V_1 \text{ (longitudinal)} = \sqrt{\frac{\lambda + 2\mu}{\rho}} = \sqrt{\frac{E(1-\nu)}{\rho(1+\nu)(1-2\nu)}} \quad \ldots\ldots\ldots\ldots (1)$$

$$V_2 \text{ (transverse)} = \sqrt{\frac{\mu}{\rho}} = \sqrt{\frac{E}{2\rho(1+\nu)}} \quad \ldots\ldots\ldots\ldots (2)$$

where

λ = Lame's constant $= \dfrac{\nu E}{(1+\nu)(1-2\nu)}$

μ = coefficient of rigidity $= \dfrac{E}{2(1+\nu)}$

E = modulus of elasticity

ν = Poisson's ratio

ρ = density

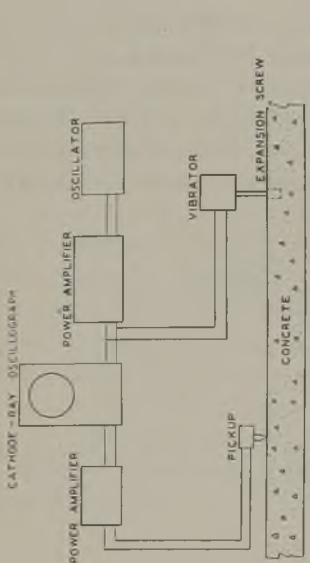


Fig. 1—Schematic diagram of velocity test apparatus—sustained vibration method

Fig. 2—Sustained vibration apparatus

Fig. 3—Electronic interval-timer apparatus

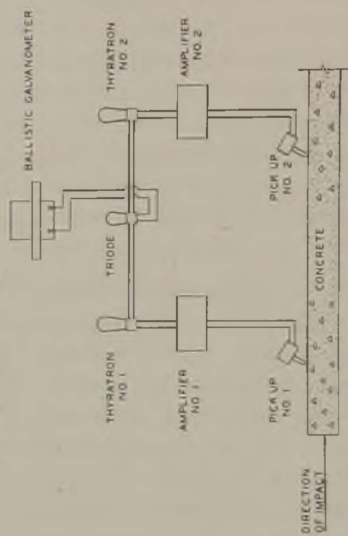


Fig. 4—Schematic diagram of velocity test apparatus—single impact method

The mathematical derivation of those relationships has been presented by Timoshenko [8], Macelwane & Sohon [11], Love [9] and others.

In elastic solids which are *not* infinite in extent, a number of wave types (and combinations thereof) are possible; however, for the sake of brevity and for the immediate purpose of this paper, only one additional type, the "transverse *vibration* wave" need be considered. This wave is present in elastic prisms (such as concrete pavements) in which the thickness is small compared to width and length, and should not be confused with the ordinary transverse (shear) wave previously referred to. If, in the analysis of the waves in such an elastic solid, the displacements are considered plane, then

$$(\lambda + 2\nu)(\nabla^2\Delta) + \rho p^2 \Delta = 0 \qquad (3)$$

$$\nu\nabla^2 \omega + \rho p^2\omega = 0, \text{ where} \qquad (4)$$

$$\Delta = \frac{\partial u}{\partial x} + \frac{\partial v}{\partial y}$$

$$W = \frac{1}{2}\left(\frac{\partial v}{\partial x} - \frac{\partial u}{\partial y}\right)$$

$$\nabla^2 = \frac{\partial^2}{\partial x^2} + \frac{\partial^2}{\partial y^2}$$

If the boundaries are free to move, Timoshenko [10] has shown that the solution of these equations is

$$4\sqrt{(1-f^2)\ (1-h^2)}\ \mathrm{Tanh}\frac{2\pi C}{l}\sqrt{1-h^2} = (2-h^2)^2$$

$$\mathrm{Tanh}\ \frac{2\pi C}{l}\sqrt{1-f^2} \qquad (5)$$

where

l = wave length of transverse vibration wave
$2C$ = thickness of prism
f = V/V_1
h = V/V_2
V = p/α, velocity of transverse vibration wave
V_1 = velocity of longitudinal (compressional) wave
V_2 = velocity of transverse (shear) wave
α = $2\pi/l$
p = $2\pi n$
n = frequency

Actually materials such as concrete are not perfectly elastic as assumed in deriving the equations; it is probable that both the modulus of elasticity and Poisson's ratio vary with wave-length and intensity of the wave source. For example, these so-called constants may vary with the velocity and weight of traffic over a concrete pavement. In analysis of the test results presented herein, these facts should be borne in mind.

DESCRIPTION OF "ELECTRONIC INTERVAL TIMER"

The electronic interval timer (Fig. 4 and 5) consists essentially of (a) two similar, vibration-pickups and amplifiers; (b) two similar thyratron tube circuits, and (c) a triode-ballistic galvanometer circuit. (Fig. 6 shows a detailed wiring-diagram). The ballistic galvanometer used with this instrument is primarily designed for laboratory use, and is not too well adapted to field testing. A number of suggestions have been made to replace the galvanometer, the most promising of which appears to be a condenser and vacuum tube voltmeter, the operation of which has been described by Weisz [15].

Fig. 5—Electronic interval-timer field test

TEST PROCEDURE

The testing procedure, using the electronic interval timer to measure longitudinal wave velocity, is as follows: An impact (hammer blow) is applied to the concrete in a horizontal direction, approximately in line with the two pickups, thus initiating a wave impulse which actuates pickup number 1 and pickup number 2 in turn, as the wave travels through the concrete. The voltage generated in pickup number 1 is amplified and serves to ionize thyratron tube number 1 which, in turn, starts the flow of a constant current through the triode-ballistic-galvanometer circuit.

When the wave impulse reaches pickup number 2 the voltage generated is amplified and ionizes thyratron number 2 which, in turn, reduces

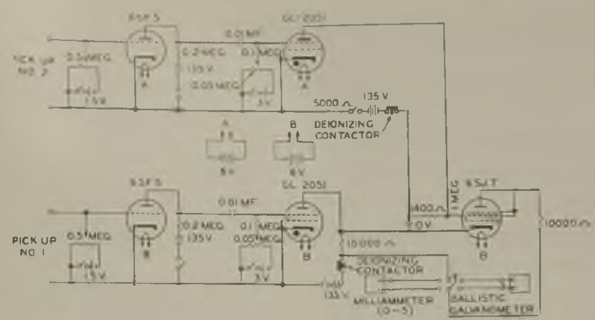


Fig. 6—Electronic interval-timer detailed wiring diagram

the current through the triode-ballistic-galvanometer circuit to zero. The effects of the wave impulse are then, (a) to start the flow of a constant current through the ballistic galvanometer when the impulse actuates pickup number 1, and (b) to stop the flow of this current when the impulse actuates pickup number 2. The deflection of the galvanometer is directly proportional to the time required for the wave to travel the distance between the pickups. The constant current is measured by ionizing thyratron number 1 and inserting a milliammeter in the triode-ballistic-galvanometer circuit.

COMPUTATION OF VELOCITY, AND MODULUS-OF-ELASTICITY

The velocity of wave transmission is computed from the formula,

$V = \dfrac{I\ D}{g\ (m\text{-}m_o)}$, in cm. per sec., where . . . . (6)

I = current through galvanometer, in microamperes

D = distance between pickups, in cm.

g = galvanometer-constant, in microcoulombs per millimeter

m = deflection of galvanometer, in millimeters, corresponding to D spacing of pickups.

m_c = deflection of galvanometer, in millimeters, which is the ordinate intercept of the straight line drawn through several points of D plotted versus m, where D-values are abscissa and m-values are ordinates.

The use of m_o in the formula eliminates the necessity of adjusting the time-constants of the two pickup circuits to precisely the same value in order to have the ordinate intercept equal zero, which would be necessary if m_o were omitted. Its use is advantageous in that it also eliminates errors in timing due to slow, unequal changes in the time constants of similar units in the two pickup circuits.

The modulus of elasticity of concrete is computed from the formulas,

$E = V^2\rho$, in dynes per sq. cm. (for laboratory beam specimens), (8) . . . . (7)

$$E = V^2\rho\,(1-\nu^2), \text{ in dynes per sq. cm. (for pavements),}^{(13)} \quad \ldots\ldots\ldots\ldots (8)$$

$$E = V^2\rho\,\frac{(1+\nu)\,(1-2\nu)}{1-\nu} \text{ in dynes per sq. cm. (for mass concrete),}^{(11)} \quad \ldots (9)$$

V = longitudinal wave velocity in cm. per second.
ρ = density of concrete, in grams per cubic centimeter.
ν = Poisson's ratio (assumed to be 0.16 for concrete).
The value of E can be expressed in pounds per square inch by multiplying dynes per square centimeter by 1.45×10^{-5}.

It will be noted that E has no correction due to Poisson's ratio, when determined from longitudinal wave velocity in laboratory beam specimens. This is explained by the assumption that in laboratory specimens, longitudinal strain is accompanied by lateral expansion or contraction thus retarding the wave, whereas, in mass concrete lateral displacements are suppressed [8] [11], causing the wave to travel at a slightly greater velocity. The situation represented by pavements is intermediate between those represented by beam specimens and by mass concrete [12], since the lateral displacements are suppressed in the direction of width but not in the direction of thickness. Assuming a Poisson's ratio of 0.16, the corrections reduce the E values by approximately 2.4 per cent and 6.1 per cent for pavement and mass concrete, respectively. It is obvious from these comparatively small correction values that they are not sources of appreciable error in the computed E values, and therefore do not, for the purposes of the present paper, require further exposition herein.

DISCUSSION OF TEST DATA

Table 1 presents test data of concrete representing several airfield runway pavements. Beams were sawed from the pavements and tested in the laboratory for dynamic E (flexural resonance method), static E, flexural strength (third point loading) and compressive strength (modified cubes). Adjacent to the location from which the beams were taken, tests of the concrete were made with the electronic interval timer for comparison with the dynamic E and flexural strength test values obtained in the laboratory. It should be emphasized here, that the concrete tested in the field was at a different temperature and moisture content than the corresponding concrete beams tested in the laboratory and for this reason the test values should not be in exact agreement. The laboratory beams were submerged in water at 70 F. for 7 days previous to testing, whereas the concrete in the field varied in temperature roughly from 50 to 100 F; and the moisture content was unknown.

The effect, of moisture at least, is of considerable significance since it has been determined, on the basis of laboratory tests conducted over

TABLE 1—COMPARATIVE TEST RESULTS OF CONCRETE PAVEMENTS*

Feature	(1) Density	Strengths		Lab. "E" Vaues		Velocity Values		
		(2) Flex.	(3) Comp.	(4) Dynamic	(5) Static	(6) Ft/Sec.	(7) "E"	(8) Flex.Str.
Army Airfield A								
Runway	2.49	850	7020	6.90	5.14	14,720	7.08	915
Apron	2.48	975	6890	6.97	5.42	14,930	7.28	960
Taxiway	2.46	800	5970	6.84	5.11	14,950	7.24	950
Army Airfield B								
Runway (a)	2.53	820	9740	7.60	5.51	14,040	6.55	830
Runway (b)	2.50	970	6520	7.05	4.68	15,400	7.80	1050
Army Airfield C								
Apron	2.49	910	8140	6.89	5.09	14,680	7.10	920
Taxiway	2.42	830	6420	5.77	4.69	14,090	6.31	790
Army Airfield D								
Runway (a)	2.46	745	6250	6.79	4.96	15,360	7.61	1010
Runway (b)	2.47	700	5600	6.77	4.72	14,390	6.73	860
Army Airfield E								
Runway (a)	2.40	800	5850	5.72	4.28	14,550	6.68	850
Runway (b)	2.48	980	8820	6.88	5.12	14,450	6.89	890
Army Airfield F								
Runway (a)	2.48	1015	8130	7.16	5.66	14,840	7.18	940
Runway (b)	2.45	805	6630	6.33	4.66	14,260	6.56	830
Army Airfield G								
Runway (a)	2.41	890	5920	6.60	4.72	13,700	5.96	730
Runway (b)	2.43	970	7270	6.74	5.48	13,700	6.00	740
Runway (c)	2.46	1010	7370	6.68	5.77	14,570	6.86	885
Army Airfield H								
Runway (a)	2.47	885	7050	6.75	4.83	14,100	6.31	790
Runway (b)	2.43	880	5660	6.26	4.72	13,890	6.27	785

*All values are averages of at least three determinations; the strengths are in psi. and "E" values are in millions psi.
(1) Determined from normal and submerged weights.
(2) Third point loading.
(3) Ends of beams, tested as "modified cubes".
(4) Flexural (resonance) vibration method.
(5) Tested in flexure; third-point loading.
(6) "Electronic Interval Timer".
(7) Computed from equations (6) and (8)
(8) Derived from curve Fig. 7.

a period of several years, that variations of E values, of the order of 12 to 15 percent may be expected between oven-dry concrete and saturated concrete. The effects of variations in temperature have not been studied to date, but it is probable that they are not so important as are those attributable to changes in moisture content; and, in all probability, moisture *gradient*.

Table 1 also presents flexural strength values corresponding to the E test values obtained with the electronic interval timer. These values were taken from the curve of Fig. 7.

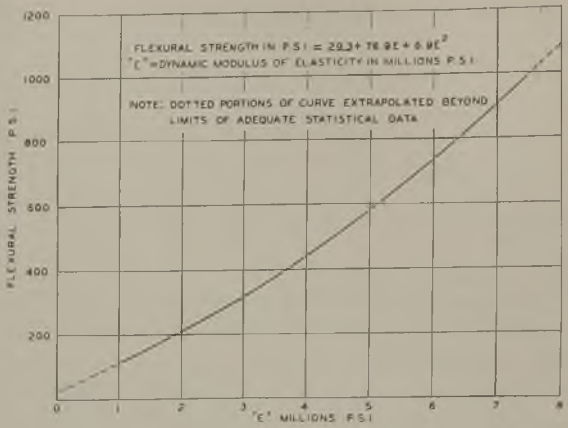


Fig. 7 — Relationship between dynamic modulus of Elasticity (E) and flexural strength of concrete beams

RELATIONSHIP OF *E* AND FLEXURAL STRENGTH

The curve of Fig. 7 was obtained from a statistical analysis of the dynamic E and flexural strength test values of approximately 1400 concrete beam specimens ranging in size from 4 by 4 by 16 in., to 8 by 9 by 40 in. However, about 1250 of these beams were of the 4 by 4 by 16 in. size. Dynamic E values were determined by the flexural resonance method; and flexural strength values were determined by the third point loading method. In the analysis of the data all of the test values were used, except for those falling in the extreme class intervals whose frequency was too low to be considered. In these instances the entire class intervals were disregarded. The effect of discarding the extreme (individual) values, which varied widely from the arithmetic mean, would have been to considerably decrease the "probable error", but would not have appreciably changed the formula for the curve of Fig. 7. The test data were divided into fourteen class-intervals of dynamic E, and the arithmetic mean was determined from the corresponding flexural strength values of each of these groups. The frequency of each class-interval used was twenty-five or more. The coordinates of the fourteen points thus determined were used in computing the constants of the second degree parabola (14) plotted in Fig. 7. This curve represents the relationship between dynamic E (flexural resonance method) and flexural strength, with a probable error of approximately 70 psi. It is possible then, to estimate with a reasonable degree of accuracy the flexural strength of concrete from laboratory dynamic E test values of beams, or from field test values of dynamic E obtained with the electronic interval timer on concrete, in situ.

It will be noted in analyzing the data that, in general, there is good agreement between the dynamic E test values obtained in the field and those obtained in the laboratory. The greatest individual difference between the test values of the 18 field locations tested is approximately 17 percent. The average of all field values differs approximately 1.5 percent from the average of all laboratory values. A similar comparison, of the flexural strength values obtained from the field E values and the use of Fig. 7, and those obtained in the laboratory, shows that the greatest individual difference is approximately 36 per cent; and the average of all field determined values differs less than 1 percent from the average of all laboratory values. These values, shown in Table 1, demonstrate that there is a high degree of correlation between the flexural strength test values (column 2) and those obtained from the electronic interval timer E values (columns 7 and 8).

PAVEMENT THICKNESS DETERMINATION

In order to determine pavement thickness from the tests discussed above, it is necessary to know the wave length and velocity of the transverse vibration wave, and the velocity of the longitudinal wave. Fig. 6 presents a graph of Timoshenko's [10] "frequency equation" showing the relationship between, the ratio $\frac{(l)}{2c}$ of wave-length (of transverse vibration waves) to thickness, and the ratio ($t = V/V_1$) of transverse vibration velocity to longitudinal velocity, for two values of Poisson's ratio (0.16 and 0.25).

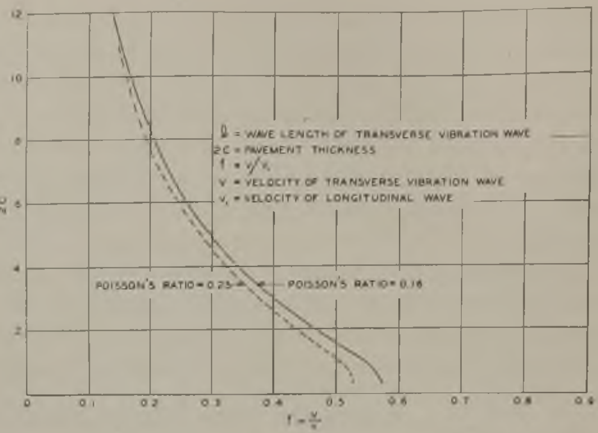
Since the ratio of V_1 to V_2 depends only on Poisson's ratio [11], h can be evaluated in terms of f. V and l can be determined from the sustained vibration test; and V_1 can be determined from the test with the electronic interval timer. This leaves but one unknown in the frequency equation; the thickness ($2c$), which can be computed.

For example; the following values l, V, and V_1 were obtained in actual tests of a concrete pavement nominally 6 in. thick: $l = 2.55$ ft. $V = 3760$ ft. per sec., $V_1 = 12{,}520$ ft. per sec. By making the required substitutions in the above formula, or by using Fig. 8, the value of thickness is found to be 6.3 in. Although it appears possible to determine the thickness of pavement to a reasonable degree of accuracy by the method described, the sustained vibration procedure of determining the wave length and velocity of the transverse vibration wave is rather tedious, and should be improved upon before it is generally adopted as a practicable routine test.

CONCLUSIONS

It is concluded (a) that the Electronic Interval Timer described herein is capable of measuring, with precision, velocities of wave transmission through concrete (or other elastic or semi-elastic materials), (b) that the

Fig. 8 — Graphical representation of Timoshenko's frequency equation



dynamic modulus of elasticity E, calculated from such velocity values, is quite as dependable as are moduli obtainable by any other method; (c) that the relationship existing between the flexural strength and dynamic modulus E of concrete has been shown to be sufficiently consistent to permit reasonably accurate predictions of flexural strength when dynamic E is known; and (d) that the method described forecasts the eventual abandonment of older, more laborious and time- consuming, and much more costly methods of obtaining knowledge concerning the flexural strength of concrete, in situ; and (e) that dynamic E values, although higher than the customary values obtained by static loading method, (being virtually dissociated from plastic flow or time yield effects) may prove to be more reliable and valuable than are the values usually used in design. This suggests the need for determinations of dynamic E and Poisson's ratio over a wide range of time, and loading (wave length and impulse intensity).

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Discussion of this paper should reach the ACI Secretary in triplicate by April 1, 1945 for publication in the JOURNAL for June 1945.

A part of PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE Vol. 41

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

Vol. 16 No. 3 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN January 1945

Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers

Substantial temporary construction used for a naval training station

THOMAS H. CREIGHTON, *Engineering News-Record*, V. 133, No. 6 (Aug. 10, 1944), pp. 90-91.
Reviewed by S. J. CHAMBERLIN

The masonry block exterior walls are supported by a continuous reinforced concrete grade beam carried on capped wooden piles. The walls were reinforced above and below openings and at changes in section or framing to minimize cracking. The 8-in. concrete block were made in various special shapes and high pressure steam cured to speed construction. Scarcity of timber, availability of masonry labor and low maintenance were factors in the selection of masonry walls.

Resistance of concrete to freezing and thawing as affected by aggregates

STANTON WALKER, Director of Engineering, National Sand and Gravel Association.
Reviewed by the Author

Reprint of a paper presented before the Conference on Plans for Postwar Highways held at the University of Tennessee, Knoxville, May 12-13, 1944. Discusses general problem of concrete durability as related to aggregates and consists principally of summary of results of freezing and thawing tests of concrete by National Sand and Gravel Research Foundation at University of Maryland and made with a variety of fine and coarse aggregates from widely separated sources. Particular attention is paid to tests of concrete made with chert gravel under different conditions of moisture content. Effects of freezing and thawing on laboratory specimens were measured by determinations of dynamic modulus of elasticity.

Simplified concrete rigid frame design

JULIAN C. SPOTTS, *Engineering News-Record*, V. 133, No. 4 (July 27, 1944) pp. 80-83
Reviewed by S. J. CHAMBERLIN

Rigid frame concrete bridges with curved soffit decks are designed by first assuming relative proportions of the frame dimensions, completing the detailed analysis, and finally assigning true numerical values to the span, height and crown thickness. Four sets of charts are presented to simplify the design. The first set gives the coefficients for fixed-end moments, for uniform load, relative stiffness and carry-over factors for parabolically shaped fixed beams. The second chart gives the moment coefficients for parabolic loads due to the weight of the beam. The third set gives the negative moments for a moving concentrated load. The fourth set gives the elements of the fixed beams (acting as the legs the rigid frame) for uniform side pressures, triangular earth pressure and for unbalanced sidesway. The procedure is illustrated by the design of a typical highway bridge.

Specifying controlled concrete

R. F. Moss, *Engineering News-Record*, V. 133, No. 6 (Aug. 10, 1944) pp. 86-89. Reviewed by S. J. Chamberlin

No incentive is provided under present-day specifications for the producer of concrete to use accurate control methods. The author recommends a straight strength specification, then it would be to the producer's advantage to have a scientifically designed mixture with the best water control and other control systems that he could get. Accurate batching equipment is needed and should be specified. Successively larger bonuses to the producer are suggested for each class of concrete within a limited strength range. If a minimum cement content is specified it should be based on approximate laboratory accuracy without factors of safety. If the cylinder tests are to govern extreme care is needed in the sampling, making, storing and testing of the specimens. The author believes that simpler specifications would result in a positive improvement in concrete quality and a reduction in its cost as well as reduction in the cost of inspection. The producer and consumer interests would be more parallel with a reduction in friction and evasion. The competitive position of concrete as a structural material would be improved.

Precise concrete curves in wind tunnel

Engineering News Record, V. 133, No. 6 (Aug. 10, 1944), pp. 108-110) Reviewed by S. J. Chamberlin

The 450-ft. long circuit varies in cross-section from 27 ft. 6 in. by 29 ft. 6 in. back of the fan to an 8 by 12 ft. test section with smooth changes of varying shape and length. The tunnel was built entirely in mass concrete to reduce vibration and improve operating conditions. Finished surfaces were brought to within $\frac{1}{16}$ in. of the designed dimension at the throat section and $\frac{5}{16}$ in. elsewhere without plaster finish of any kind. Most of the inner form faces were made with water-resistant plywood, carefully waxed and shellacked, and supported and stiffened to prevent movement. The inner form for the bellmouth was constructed on end with asbestos fiber plaster on expanded metal lath fastened to a wood backing. After the plaster was shaped to the right curvatures and hardened, it was waxed and shellacked and the whole form tilted into position. Part of the cage reinforcement was carried by overhead supports and part by concrete "chairs". The floor of the bellmouth, together with the side walls up to the level of the floor of the test section, was placed first as a separate casting, the balance being placed in a second operation. The only cold joints were horizontal, except at expansion joints where a steel slip plate was used, designed to function under either vacuum or pressure. Entrapped air in the wall forms was allowed to escape through ¼-in. holes bored through the forms. These holes did not leave form marks, evidently filling up after fulfilling their function.

Aircraft factory in France has concrete arch roof

Leon Beskin, *Civil Engineering*, V. 14, No. 6, June, 1944, pp. 240-242. Reviewed by J. R. Shank

An aircraft factory at Toulouse, France is an example of the ingenuity of French engineers where labor is cheap in comparison with materials. It also illustrates what can be done when advantage is taken by designers thoroughly conversant with mechanics, who take advantage of every method for reducing the quantities of materials used. The main building has a floor space clear of columns which is 170 x 660 ft. The construction is completely of reinforced concrete. The amount of concrete used for the entire plant was equivalent to a 7 in. slab over the floor area. The steel used amounted to 9 lbs. But 3.5 sq. ft. of forms were necessary.

The plant consists of three main buildings and several extensions. On both sides of the main 170 x 660 ft. building are two-story bays of 50 ft. span. The roof of the main building is of saw-tooth construction on reinforced concrete trusses in the planes of the lights and mid-way between them, spanning 170 ft. The roof slabs are 2 in. sloped cylindrical shells spanning 16.5 ft. In addition to supporting the roof shells with all of their special thrusts the trusses support and are a part of the wind bracing and carry two run-ways at their third-points for three 55-ft. span, 3-ton traveling cranes. The 50-ft. span side buildings are designed to brace the main building in the planes of the trusses.

The article discusses the development of the composite design showing how the thin slab takes and delivers its loading. The concrete was designed for 4500 psi at 28 days. The steel had a minimum yield point of 50,000 psi and an ultimate strength of 80,000 psi.

Aircraft hangars and terminal buildings of reinforced concrete

CHARLES S. WHITNEY, *Aeronautical Engineering Review*, V. 3, No. 9, Sept. 1944.

FROM AUTHOR'S SYNOPSIS

In this time of rapid development of new materials in the aircraft industry, reinforced concrete should be given consideration because of its adaptability to the long spans that will be needed to house large aircraft.

Recent developments in the technique of the design and manufacture of concrete have been so rapid that its possibilities have not yet been fully developed.

Many things have been discovered in the last 10 years about the characteristics of concrete which permit more precise design of structures and more accurate control of the material. Transportation to the forms by pumping and consolidation by high-speed vibrators are examples of improved construction methods. Improvements in cement result in almost doubling the strength of concrete and increasing its reliability.

It is the purpose of this paper to point out what has been accomplished in the past and how concrete can be used in the construction of hangars and accessory space for any terminal facilities that may be needed.

The period immediately after the war will undoubtedly see a tremendous development of commercial airports. It is important that hangars and terminal buildings be substantially built and economically designed so as to result in the lowest possible operating cost to the air lines using them.

This paper describes a newly developed type of long-span reinforced concrete shell construction which should provide the kind of structure needed.

Concrete in Puerto Rico

H. K. EGGLESTON, *Civil Engineering*, V. 14, No. 6, June, 1944, pp. 237-239.

Reviewed by J. R. SHANK

The engineer inexperienced in concrete construction in the United States is surprised and disillusioned when he meets the materials available and the procedures common in Puerto Rico.

Three types of sands are available; beach, river, and crushed stone. The beach sands are of shell, coral, or silica, the silica very rare. Only one out of seven or eight beach sources is satisfactory. All beach sands are of one size, 30-50 or 50-100 mesh, rather fine but devoid of fines under 100 mesh The structural qualities of the river sands are good, having been derived more or less from metamorphic and igneous rocks but the grading is very coarse. The crushed stone sands are deficient in sizes below No. 16 mesh and contain excessive quantities of dust. These artificial sands are usually harsh and unworkable, requiring more cement per yard than natural sands, and give trouble in obtaining good surface finishes, as for floors. The best results are obtained by mixing the fine, uniform beach sands with the coarse river sands. Comparative tests with standard Ottawa sand mortars show the best of these sands to be about 75 per cent whereas 120 per cent is not uncommon for the glacial sands of northern United States. The beach sands contain some salt but this is not serious inasmuch as sea water is often used for mixing without an undue reduction in strength and durability.

Two types of cement are being produced in Puerto Rico. The Ponce cement is of the type having a so-called "moderate heat" of hydration or federal specification 206a. The Puerto Rico cement is a "standard portland" or federal specification 191b. The qualities of both compare favorably with similar types in the United States.

Field control is not always popular, however economical it may be. Such questionable practices as short mixing time, plastering, dry cement dusting, laying reinforcement on the bottom forms, poor construction jointing, and poor curing protection must be combatted.

Gouin Dam, Canada, repaired with Gunite

O. Graham, *Civil Engineering*, V. 14, No. 4, Apr. 1944, pp. 147-149. Reviewed by J. R. Shank

A power dam structure on the Upper St. Maurice River, 52 miles north of Sanmaur, Que., built in 1917-18, began to show scaling disintegration after 1930 which increased rapidly so that it became necessary to make repairs. The greatest deterioration occurred on the upstream face at the winter season water level. On the downstream face, the surface of the piers of the sluice section and at expansion joints showed advanced stages of deterioration, some defect as deep as 10 in.

The repairs, in order, consisted of unwatering the upstream face for a depth of about 20 ft., chipping out the deteriorated or loose concrete and cleaning the newly exposed surfaces, applying a metallic coating, fixing of special anchorages for wire mesh fabric, placing the wire mesh and guniting in the most approved manner.

Steel sheet-pile cofferdams were used on the upstream face where the depth did not exceed 25 ft. In deep water floating caissons with three walls and a bottom were made and equipped with air-tight buoyancy compartments, so designed that the amount of water and air in these compartments could be adjusted to float them to the face of the dam, and attach them by means of anchorages, previously prepared, to the concrete. Wood cushions were provided for contact with the concrete of the dam and divers calked them with wood wedges and canvas. After this the air and water in the compartments were adjusted so that they could be dewatered without producing undue stress on the anchorages.

The metallic coating was cast-iron powder and an oxidizing agent applied with a brush and allowed to oxidize. The wire mesh was No. 6 gage 3 in. x 3 in. mesh, anchored 20 in. on centers, staggered, placed within ½ in. of the concrete and 1¼ in. in from the finished surface of the gunite. The anchors were steel nails of a special make, ¼ in. diameter and 5 to 12 in. long with a two-unit expansion shield, two of lead and two of steel. A second layer of mesh was used when the thickness of the lining exceeded 3 in.

The gunite used was 3½ cu. ft. of sand, max. size ⅜ in. per bag of cement or four parts of sand to one of cement. The gunite was placed in layers not exceeding 2 in. at a time, each succeeding layer having begun to set before the next was begun. An admixture was used in the final layer on the upstream surfaces and the top of the dam to prevent setting cracks. The surface of the last layer was screeded and floated with wooden trowels.

Micromeritics: the technology of fine particles

J. M. DALLA VALLE, Pitman Publication Corp. (New York and Chicago), 1943. Price $8.50
Reviewed by D. E. PARSONS

This excellent book brings together for the first time a large amount of widely scattered theoretical and experimental information on methods of particle-measurement, size distributions, packings, dynamics of particles, physical properties of finely-divided substances, and related subjects, and its application, especially to problems in engineering, geology, hydrology and industry. The range of sizes of particles considered is limited to 10^1 to 10^5 microns, but this is a wide range as it extends from the upper limit of colloidal materials to the larger sizes of concrete coarse aggregates. Thus, the book considers most of the particle sizes that are dealt with by cement and concrete technologists and civil and mining engineers. However, the subject matter of the book is not limited to topics that are of concern only to a few professions or industries; the fundamental principles discussed and much of the empirical data are applicable to problems in many fields. Both theoretical and empirical equations are given when these are available, and each chapter concludes with problems illustrating the use of the equations

Chapter 1 discusses the scope and fields of application of the text. Chapter 2 begins with the application of dimensional analysis to problems relating to fine particles, using Buckingham's method, and then gives the various laws of motion of particles in a fluid. Chapter 3 is a discussion of the shape and size distribution of particles and contains an excellent summary and discussion of methods of representing particle shapes and sizes and particle-size distributions. Chapters 4 and 5 consider methods of particle-size measurement, giving detailed descriptions of direct and indirect methods of measurement. The discussion of sedimentation and sieving methods is especially comprehensive. Chapter 6 discusses such characteristics of packings as the size, shape and volume of voids, and the distribution of and effects of pressures. Chapter 7 discusses electrical, optical and sonic properties of fine particles, and Chapter 8 the thermodynamics of particles, including theoretical and experimental studies of heat effects and conduction, and adsorption, condensation and evaporation phenomena. Chapter 9 treats of such chemical properties as rates of solubility, crystal growth, and oxidation, and includes a brief discussion of dust explosions. Theoretical and experimental studies of the flow of fluids through packings are discussed comprehensively in Chapter 10 and percolation, moisture-holding capacity, infiltration and related subjects in Chapter 11. Chapter 12 considers capillarity. Chapter 13 gives methods for estimating the surface areas of particles from diameters and shape factors and by permeability, adsorption and optical methods. Chapter 14 on muds and slurries discusses such properties as viscosity, plasticity, consistency and settling. Chapter 15 includes an excellent summary of information on the transportation of particles by liquids and the movement of muds and slurries in pipes; Chapter 16 the theory of fine grinding and includes experimental data; Chapters 17 and 18 are of special interest to those concerned with the collection and separation of particles from air and the control of atmospheric and industrial dusts. A large and well selected bibliography is included and both the author and the subject indexes are excellently arranged.

The book covers the broad field of fine particle technology in a thorough manner. A very small amount of space only is used in presenting current specifications and empirical rules that are likely to be out of date soon; much of the material is fundamental and should require supplementing rather than radical revision during the next few years. Research workers and engineers interested in the crushing, grinding, grading and use of pulverized materials should find that the information given is a valuable supplement to the more specialized data in their respective fields. Some readers of the ACI JOURNAL may note the omission of discussions of dilatancy and thixotropy, properties that often are of interest and importance. The book includes an adequate mathematical treatment of the problems without an unnecessary use of higher mathematics, and interpretations that permit those not well versed in mathematics to apply the information without undue difficulty. The material is well arranged and is clearly and concisely presented; the book is well written. It should be useful to the student and as a handbook for specialists in all fields in which there are problems about the properties of finely-divided substances.

Expanding cements and their application—self-stressed concrete

A. Caquot and Henry Lossier, *Le Genie Civil*, V. CXXI, No. 8, Apr. 15, 1944, pp. 61-65 and No. 9 May 1, 1944 pp. 69-71. Reviewed by R. L. Bertin

In his introduction of Mr. Lossier's paper, Mr. A. Caquot describes briefly the principles and effects of prestressing concrete by embedding therein initially stressed reinforcing steel rods. Because of the quadruple effects of immediate elasticity, shrinkage, delayed elasticity and plasticity only a fraction of the initial induced compression remains eventually in the prestressed member. For that reason, steel of high yield value with large spring effect is most effective in ultimately maintaining a high percentage of the original compression. This concept was first applied to structures by Mr. Freyssinet.

Mr. Caquot cites the principle of hooping concrete conceived by Considere as another method of restrained concrete. In this case the transverse strains of the concrete are resisted by the transverse reinforcement, thus setting up a state of volumetric restraint giving to the concrete real ductility. Mr. Caquot introduces the method proposed by Mr. Henry Lossier whereby the reinforcing steel is pretensioned automatically by using in the concrete a cement possessing the special property of swelling regularly during the setting period. At the request of Mr. Lossier the research department of Poliet and Chausson, cement manufacturers, developed such a cement. This method is the subject matter of Mr. Lossier's paper.

The author presents a chronological review of the development of prestressed concrete, starting with the descriptions of German patents granted to Doehring in 1888 on prestressed concrete floor beams. United States patents to Haas in 1909 and to Crisenberry in 1915, both of which used prestressing of the reinforcing bars particularly for the purpose of holding them in exact position during construction.

He reviews tests conducted by Lund and Koenen in 1907 and by Considere in 1903 on prestressed reinforced concrete specimens. The originators of these tests had in mind the prevention of cracks in the tension zones of reinforced concrete structures.

The expanding cements are composed of three well known stable elements: (a) ordinary portland cement, (b) sulpho-aluminous cement which acts as the expanding factor, (c) a stabilizing element which after a regulated time stops the expansion by absorbing the principal reagent. The expansion both in intensity and duration can be regulated with surprising precision by judicious proportioning of the components. The mechanism of expansion from the chemical and physical standpoints is gone into in detail.

The physical characteristics derived from laboratory and field tests are described at length for four classifications of this special cement differing only in the magnitude of their total expansion of the neat paste cured under water as follows: (a) cement fictitiously called without shrinkage 0.2 to .3 per cent; (b) slightly expansive, .5 to .6 per cent; (c) moderately expansive .8 to 1.0 per cent and (d) strongly expansive, 1.2 to 1.5 per cent; these cements are intended for use in roads and pavements, arches and for voussoirs of arches and certain reinforced concrete elements.

The period of gradual expansion terminates in 10 to 15 days after mixing for submerged pure cement paste specimen. Curves showing the shrinkage and elongations of ordinary portland and the above listed cement specimens over a period of 5 years in terms of time both water cured and water and air cured indicate that those special cements cured under water expand gradually from .30 to 1.1 per cent in about 10 days when they become stabilized.

Those cured 7 days under water then in air exhibit about the same shrinkage characteristics from 7 days to 5 years as ordinary portland at which time they seem to stabilize with a residual expansion varying from .06 per cent for type (a) to .84 per cent for type (d).

The expansion of concrete made with these cements increases with the cement content up to 500 Kg per c.m. when it reaches about ⅔ of that of neat cement after which the rate of increase drops very fast. The setting time is slower than that of portland cement requiring from 8 to 10 hours after mixing. The concrete compressive resistance generally is less than that of ordinary portland cement at early ages, about equal at 28 days and higher from then on. Concrete made with these cements is less permeable than that made with ordinary portland particularly if reinforced. Because of the high content of SO_3 the use of aggregate free of sulphates is mandatory.

Methods of testing concrete specimens made with these cements are described at length to establish the influence of composition and method of placing on the mechanical characteristics and the intensity, amplitude and energy of expansion.

A second series of tests is described relating to the adhesion between steel bars and expanding cement concrete. Also to the effect of moisture on thin and thick sections wetted on some and all of the faces. Full sized models were tested using the different classes of expanding cements in pavements, arches, combination structural steel and concrete floor systems, reinforced concrete beams and slabs.

Mr. Lossier sums up the result of his work as follows: Expanding cements have been studied and improved for more than eleven years and have reached a state of development which yields stable and well defined chemical and physical characteristics. The intensity and period of expansion are capable of exacting control both in the manufacture of the cement and in its application. The use of these cements does not differ from that of ordinary cement except for the necessity of water curing for the period of expansion—10 to 15 days. Eleven years of experimental research indicatex that concretes made with these cements are stable chemically and maintain their strength and acquired expansion. The compression resistance is at first less than that of portland cement but it gradually equals and ultimately surpasses it. When the expansion is restrained either by abutments or reinforcements, the gradual increase in compaction of the concrete increases its ultimate compressive resistance. The utilization of the energy of expansion of expanding cement concrete is found useful in the following types of structures:

Foundations—for all work cast against the earth as for instance, cast in place piles, footings, wells, etc. the resisted expansion of the concrete increases its compression and frictional resistance.

Underpinning—the expansion of the concrete takes the place of jacks in securing contact between the parts.

Repair of defective or accidently damaged work—wherever cavities or cracks in masonry structures need to be filled, the use of expanding mortar or concrete restores the structure in a manner impossible of attainment with ordinary cement.

Arched and bow string trusses—the use of expanding concrete permits the nullification of the action of shrinkage and normal plastic and elastic deformation by realizing an automatic decentering of the structure.

Tunnels, subterranian structures, dams and the like—grouting of such structures between the earth and their walls with expanding mortars improve their water tightness and eventual stability.

Pavements and roads—Observation of actual sections under working conditions indicates the possibility of eliminating the formation of cracks by suitably binding their boundaries.

Reinforced concrete—the energy of expansion of the concrete restrained by the reinforcing accomplishes automatically during the setting period, the work of prestressing utilized in structures such as arches, tanks, which can be extended advantageously to many other types such as bridges, ships, floor systems and the like.

23rd annual Proceedings of the Highway Research Board

HIGHWAY RESEARCH BOARD, 1944. Reviewed by FRED BURGGRAFF

The Proceedings of the Twenty-third Annual Meeting of the Highway Research Board contains 50 papers covering various phases of highway economics, design, materials, construction, maintenance, traffic, soils and aerial photography. Following are brief reviews of eleven of the papers, which contain information of interest to the concrete technician:

Report of Committee on Rigid Pavement Design, R. D. Bradbury, Chairman—The extensive program of airport construction during the past two years has centered attention more upon airports than upon highways. This has raised the question as to whether airport conditions are such as to require or permit substantial modification of the eatablished principles of highway pavement design when applied to airports, notwithstanding the fact that both classes of pavement are subjected to the same kind of stress-producing conditions—namely, vehicular wheel loads and climatic exposure. Such differences in conditions as do exist are embodied largely in the relative magnitudes of applied wheel loads, their corresponding areas of distribution on the pavement surface, and possibly the relative frequencies of critical stress repetition. It is possible that the differences between airport and highway loadings—although differences in degree only—might be of such a high order of magnitude as to require a fundamental modification of the commonly-used theory of stress analysis.

In the highway field, certain information that is being disclosed by the service behavior of some of our pavements under wartime traffic may have an important influence on future design practice. In the case of trucks, more vehicles are using the highways and heavier loads are being carried. Whether due wholly to this over-loading or to a combination of causes, the fact is that some new concrete pavements built under wartime restrictions have, under wartime traffic, developed serious breakage after having been in service less than six months. Undoubtedly valuable lessons are to be learned from such cases.

Experiments with Continuous Reinforcement in Concrete Pavement—A Five-Year History by H. D. Cashell and S. W. Benham—In the fall of 1938 a number of experimental sections of reinforced concrete pavement were constructed near Stilesville, Ind., on U. S. Route 40 as a cooperative research project by the Public Roads Administration and the State Highway Commission of Indiana.

The data obtained during the 5-year period show that: (1) changes in pavement elevation have been generally small and non-uniform and there is nothing to indicate that these changes have affected the length changes and the crack patterns of the sections; (2) excepting the very short sections, daily and annual changes in section length are not directly proportional to length of section; (3) the magnitude of the restraint offered by the subgrade is a function of the time in which a given temperature or moisture change occurs in the pavement; (4) maximum tensile stresses originating from subgrade restraint develop during the late summer and fall; (5) frequency of cracking increases with increase in section length; (6) surface appearance of cracks is a function of width and crack width decreases with increase in amount of longitudinal steel; (7) all of the cracks have remained so tightly closed that they have little if any structural significance.

Rigid Type Pavement Joints and Joint Spacing, by H. F. Clemmer—The Highway Department of the District of Columbia has found that the shrinkage of the concrete in setting will provide sufficient expansion space for pavements constructed during normal summer conditions, and that if planes of weakness are constructed at approximately 15 ft. intervals a certain degree of dowelling across the joints will be maintained.

Experiment and experience over ten years have proved that the division of concrete bases into slabs 12½ ft. long will provide definite control of cracking in sheet asphalt surfaces; the movement at any one joint being so slight as to be absorbed by the resilience of the bituminous surface.

The Department recommends the construction of reasonably short slabs, and when available the use of well distributed reinforcement, tie bars at all construction joints and load transfers across all transverse expansion and contraction joints.

Transverse Joints in the Design of Heavy Duty Concrete Pavements, by H. W. Giffin—This paper is an account of experience with concrete pavements in New Jersey. The situation of New Jersey is such that it is an ideal laboratory for quick and thorough testing designs of concrete pavements. Transverse joints, cracks, and pavement design are discussed as relating to ability to sustain the amount of heavy traffic using the State Highway System. Main conclusions reached are: (1) Heavy duty pavements require designs that provide load distribution at all joints and cracks; (2) Joint structures should be designed to reduce dowel restraint to the lowest practicable limit; (3) Surface water should be excluded from access to subgrade in so far as practicable; (4) The surface on which the pavement is laid should be non-erodible, have high bearing value, and preferably be somewhat porous; (5) Earth infiltration in open joints and cracks leads to spalling, blow-ups, and destruction of load transfer often accelerated by pavement growth; (6) Joint fillers should fill the joint space at all times; (7) The use of precompressed wood should be tried extensively for joint fillers; (8) Contraction joints are unsuitable for heavy duty concrete pavements and warping joints appear to be undesirable; (9) Concrete sills may be a better load distributing device than steel dowelled structures; (10) Consideration toward the early abandonment of the free end theory in construction of concrete pavements and adoption of the controlled compression theory, in which pavements are kept under compression at all times by the use of spring dowelled joint structures, is suggested as being more in accord with concrete's inherent characteristics.

Uncertainties in Design of Concrete Pavements Due to Differential Settlements and Volumetric Changes, by Francis M. Baron—The paper presents a qualitative and quantitative study of the effect of some variables on the design of concrete runways. Studied are the sensitivity of computed and measured strains, deflections, and subgrade reactions to uncertainties in variables of weather, differential settlements, and properties of materials.

Strains caused by volumetric changes of the material of the pavement and of subgrade may be, and often are, of primary importance. In some regions the design of a runway may depend upon designing for weather rather than for present loads of air carriers. Strains caused by wheel loads, however, are of interest as they may affect progressive demoralization of the runway. Clearly the importance of each source of strain depends on the scale, relative magnitude, range of uncertainty and also on the character of the source. Design requirements of runway and subgrade for purposes of weather may be opposite to those for purposes of load.

Geometry of the deflected pavement is the dominating characteristic of deformations produced by volumetric changes of subgrade or of the material of the pavement. Strains produced by these sources are studied essentially as problems of geometry and not of stress.

Temperature Changes and Duration of High and Low Temperatures in a Concrete Pavement, by W. J. Arndt—The data for this paper were obtained by the State Highway Commission of Kansas during 1936 to 1941, as a part of an experimental concrete pavement project having various types of subgrade concrete and expansion joints, under observation in Douglas County, Kan. The temperatures observed were continuous during the period of observation. A summary of the extreme high temperatures and the extreme low temperatures recorded 1 in. from the top of the pavement and 1 in. from the bottom of the pavement and from the subgrade is presented.

Considerable significance is attached to the amount and duration of the high temperatures recorded, and to the amount and duration of the low temperatures.

(1) The low temperature data are significant with regard to a freeze and thaw soundness test both as to the temperature to be used and the number of cycles to be considered.

(2) No reliance can be placed upon a study of air temperatures when one is considering concrete pavement temperatures. Naturally as the air temperatures become higher the pavement temperatures will also become higher but they do not follow any given formula or any set pattern.

(3) Any study of temperatures should be for a considerable period. No one of the years during which these data were recorded was above or below average normal temperatures by more than about 2 degrees F. Yet, any one year of the five would present a far different picture from the others.

Progress Report on California Experience with Cement Treated Bases, by T. E. Stanton, F. N. Hveem and J. L. Beatty—The California Division of Highways has built 123 miles of pavement base by mixing cement with granular materials of many kinds and compacting on the subgrade by rolling or tamping. Most of these bases have been for first class road improvements. The materials have included fine silty sands, streambed gravels, disintegrated granite, soft crushed sandstone, fairly clean sand and aggregates suitable for concrete, and many construction methods have been tried. Twenty-eight projects ranging in length from one-half mile to 13.1 miles have been constructed.

Since in most cases the material for cement treated bases in California is brought from an approved pit or quarry it was found economical and otherwise desirable to use plant mixing rather than road mixing.

These cement treated bases are giving satisfactory service. Attempts to use thin bituminous surfaces on the cement treated bases did not give satisfactory results and hence a 3-in. layer of bituminous surfacing has been adopted as standard practice on California primary roads of moderate to heavy traffic.

The mixture design is based on compressive strength requirements of 850 psi at 7 days and 1000 psi at 28 days.

Effect of Calcium Chloride on the Water Requirements, Specific Weights and Compressive Strengths of Concretes Made with Plain and Treated Cements, by H. C. Vollmer—Slump and flow tests were made on concretes using a portland cement, a wide range of water-cement ratios, and two cement factors. Comparative tests were made on concretes containing two per cent commercial calcium chloride. Calcium chloride increased the workability of all concretes tested, being slightly more effective on the richer mix in the range of slump of two to six inches.

Water requirement, compressive strength and specific weight tests were also made on concretes of constant slump using four plain cements and four cements containing an interground air-entraining agent. Comparative tests were made with two per cent calcium chloride. Compared to the plain concretes, the concretes made with treated cements required from 0 to 2 per cent less mixing water and had lower specific weights, and in most cases had lower compressive strengths; the addition of calcium chloride to the vinsol resin cement concretes decreased the required water and increased the early strengths and the specific weights.

Pumping of Rigid Pavements in Indiana, by K. B. Woods and T. E. Shelburne—This paper is a report of one of the researches conducted by the Highway Research Laboratories of Purdue University co-operating with the State Highway Commission of Indiana. Pumping action as well as the factors affecting pumping are described. Surveys of Indiana pavements show that approximately 250 miles (6 per cent) of the concrete pavements are subject to this action. In addition many hundred miles of pumping pavements have been observed in other Midwestern, Southern, and Southwestern states

Pumping is prevalent during periods of heavy rainfall on those roads which carry a large volume of heavy truck traffic. However, portions only of these roads, where certain soils predominate were found to be pumping. Test results on samples of pumping soils show that pumping generally occurs on plastic clay-like materials. Observations and results of tests on soils in several states outside of Indiana where pumping was found likewise show plastic clay-like materials. In Indiana these soil areas are lacustrine deposits, unweathered parent materials of the Wisconsin drift, weathered shales, and soils with claypan or "B" horizon development.

These studies reveal that on new construction, pumping can be minimized or eliminated by employing corrective measures. Drained insulation courses are particularly effective where plastic soils are encountered. Because of the magnitude of the problem such design features as expansion joints and load transfer devices should receive added attention. The maintenance or salvaging of the several hundred miles of pumping pavements is a serious problem, without an entirely satisfactory solution, confronting highway engineers during this critical war period.

Deflectometer for Meausuring Concrete Pavement Deflections Under Moving Loads, by R. W. Couch—The deflectometer was designed to measure concrete pavement deflections at any desired point under moving loads. The instrument that has been developed consists essentially of a ratio arm of 25 to 1 obtained by means of a train of gears and of a Kymograph drum on which deflections, speed-switch indicators and a time-graph are recorded simultaneously. Deflections of a slab at the point to be measured may be determined for any position of the moving axle load by calculating distances from the average speed of the moving load. Timing is controlled by a standard tuning fork of 100 vibrations per second. With an optimum speed of the kymograph drum, it is possible to determine the time within plus or minus 0.001 sec. Preliminary tests by static displacements show that it is possible to measure displacements of the magnification arm within plus or minus 0.001 in. Actual displacements are measured with an "88" Ames dial and magnified displacements on the kymograph chart are measured with a pair of microcalipers.

Effect of Soil and Calcium Chloride Admixtures on Soil-Cement Mixtures, by M. D. Catton and E. J. Felt—This report shows that some sandy surface soils which react poorly with cement, and therefore require high cement contents for hardening, can be improved to react in the normal manner by adding to the sand an admixture of clayey soil, or by adding a small quantity of calcium chloride.

Compressive strength, wet-dry and freeze-thaw test data are given showing the effect of the soil and clacium chloride admixtures on a number of poorly reacting sandy soils. Data are also given showing the effect of calcium chloride upon normally well reacting soils.

Construction costs are analyzed for poorly reacting sandy soils with indications that the costs may be excessive when cement alone is used. However, the admixture of clayey soil or calcium chloride to these soils, with accompanying reductions in cement requirements, will often result in costs similar to those prevailing on projects where these special problems are not present.

ACI NEWS LETTER

Vol. 16 No. 3 JOURNAL of the AMERICAN CONCRETE INSTITUTE January 1945

1945 CONVENTION MUCH CURTAILED

Business and Administrative Sessions Only

It is with very deep regret that the Board of Direction announces the cancellation of most of the plans for the 41st annual ACI convention as announced for February 13 to 16, 1945, Hotel New Yorker, New York City.

The decision was made with a high degree of Board unanimity on the basis of an urgent request from the Office of Defense Transportation. O. D. T. requests are not mandatory—many meetings are being held more or less "as usual", but the Board action made it clear that the Institute wishes to cooperate as fully as possible with the war effort, even at considerable sacrifice of its own interests in the convention's stimulation of its efforts in a specialized public service.

The 41st annual convention will include none of the customary technical sessions. Only such meetings are now scheduled as seem necessary to effective administration and the "legal formalities" of administrative succession under our by-laws; anticipated travel reduced by 90 percent.

Meetings will be as follows:

Board of Direction meets 12 o'clock, noon, at lunch Wednesday, February 14 with sessions continuing through dinner into the evening.

Advisory Committee, Raymond E. Davis, Chairman, meets 10 a.m. Thursday, February 15 and continues through the afternoon.

Publications Committee, Douglas E. Parsons, Chairman, meets 6:00 p.m. at dinner, Thursday February 15, through the evening and continuing at 9 a.m. Friday, February 16 until 11:45 a.m.

General session of ACI (the 1945 Convention) beginning at luncheon 12 o'clock noon, Friday February 16. Following the coffee: the report of tellers and induction of new officers and directors; an address by the retiring president; the award of Wason Medals (see elsewhere in these pages) and matters pertinent to the administration of Institute affairs.

To ensure the necessary quorum at the general session, dependence will be on New York City Members plus the members of the Board and of the Advisory and the Publications Committees. Local, New York members are especially urged to attend. Special provision will be made for advance reservations for the luncheon. Similar opportunity is of course open to any Institute members who plan to be in New York at that time. To observe the spirit of the compliance with the O. D. T. request (especially as no technical reports or papers will be presented for discussion) distant members are not urged to attend, except as they are included in the personnel of the meetings scheduled or are, presumptively, members-elect of the Board of Direction. (Since the annual ballot involves no contests for Board membership, the election of candidates might be assumed.)

A brief meeting of the "new Board" is scheduled to follow immediately after the general session.

The 42nd annual Convention is scheduled for New York City, February 18 to 21, 1946.

Wason Medals Won by Commodore Angas, Lt. Commander Shanley, Lieutenant Erickson and Harrison F. Gonnerman

Annual awards of Wason Medals are announced by the Board of Direction:

For "The Most Meritorious Paper" in ACI Proceedings V. 40, Commodore W. Mack Angas (CEC) USN, Lt. Comdr. E. M. Shanley (CEC) USNR, and Lt. John A. Erickson (CEC) USN, "Concrete Problems in the Construction Graving Docks by the Tremie Method" published ACI JOURNAL February 1944. This is the third successive award of this medal to members of the Civil Engineers Corps., U. S. Navy (for his 1942 paper "Architectural Concrete in the New Naval Medical Center" to Capt. Hugo C. Fischer; for their 1943 paper "The Properties and Behavior Underwater of Plastic Concrete" to Capt. P. J. Halloran and Kenneth H. Talbot.)

For "Noteworthy Research" reported in ACI Proceedings V. 40, to Harrison F. Gonnerman in the work reported in his paper "Tests of Concrete Containing Air-entraining Portland Cements or Air-entraining Materials added to Batch at Mixer," published ACI JOURNAL, June 1944. This will be Mr. Gonnerman's second research medal, he, with Paul M. Woodworth, having won it for the work reported in their 1929 paper "Tests of Retempered Concrete".

The Wason medals and certificates of award will have formal presentation at the luncheon meeting of the American Concrete Institute, Hotel New Yorker, New York City, at noon Friday, February 16th at which the medal winners will be Institute guests of honor.

New Members

The Board of Direction approved 37 applications for membership received in October and November (28 Individuals—1 Contributing—2 Corporation, 5 Junior and 1 Student) as follows:

Bamfield, A. E., Australian Cement, Ltd., Geelong, Victoria, Australia

Berg, Oswald, Jr., 1804 Lee Highway, Arlington, Va.

Blake, Leslie C., 50 Dyer Ave., Milton, Mass.

Boland, Fred L., 511 Jackson St., Petoskey, Mich.

Borden, R. C., 211 South Farr St., Provo, Utah

Brewer, Harold W., Bureau of Reclamation, Denver, Colo.

Chisholm, D. A., Dept. of Highway & Public Works, Province Bldg., Halifax, N. S., Canada

Cococcio, V. J., CMlc, B-3 14th USNCB, c/o Fleet Post Office, San Francisco, Calif.

Douglas, A. H., 65 Crichton St., Ottawa, Ontario

Gindi, Adly W., 20, Memary Pachu St., Sakaking, Cairo, Egypt

Glaze, V. L., 1402 51st St., Sacramento, Calif.

Guest, J. E., 49 North Lane, Astley, Manchester, England

Howe, Myron A., 133 Hawthorne Ave., Needham 92, Mass.

Jackman, H., Mt. Wellington Highway, Auckland, S. E. 6., New Zealand

Keniston, T. Sgt., Edward F., 3532 Center St. N. W., Washington 10, D. C.

*KOEHRING CO., 3026 West Concordia Ave., Milwaukee 10, Wisc. (R. A. Beckwith)

Laboratory for Testing Materials, 200 Dizengoff St., Tel-Aviv, Palestine (A. Arnstein)

Mauchel, Robert L., c/o Master Builders Co., 101 Park Ave., New York 17, N. Y.

McClelland, James E., 2840 Madison Rd., Cincinnati 9, Ohio

McDonell, Hubert F., 438 S. Arredonde St., Gainesville, Fla.

McGrail, Paul J., 1943 Semple St., St. Louis, Mo.

Misavage, B. A., 31 S. Locust St., Mt. Carmel, Pa.

Montgomery, Richard T., P. O. Box 844, Estes Park, Colo.

Nietz, Ernest W., P. O. Box No. 97, Gastonia, N. C.

Pennsylvania Commonwealth of, Dept. of Property & Supplies, Harrisburg, Pa. (A. Judson Warlow, Chief Engr.)

Pirtz, David, 2575 LeConte Ave., Berkeley, Calif.

Skou, C. Chr., Gimle, The Grove, Marton-in-Cleveland, Middlesbrough, Yorks, England

Sleath, Aubrey B., Box 533 N., Niantic, Conn.

Stewart, Leslie M., 155 Highland Ave., Winchester, Mass.

Sussman, William, 649 Park Ave., East Orange, N. J.

Tosca, Ernesto, Sta. Catalina No. 212 Rpto., Mendoza, Vibora, Havana, Cuba

Van Der Mark, H. J. C., P. O. Box 5467, Johannesburg, Transvaal, So. Africa (Maas & Van Der Mark)

Whelan, Donald V., 207 E. St. N. W., Washington 1, D. C.

Winslow, Ralph E., Dept. of Architecture, Rensselaer Polytechnic Institute, Troy, N. Y.

Witte, Leslie P., 1585 Elm St., Denver 7, Colo.

Wolff, Chas. R., Jr., 113 Third Ave., Columbia, Tenn.

Woodhouse, Earl H., 4095 Albatross, San Diego 3, Calif.

WHO'S WHO

Lewis H. Tuthill

has been an ACI member since 1926 and for most of that time an active one. His present paper (p. 137) is not only an important record of construction practice in World War II's concrete shipbuilding but is so presented as to involve a wealth of information applicable to high quality concrete construction for any purpose.

Mr. Tuthill is not by any means new to ACI JOURNAL pages as the record on page 96 of the current ACI Directory shows. He was chairman of Committee 614, whose report was adopted 1942, "Standard Recommended Practice for Measuring, Mixing and Placing Concrete". He has been an active worker for new ACI members as Honor Roll records will show. He was long with the Metropolitan Water District of Southern California and since 1939 with the Bureau of Reclamation, except for his recent work on concrete ships; from November 1942 to March 1943 with McCloskey & Co., Tampa, Fla. and then with the United States Maritime Commission until July, 1944 when he returned to duty with the Bureau of Reclamation. His contributions to and wide knowledge of concrete construction practice led to his appointment to the Advisory Committee recently as Chairman of Department 600, Construction.

Paul William Abeles

ACI Member since 1941, was born at Mistelbach, near Vienna, Austria, Jan. 17, 1897. He attended Classical High School, Vienna, and was graduated from Technical University, Vienna (Austrian Eng. Diploma after 5 years' course in Civil Engineering) 1921. In 1928 he took his degree of doctor of engineering (Dr. Ing.).

1922 to 1923 he worked as structural engineer with the Building Authorities at Recklinghausen, German Ruhr-coal district (examination of statical calculations and supervision of erection of industrial buildings for 17 coal mines); 1922-29 was first designer and later head of the designing and estimating department of N. Rolla & Neffe, (building and civil engineering contractors), Vienna; in this capacity designed the frame constructions of the Amalia bath, Vienna. From 1929-1939 he was a consulting engineer in Vienna, mainly engaged in industrial buildings in Central and Southeastern Europe. He carried out extensive tests on spun concrete poles, tubular and half-tubular beams and on concrete reinforced with high strength steel, with a special view to the behavior at cracking and at failure, and to the danger of rusting.

He left Austria in 1939 for London as refugee from Nazi oppression; worked in London first as designer, then as technical advisor and consultant in structural and civil engineering and investigated "prestressing".

He published simplified methods on the calculations of frame structures by the four moment theorem in *Beton und Eisen* (1924 and 1929) and in *Concrete & Constructional Engineers* (1942). A textbook on "Statics of Building Construction" appeared in Vienna 1931. Reports on the tests on spun concrete and on high strength reinforced concrete as well as treatises on the danger of rusting, on high strength reinforced concrete and on the elasticity of concrete were published in *Beton und Eisen* (1935 and 1937). *Zement* (1937) and *Concrete and Constructional Engineering* (1940 and 1942). Reports on the practical application of spun concrete, poles and floors appeared in *Zeitschrift d. Oe.I. & A.V.* (1935 and 1937). Various other publications appeared in periodicals and books. He has been a contributor to discussion in the ACI JOURNAL but this issue carries his first paper (p. 181).

Bartlett G. Long, Henry J. Kurtz and Thomas Sandenaw

who make their first appearance as authors of a paper in this JOURNAL (p. 217) are all ACI members—Long since 1934, Kurtz since 1942, Sandenaw since 1943, all of them until recently actively identified with the Army Engineer Corps' Cincinnati Testing Laboratory, at Mariemont, Ohio.

"Bart" Long, well known to many ACI members, severed his connection with the laboratory last summer and joined Mrs. Long on their ranch at Pecos, N. M. The last word ACI had from him, expressing an interest in seeing proofs of the paper, was a letter written from a Santa Fe hospital where he was getting on his feet again after being struck down by coronary thrombosis.

Mr. Long was born in St. Louis, Missouri, Apr. 3, 1892. He attended the University of Missouri, from Sept. 1911 to June, 1912, and the Washington University, from Sept. 1912 to June, 1914. He was a 1st Lt. in World War I from July 26, 1917 to May 2, 1919, having served as an instructor in the Air Service.

His professional experience includes employment by Nelson Cunliff, Engineer, and Park Commission, St. Louis, Missouri; Florida State Road Dept., Tallahassee; Woods-Hoskins-Young Co., Chicago; State Highway Department, Santa Fe, New Mexico; Bureau of Reclamation, Denver, Colo.; War Department, U. S. Engineers, Seattle; U. S. National Park Service, San Francisco; War Department, U. S. Engineers, St. Louis, and Mariemont,Ohio.

Mr. Kurtz was born in Flint, Mich., Jan. 8, 1897; was graduated from Michigan State College, B.S. in 1921, M.S. in 1928.

His professional experience includes employment by Consumers Power Co., Battle Creek, Mich.; Commonwealth Power Corp., Jackson, Mich.; Pacific Telephone & Telegraph Co., Sacramento, Calif.; Bell Telephone Laboratories, New York City; Remler Radio Co., Inc., San Francisco; National Bureau of Standards, Washington, D. C.; American Society for Testing Materials, Washington, D. C.; U. S. Coast Guard, Cambridge, Mass.; War Department, U. S. Engineers, Mariemont, Ohio.

Mr. Sandenaw took his B. S. in industrial chemistry from Montana State College in 1934; attended Ohio State University for graduate work in chemistry in 1935-36; was employed in the chemistry section United States Engineers laboratory Fort Peck, Montana; 1936-38, took an M. S. degree in chemistry from Carnegie Institute of Technology in 1939; analytical chemist in the U. S. Navy Inspection laboratory Marshall, Penna., 1939-41; employed in the chemical section of General Materials Laboratory, special engineering division, the Panama Canal, Diablo Heights, C. Z. 1941-43; in the concrete section Cincinnati Testing Laboratory, Mariemont, Ohio, 1943-44. He is now employed by the RCA Victor Division of Radio Corp. of America as a chemist in the engineering department of the vacuum tube manufacturing plant.

Harry F. Thomson

vice president, General Material Co., St. Louis, Mo. a member of the Institute since 1928, and on two occasions the author of a paper in the ACI Proceedings, a member of the ACI Board of Direction 1939 and 1940 as director for the Fifth district, again as director-at-large since 1942 has been nominated by the American Society of Civil Engineers as director for the ASCE district fourteen.

Alfred E. Lindau

died at Pearl Harbor, Hawaii, December 14. He had completed five years service as Chief Civilian Engineering Assistant to the Officer-in-Charge, Pacific Naval Air Bases—with a range of engineering service and responsibility exceeding the implications of his official title. He was reported to have been in good health; "suffered no illness; death was sudden from a heart attack."*

Very few, if any, ACI members had contributed so much over so many years and from such a wide range of activity as had Alfred Lindau and few indeed have been held in such esteem by their fellow members—for a high degree of intellectual integrity, a broad cultural background, a lively interest in a wide range of human experience and a deep human understanding. He had the rare combination of objective judicial-mindedness with great warmth of personality. His kindness was unfailing. He was unexcelled as an Institute convention presiding officer. He brought to the leadership of discussion not only his broad knowledge but his dynamic spirit and a contageous interest.

Lindau had been a member of ACI since 1909—an Honorary Member since 1935. He was a member of the Board as a Director 1915 and 1916 and continuously a Board member from 1922-1935—as vice-president 1922 and 1923, president 1924 and 1925, past president member 1926 to 1935.

Lindau was born in Sweden, March 15, 1874. An only child, and his father having died, he was brought to this country by

*In a personal letter written December 10, four days before he died (but received by the ACI Secretary more than 2 weeks later), Lindau had said " . . . I'm just now taking your advice to slow down. The pace has been a little fast for the last two or three years. I feel that the peak of construction has passed and that the facilities for carrying on the war have about been completed . . . . So-called permanent construction has given way to temporary buildings expected to last out the war." He said he had recently taken up two engineering subjects, soil mechanics and hydraulics of water distribution and had been supervising the testing of runway slabs for wheel loads far in excess of those reported thus far. "I may have something to send you one of these days when war censorship permits free elbow room."

his mother at the age of eight. He went through grade school in Chicago and worked in engineering offices as office boy, later getting some high school work, but did not complete a regular course. Admitted to the University of Michigan with much less than the usual preparatory work, he was graduated class of 1900. He won the interest of several professors who helped him acquire that foundation of mathematical and scientific knowledge on which he so richly capitalized. Some of his early engineering work was with the C. B. and Q. and the Rock-island railways. He was assistant bridge engineer with the Rock Island when he quit railroading to join A. L. Johnson, inventor of the corrugated bar in the Corrugated Bar Co., of which Lindau became chief engineer, then sales manager, vice-president. When Kalman Steel Co. took over Corrugated Bar (1924), Lindau became president of American System of Reinforcing with a plant at Libertyville, Ill. This enterprise was wiped out so far as Lindau was concerned by the depression years. With an agency for Viber concrete vibrators he entered upon a period of intensive work in their demonstration; was struck down by coronary thrombosis. He made such a complete recovery of his health that he later passed a rigorous examination for civil engineering consulting service with the Navy. Writing a friend from Pearl Harbor soon after Dec. 7, 1941, he said "the longest day I ever lived."

When Rear Admiral Ben Moreell (now Vice-Admiral), became Chief of the Navy's Bureau of Yards and Docks late in 1937, his first concern was to expand the shore facilities of the Navy as rapidly as possible to be ready for whatever might happen. Funds became available in June 1939 and the first contract to be awarded (August 5, 1939) was for the construction of the Pacific Naval Air Bases at a cost of $15 million—later expanded progressively to more than $300 million. The contract was also awarded for dry docks at a total of $18 million.

Alfred E. Lindau

The new construction involved important use of reinforced concrete in many diverse and novel applications, ranging from huge battleship graving docks for which structural concrete was placed under sixty feet of water, to batteries of concrete oil storage reservoirs built in the heart of a mountain. Alfred Lindau was particularly well qualified to assist and advise the officers of the Civil Engineer Corps of the Navy on the design and construction of such projects and was so engaged from the date of his arrival at Pearl Harbor, December 1, 1939 to the day of his death. The excellence of his work at all times was recognized, and his status advanced in July 1942. Due to the unusual type and characteristics of aggregates available in the Hawaiian Islands and to the use of tremies to place structural concrete in deep sea water, it was necessary to carry out extensive research in the design of a satisfactory mix that could be placed by the proposed means. Tests included the casting of large blocks of concrete (weighing up to 134 tons) in deep water, which were raised and examined. Twenty-two such blocks were made to assure fully satisfactory results in construction. Similar problems were successfully met in connection with special concrete work on other projects and with unusual soil conditions at many places. Dredged coral for concrete aggregate was tested for use on other islands. Lindau's work on materials, design and superintendence of construction included projects at Midway and other outlying islands—projects of supreme importance in the conduct of the war in the Pacific. His intellectual honesty, his long experience, and great human kindness and understanding won for him the respect and affection of all the officers and civilians with whom he worked. Characteristically, he aligned himself immediately with the Hawaiian Chapter of the American Society of Civil Engineers, presented and discussed papers at their meetings, served as vice president in 1942 and having been elected president was to have been installed in that office January, 1945.

Mr. Lindau's services to the Institute are all too briefly set down in the record following his name, page 68 of the 1944 ACI Directory beginning with his first paper in 1910. He worked on many committees; he had been a member of the first Joint Committee on concrete and reinforced concrete which reported in 1916, the second Joint Committee, 1920 to 1924; he became chairman of the third Joint Committee in 1932, following the death of Willis A. Slater; continued to head the committee's work until completed with the publication of the current report in 1940.

Mrs. Lindau was with her husband at Pearl Harbor. He is also survived by two daughters, Mrs. Allison O. Hunt, of Wooster, Ohio, and Capt. Marjorie Lindau, Army Nursing Corps., somewhere in France, and grandson, Stephen Hunt.

It seems appropriate to republish a sketch of Mr. Lindau and his activities written by Arthur R. Lord. It was one of a series of brief essays in which Mr. Lord discussed "Fellows of the Institute." Lindau's activities in the thirteen and a half intervening years have served to underscore much of what Mr. Lord wrote then.

FELLOWS OF THE INSTITUTE*—ALFRED E. LINDAU

BY ARTHUR R. LORD

(from *News Letter*, ACI Journal, Sept. 1931)

What is the secret formula for eternal youth that this man has somehow discovered when so many others have failed dismally in the quest? We can look back on the beginnings of reinforced concrete and there Lindau stands calmly doing a liberal share of the pioneer designing, writing the early papers and discussions, along with Talbot, Hatt and Turneaure. We can see him now no less, still active in the Institute's affairs, still leading at the thin edge of progressive advance. If he were not so masterfully a part of the present and the future we would expect to find him lonesome for many of his companions of the past. But he is a difficult subject to imagine as solitary or lonesome in any age.

And the ranging sweep of his enthusiasms! Are you fond of good books, ancient classics, modern rebellions? Alfred has read them, will be only too happy to discuss them. Are gardens and hills and natural beauty your sources of strength and peace? They are his, no less. Do you find joy in walloping an inoffensive ball as far and as accurately as you can? He's up at the crack of day (with John Ahlers) to get in one more round than the rest of us consider necessary for a golfful day. Is your recreation aided by mathematical analysis, mysterious Greek symbols and page-long equations? Well, Alfred shows every indication of heartily enjoying the conversation of Westergaard and Timoshenko when they are whooping up their specialty. And of course we all know that he sells steel and service as a bread-and-

*A special order of distinction to which nominations are made from among his friends and associates in the Institute work by the author of a series of biographical sketches of which this is the fifth—EDITOR.

butter diversion through his American System of Reinforcing.

Identified actively and prominently—almost I had said exclusively—with concrete, Alfred E. Lindau has happily avoided the professional bigotry, the narrowness of many specialists. He's an acceptable and invited speaker before the American Institute of Steel Construction. He presides over meetings at which wood and steel and concrete lie down together as lambs in the fold. Reaching out eagerly for those original contributions of thought and experience that others offer, he does not fail to repay his debts in kind and in handsome measure. He has served the American Concrete Institute with long maintained distinction, travelling great distances, displaying unflagging interest and enthusiasm, as Committeeman and as President.

In such a full rounded life it would be useless to emphasize any single achievement. Some of us hammer away so long and so noisily, driving our special spike in the Palace of Progress, that the calcium light is turned on us finally to discover what in the world all the disturbance can be about. Alfred travels so extensively and so rapidly over the entire structure, from foundations to roof (and over the whole neighborhood for that matter), that nothing less than a general illumination would serve to disclose at all adequately, to appreciate at all properly, his contributions. Each department proves too fascinating to him to permit any other to monopolize his gallant attentions. Not connected with any college faculty, he yet possesses to a superlative degree those qualities that have made certain professors preeminently useful—sympathy, understanding, outlook, inspiration, personality, humor. Some university has lost—a far greater world has gained—a wise counsellor and friendly leader by the accident that Lindau, out of many absorbing interests, chose business as the objective of his directed hours. He would have been successful likewise in any one of several engineering professions.

Frank William Capp

who for 17 years as a structural engineer with the Portland Cement Association had devoted most of his time to concrete problems of the railroad field, died of a heart attack the morning of December 22, four minutes after reaching the PCA offices, apparently in perfect health. He was not an ACI member but had contributed to its work, notably at the 1936 convention with a paper "Maintaining Concrete Structures" (ACI JOURNAL, May-June 1936).

He was born in St. John, New Brunswick, Canada, June 6, 1887, the son of a minister, Thomas Henry and Ada Ella (Baker) Capp. He was brought to the United States by his parents in 1891, and spent most of his boyhood and early manhood in Missouri. Throughout this period Mr. Capp read much in his father's library which, together with his early training imparted by his father, influenced his life greatly. He was graduated from the University of Missouri in 1909 with the degree of Bachelor of Science in Civil Engineering. He was awarded the degree of Civil Engineer by his Alma Mater in 1912. After his first work as a draftsman with the American Bridge Co. in Toledo, from 1909 to 1910 he became instructor in Civil Engineering at the University of Missouri, leaving there in 1912 to get into engineering practice with the Kansas City Terminal Railway for a short period and then with the bridge department of the Great Northern Railway, St. Paul, Minn., where, as a designer he remained from 1913 to 1920. He then joined the staff of Adolph F. Meyer, consulting hydraulics engineer in Minneapolis as an assistant engineer during the period from 1920 to 1924. He got back into railroad work in 1924 as an assistant engineer on the Chicago terminal improvements of the Illinois Central Railroad, leaving that work in 1927 when he joined the staff of the Portland Cement Association as structural engineer where he was employed chiefly on railway problems up to the time of his death.

He was active on the Committee on Masonry and the Committee on Roadway and Ballast of the American Railway Engineering Association and on a technical committee of the Way and Structures Division of the American Transit Engineering Association to which he contributed generously of his time—a member of both organizations, Mr. Capp's intimate knowledge of concrete and of railroad work made his counsel on these committees invaluable.

Mr. Capp made a careful study of railroad track construction and maintenance problems. In that connection he conceived the possibility of stabilizing track and thereby reducing maintenance cost as well as improving the riding quality of the track by injecting portland cement grout into the subgrade under pressure. The first work of this nature was done under Mr. Capp's supervision in 1936 and since that time the practice of pressure grouting roadbed has been adopted by most of the major railroads of this country

June 8, 1917 Mr. Capp married Kitcy Helen Roberts of Chicago. Mrs. Capp and his two children, Thomas Milton and Cornelia Helen survive him. The son was home on furlough after two and a half years service in the Army, mostly in North Africa and at Foggia Field, Italy. The family was planning what promised to be its most delightful Christmas day since the son and daughter were very young.

Frank Capp had a host of friends and was held in high esteem in the railway field, where he had done much valuable work, and elsewhere.

If you are going to be in New York City, February 16 and wish to attend the one brief ACI Convention Luncheon Session, write the ACI Secretary and buy a luncheon ticket—$2.25. All "noses" must be counted in advance.

Honor Roll

Feb. 1 through December 31, 1944

ACI Members listed have proposed new ACI Members as shown after their names. There should be more names on this list and more new members tallied. It is all a matter of being "Member Conscious".

F. E. Richart	13
Mehmet Tokay	8
Lewis H. Tuthill	8
J. W. Kelly	6
A. J. Boase	5½
Jacob J. Creskoff	5
Harry Erps	4
William A. Jones	4
Charles E. Wuerpel	3½
J. A. Crofts	3
Kenneth K. Knight	3
F. W. Panhorst	3
H. F. Gonnerman	2½
Douglas E. Parsons	2½
Hugh Barnes	2
Elmer B. Belt	2
R. F. Blanks	2
C. E. Everhart	2
P. J. Freeman	2
Ernst Gruenwald	2
Fred E. Hale	2
W. G. McFarland	2
H. W. Mundt	2
D. F. Roberts	2
Kenneth H. Talbot	2
A. E. Cashe	1½
Raymond E. Davis	1½
H. B. Emerson	1½
Bengt Friberg	1½
Albert Haertlein	1½
Warren Raeder	1½
K. E. Whitman	1½
R. R. Zipprodt	1½
Owen Arthur Aisher	1
R. Howard Annin	1
L. J. Baistow	1
J. F. Barton	1
S. D. Burks	1
Morgan R. Butler	1
Julian B. Carson	1
S. J. Chamberlin	1
H. Clare	1

H. F. Clemmer....................1
Joseph Di Stasio....................1
B. M. Dornblatt....................1
John J. Earley....................1
Frederic Faris....................1
H. D. Farmer....................1
H. E. French....................1
H. M. Hadley....................1
Elliot A. Haller....................1
G. H. Hodgson....................1
W. M. Honour....................1
Norman J. Huber....................1
M. E. James....................1
William R. Johnson....................1
R. R. Kaufman....................1
Paul F. Keatinge....................1
Henry L. Kennedy....................1
W. E. Lumb....................1
Donald R. MacPherson....................1
H. E. A. McCarty....................1
Roscoe J. Mason....................1
Rene Paulido Morales....................1
Ben E. Nutter....................1
R. A. Plumb....................1
John W. Poulter....................1
Walter H. Price....................1
Ansel T. Rogers....................1
John R. Ruhling....................1
M. K. Scheirer....................1
Ralph A. Sherman....................1
R. T. Sherrod....................1
B. Skramtaiev....................1
Charles Snyder....................1
E. Viens....................1
J. J. Waddell....................1
Carl B. Warren....................1
Geo. W. Whitesides....................1
Benjamin Wilk....................1
Chas. P. Williams....................1
R. J. Willson....................1
M. O. Withey....................1
E. B. Wood....................1
T. Van Dyke Woodford....................1

Each of 45 other ACI Members divides credit 50-50 with another Member.

Walter V. Allen
D. M. Asarpota
P. G. Bowie
J. M. Breen
Geo C. Britton
A. D. Ciresi
Herbert K. Cook
Theodore Crane
R. W. Crum
R. A. Crysler
G. E. Davis
R. E. Davis
Harmer Davis
R. F. Dierking
A. W. Dudley
John R. Dwyer
H. H. Edwards
W. J. Emmons
H. J. Gilkey
Homer M. Hadley
P. J. Halloran
Walter N. Handy
P. W. Helsley
M. Hirschthal
L. I. Johnstone
John V. Konlhaas
B. Leon
G. L. Lindsay
N. M. Loney
F. R. McMillan
Carl A. Menzel
Maurice C. Miller
W. H. Noonan
Charles O'Rourke
F. W. Paulson
Warren Raeder
H. H. Scofield
G. M. Serber
Roy T. Sessums
R. H. H. Stanger
A. L. Strong
Sanford E. Thompson
Calvin T. Watts
Kurt F. Wendt
E. P. H. Willett
R. W. Winters
J. C. Witt

Illinois Men Win ASCE Awards

For the first time in the history of the American Society of Civil Engineers its two annual awards have been made to the same institution—the University of Illinois. Dr. Ralph B. Peck and Dr. Nathan M. Newmark, both members of the University's civil engineering staff, will be the recipients.

One other university possesses two medals, won in different years. Illinois previously had received three medals in different years.

The honors will be formally bestowed in the ASCE annual meeting, in New York this month.

Dr. Ralph B. Peck, U. of I. research assistant professor of soil mechanics, is to receive the Norman medal, highest award of the ASCE, for his paper on "Earth Pressure Measurements in Open Cuts of the Chicago Subway."

Dr. Nathan M. Newmark, research professor in civil engineering, an ACI Member, now on leave of absence from the university for service with the army in the Pacific, is to receive the Croes medal, which since 1912 has been awarded by the ASCE to the paper "next in order of merit to the paper to which the Norman medal is awarded." Dr. Newmark's paper is on "Numerical Procedure for Computing Deflections, Moments and Buckling Loads."

A.C.I. Announces its 42nd Annual Convention: New York, Feb. 18-21, 1946

WPB Removes Cement Type Restrictions

The War Production Board announced that restrictions on portland cement, which formerly limited manufacture to three specified types, were removed November 25 through revocation of Order L-179.

When the order was originally issued in August, 1942, requirements for portland cement for military and other essential construction work were unusually high and were expected to go higher, WPB said. Total 1942 consumption was 185,000,000 barrels, the highest consumption figure on record. The order was designed to increase production of the three most commonly used types by prohibiting manufacture of modifications of these types. Other provisions of the original order, subsequently removed, had prohibited the earmarking of storage bins for individual customers. This restriction was intended to promote full utilization of all available storage space.

Total actual capacity of the cement industry is approximately 215,000,000 barrels per year, WPB Building Materials Division officials stated, and that Consumption in 1944 will amount to an estimated 40 per cent of this capacity (88,000,000 barrels); 1945 consumption, if present construction restrictions remain unchanged, is estimated at from 50 to 60 per cent of capacity.

THE AMERICAN CONCRETE INSTITUTE

is a non-profit, non-partisan organization of engineers, scientists, builders, manufacturers and representatives of industries associated in their technical interest with the field of concrete. The Institute is dedicated to the public service. Its primary objective is to assist its members and the engineering profession generally, by gathering and disseminating information about the properties and applications of concrete and reinforced concrete and their constituent materials.

For nearly four decades that primary objective has been achieved by the combined membership effort. Individually and through committees, and with the cooperation of many public and private agencies, members have correlated the results of research, from both field and laboratory, and of practices in design, construction and manufacture.

The work of the Institute has become available to the engineering profession in 40 annual volumes of ACI Proceedings (since 1929 issued periodically in the Journal of the American Concrete Institute) and in many separate publications.

Recent ACI Standards

Building Regulations for Reinforced Concrete (ACI 318-41)

63 pages in covers: 50 cents per copy. (40 cents to ACI Members)

Recommended Practice for Measuring, Mixing and Placing Concrete (ACI 614-42)

28 pages in covers: 50 cents per copy (40 cents to ACI Members)

Recommended Practice for the Use of Metal Supports for Reinforcement (ACI 319-42)

4 pages: 25 cents per copy

Recommended Practice for the Design of Concrete Mixes (ACI 613-44)

24 pages in covers: 50 cents per copy (40 cents to ACI Members)

Specification for Cast Stone (ACI 704-44)

4 pages: 25 cents per copy

Specifications for Concrete Pavements and Bases (ACI 617-44)

30 pages in covers: 50 cents per copy (40 cents to ACI Members)

Proposed Standards

The Nature of Portland Cement Paints and Proposed Recommended Practice for Their Application to Concrete Surfaces

Reported by Committee 616 as information and for discussion only. 20 pages, 25 cents per copy (Reprint from ACI JOURNAL, June 1942)

Proposed Recommended Stresses for Unreinforced Concrete

Reported by Committee 322 as information and for discussion only. 4 pages, 25 cents per copy. (Reprint from ACI JOURNAL Nov. 1942)

Proposed Recommended Practice for the Construction of Concrete Farm Silos

Reported by Committee 714 as information and for discussion only. 16 pages. 25 cents per copy. (Reprint from ACI JOURNAL, Jan. 1944)

Proposed Minimum Standard Requirements for Precast Concrete Floor Units

Reported by Committee 711 as information and for discussion only. 16 pages, 25 cents per copy. (Reprint from ACI JOURNAL, Feb. 1944)

For further information on ACI Membership and Publications address

AMERICAN CONCRETE INSTITUTE
New Center Building DETROIT 2, MICHIGAN

Sources of Equipment, Materials and Services

A reference list of advertisers who participated in the Third Annual Technical Progress Issue of the ACI JOURNAL—the pages indicated will be found in the February 1944 issue and (when it is completed) in V. 40, ACI Proceedings. **Watch for the 4th Annual Technical Progress Section in the February 1945 JOURNAL.**

Concrete Products Plant Equipment

page

Besser Manufacturing Co., 800 44th St., Alpena, Mich. 349
—Concrete products plant equipment

Stearns Manufacturing Co., Inc., Adrian, Mich. 339
—Vibration and tamp type block machines, mixers and skip loaders.

Construction Equipment

Baily Vibrator Co., 1539 Wood St., Philadelphia 2, Pa. 352
—Concrete vibrators

Blaw-Knox Division of Blaw-Knox Co., Farmers Bank Bldg., Pittsburgh, Pa. 340-1
—Truck mixer loading and bulk cement plants, road building equipment, buckets

Butler Bin Co., Waukesha, Wis. 379
—Central mix, ready-mix and bulk cement plants, cement handling equipment

Chain Belt Co., Milwaukee, Wis. 376-7
—Mixers, pavers, pumps

Electric Tamper & Equipment Co., Ludington, Mich. 366-7
—Concrete vibrators

Fuller Co., Catasauqua, Pa. 343
—Unloading and conveying pulverized materials

Heltzel Steel Form & Iron Co., Warren, Ohio 388-9
—Pavement expansion joint beams

Jaeger Machine Co., The, Columbus, Ohio 344-5
—Concrete paving equipment

Kelley Electric Machine Co., 287 Hinman Ave., Buffalo 17, N. Y. 368-9
—Floor finishing equipment

Koehring Co., Milwaukee, Wis. 347
—Tilting and non-tilting construction mixers

Mall Tool Co., 7703 So. Chicago Ave., Chicago 19, Ill. 365
—Concrete vibrators

Master Vibrator Co., 200 Davis Ave., Dayton, Ohio 394
—Concrete vibrators

Ransome Machinery Co., Dunellen, N. J. 378
—Paving mixers

Smith Co., The, T. L. 2897 No. 32nd St., Wilwaukee 10, Wis. 374-5
—Mixer and truck mixer efficiency

Viber Co., 726 So. Flower St., Burbank, Calif. 355
—Concrete vibrators

Whiteman Manufacturing Co., 3249 Casitas Ave., Los Angeles 26 Calif. 392-3
—Floor construction and finishing equipment

Contractors, Engineers and Special Services

Materials and Accessories

Special ACI Publications

ACI Manual of Concrete Inspection (July 1941)

This 140-page book (pocket size) is the work of ACI Committee 611, Inspection of Concrete. It sets up what good practice requires of concrete inspectors and a background of information on the "why" of such good practice. Price $1.00—to ACI members 75 cents.

"The Joint Committee Report" (June 1940)

The Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete submitting "Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete," represents the ten-year work of the third Joint Committee, consisting of affiliated committees of the American Concrete Institute, American Institute of Architects, American Railway Engineering Association, American Society of Civil Engineers, American Society for Testing Materials, Portland Cement Association. Published June 15, 1940; 140 pages. Price $1.50—to ACI members. $1 00.

Reinforced Concrete Design Handbook (Dec. 1939)

This report of ACI Committee 317 is in increasing demand. From the Committee's Foreword: "One of the important objectives of the committee has been to prepare tables covering as large a range of unit stresses as may be met in general practice. A second and equally important aim has been to reduce the design of members under combined bending and axial load to the same simple form as is used in the solution of common flexural problems."—132 pages, price $2.00—$1.00 to ACI members.

Concrete Primer (Feb. 1928)

Prepared for ACI by F. R. McMillan, it had five separate printings by the Institute alone (totalling nearly 70,000 copies). By special arrangement it has been translated and published abroad in many different languages. It is still going strong. In the foreword the author said "This primer is an attempt to develop in simple terms the principles governing concrete mixtures and to show how a knowledge of these principles and of the properties of cement can be applied to the production of permanent structures in concrete." 46 pages, 25 cents (cheaper in quantity).

Air Entrainment in Concrete (1944)

92 pages of reports of laboratory data and field experience including a 31-page paper by H. F. Gonnerman, "Tests of Concretes Containing Air-entraining Portland Cements or Air-entraining Materials Added to Batch at Mixer" and 61 pages of the contributions of 15 participants in a 1944 ACI Convention Symposium, "Concretes Containing Air-entraining Agents," reprinted (in special covers) from the ACI JOURNAL for June, 1944. $1.25 per copy; 75 cents to Members.

For further information on ACI Membership and Publications address

AMERICAN CONCRETE INSTITUTE

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"I'll tell you GOOD TIMES ARE COMING!"

"I'll tell you BAD TIMES AHEAD!"

What's it to you?–PLENTY!

OKAY! Maybe the optimists are right. There'll be good times after the war.

OKAY! Maybe the pessimists are right. We'll have another depression.

What's it to you? PLENTY! It's largely in *your* hands as to which we'll have.

The one way to make it *good times* is to do your share to help keep prices down now!

That means *buying only what you really need*. It means *paying off your debts, saving your money*.

And here's where you're lucky. The same program that helps insure prosperity is also the best possible way to get yourself in shape to take another depression if one does come. So what? *You're right both ways*—if you save your money. *You lose both ways*—if you splurge right now.

Think it over, fella. Then get in there and fight. Read—and observe—the four rules to head off inflation. The war isn't over yet. And the war against *inflation* isn't over yet—by a long shot. Remember World War I? The cost of living rose twice as fast *after* the war as it did during the war itself.

4 THINGS TO DO to keep prices down and help avoid another depression

1. Buy only what you really need.

2. When you buy, pay no more than ceiling prices. Pay your ration points in full.

3. Keep your *own* prices down. Don't take advantage of war conditions to ask more for your labor, your services, or the goods you sell.

4. *Save*. Buy and hold all the War Bonds you can afford—to help pay for the war and insure your future. Keep up your insurance.

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