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Design Loads for Inserts Embedded in Concrete

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T. W. Reichard, E. F. Carpenter, and E. V. Leyendecker

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Contents

Notation

f_c' = compressive strength of concrete in psi
kip = 1,000 lb
P = pull-out load of an insert in kips
P_4 = pull-out load of an insert in a 4 × 4 specimen* in kips
P_{4h} = calculated pull-out load in a normal weight concrete 4 × 4 specimen = $1.63 + 0.18\sqrt{f_c'}$ in kips
P_{4s} = calculated pull-out load in a semi-lightweight concrete 4 × 4 specimen = $3.06 + 0.13\sqrt{f_c'}$ in kips
P_{4u} = calculated pull-out load in a 4 × 4 specimen = $2.0 + 0.0012W_c\sqrt{f_c'}$ in kips
P_n = normalized pull-out load for a reference strength concrete

$$= P\,\frac{\sqrt{\text{reference } f_c'}}{\sqrt{\text{actual } f_c'}}\text{, in kips}$$

$P_{\max}$ = maximum load applied to an insert in a fatigue test in kips
$P_{\min}$ = minimum load applied to an insert in a fatigue test in kips
R = ratio of cyclic loading = $P_{\min}/P_{\max}$
W_c = unit weight of concrete in lb/ft³
W_h = unit weight of normal weight concrete in lb/ft³
W_l = unit weight of lightweight concrete in lb/ft³
W_s = unit weight of semi-lightweight concrete in lb/ft³

Insert Pull-Out Load Reduction Factors Applied to the General Pull-Out Load Formula,
$P_{4u} = 2.0 + 0.0012W_c\sqrt{f_c'}$

ϕ = total reduction factor
ϕ_{cf} = minimum combined fatigue reduction factor
ϕ_{cfh} = combined fatigue reduction factor for normal weight concrete = $\phi_{eh}\phi_{fh}$
ϕ_{cfs} = combined fatigue reduction factor for semi-lightweight concrete = $\phi_{es}\phi_{fs}$
ϕ_{eh} = experimental scatter reduction factor for inserts embedded in normal weight concrete
ϕ_{es} = experimental scatter reduction factor for inserts embedded in semi-lightweight concrete
ϕ_{fh} = fatigue load reduction factor for inserts embedded in normal weight concrete
ϕ_{fs} = fatigue load reduction factor for inserts embedded in semi-lightweight concrete
ϕ_s = sustained load reduction factor
ϕ_t = flexural cracking reduction factor for inserts embedded in slabs spanning more than four feet.

Conversion Units

Length
1 in = 0.0254 meter
1 ft = 0.3048 meter

Area
1 in² = 6.4516 × 10⁻⁴ meter²
1 ft² = 0.09290 meter²

Force
1 lb (lbf) = 4.448 newton
1 kip = 4448 newton

Pressure, Stress
1 psi = 6895 newton/meter²
1 psf = 47.88 newton/meter²

Mass/Volume
1 lb/ft³ (lbm/ft³) = 16.02 kilogram/meter³

Cement Measure
1 sack is approximately 94 lb = 42.6 kilogram

Reinforcement Steel
No. 3 bars—Nominal diameter = ⅜ in = 0.0095 meter
No. 4 bars—Nominal diameter = ½ in = 0.0127 meter
No. 5 bars—Nominal diameter = ⅝ in = 0.0159 meter

* Nominal width and length of specimen in feet.

Design Loads for Inserts Embedded in Concrete

T. W. Reichard, E. F. Carpenter,[1] and E. V. Leyendecker

Detailed test procedures are presented for a research program on cast-in-place inserts embedded in reinforced concrete. Three types of inserts, two of malleable iron and one of ductile steel, capable of receiving a $\frac{3}{4}$-inch threaded rod were tested. Other variables included concrete aggregate type, concrete strength, reinforcement cover and spacing, angular loading, flexural cracking, sustained load, and fatigue loading.

It was found that the pull-out load for an insert could be approximated by a linear function of the concrete unit weight and square root of the compressive strength in a statically loaded reinforced concrete slab. The effect of other variables is related to the insert pull-out loads in these slabs. Design recommendations are presented.

Key words: Anchors; concrete slabs; design loads; fatigue; inserts; pull-out loads; sustained load.

1. Introduction

1.1. General

As the cost of construction continues to increase, more and more designers are looking for methods to optimize floor space utilization. One method commonly used is to suspend from the ceiling equipment which might otherwise be occupying premium floor space. An increasing number of devices suitable for suspending such loads are being used in industrial, institutional, and commercial buildings.

One such device being used with concrete slab construction is an anchor commonly called a concrete insert. These concrete inserts are made to receive either an ordinary threaded rod, a bolt head, or a special nut. They are simply fastened to the formwork prior to placing the concrete. This simplicity offers advantages over other devices such as embedded anchor bolts which must penetrate the normally reusable formwork. Ordinarily, the manufacturer's catalogs are the only source of data regarding the load-carrying capacity of most of these inserts. Table 1 is a listing of such catalog data for some typical inserts made to receive $\frac{3}{4}$-in diameter threaded rods or bolts.

A recent publication [1][2] presents some load capacity data for two types of inserts. These data indicate that the load capacity of inserts is partially a function of their embedded length. In an investigation of drilled-in anchors, Adams [2] presents data which also indicate that the embedded length of the insert is a major variable. Adams also shows that the load-carrying capacity of his anchors was a function of the concrete strength. Kennedy and Crawley [3], in a report of an investigation concerning the load capacity of form anchors for mass concrete, observed that the failure of the concrete around the anchors was influenced by the bending moments. As far as is known, no systematic study of the factors which affect the load carrying capacity of inserts in reinforced concrete slabs has been published. It is known that some manufacturers have investigated certain variables although the scope and test results are not known.

1.2. Objective

Due to the limited availability of concrete insert data the Building Research Division, in cooperation with the Post Office Department, conducted a comprehensive study of the variables influencing the ultimate load carrying capacity of some commonly used inserts. The objective of the investigation was to propose design criteria for inserts embedded in reinforced concrete slabs such as those found in postal facilities.

The following variables were studied:

a. Insert type
b. Concrete aggregate type
c. Concrete strength
d. Reinforcement cover
e. Reinforcement spacing
f. Angular load effect
   1. Angular displacement of insert
   2. Angular insert load
g. Flexural cracking effect
h. Sustained load
i. Fatigue load

2. Materials and Test Specimens

2.1. Inserts

Three different inserts were used in the main part of the investigation. In preliminary tests [4], six different insert types were used, but the number was reduced to three for this study to satisfy a Post

[1] Present address: The Mitre Corporation, 1820 Dolley Madison Blvd., McLean, Va. 22101.

[2] Figures in brackets indicate the literature references at the end of this paper.

Table 1. *Typical manufacturer's catalog data for $\frac{3}{4}$-inch inserts*

Insert	Catalog name	Type of metal	Overall length	Manufacturer's ultimate strength in concrete	Manufacturer's working load
			in	*lb*	*lb*
A	Universal-all size nut	Malleable iron	$3\frac{5}{16}$	....	3,000
B	Concrete insert	Zinc casting alloy	3	12,500	....
C	....do	....do	$1\frac{11}{16}$	6,900	3,000
D	Malleable adjustable	Malleable iron	$2\frac{1}{2}$	7,650	3,020
E	Threaded insert	Steel	$3\frac{5}{8}$	10,000	....
F	....do	Gray cast iron	$3\frac{1}{4}$	12,500	3,100
G	....do	Malleable iron	$3\frac{1}{4}$	15,300	3,800
H	Rocket	....do	$3\frac{5}{8}$	11,900	3,020
I	Kohler	Gray cast iron	3	12,400	3,020
J	Thin slab ferrule	Steel	$1\frac{5}{8}$	....	....
K	Mitey-Mite	Malleable iron	$1\frac{1}{2}$	6,600	2,500

Note: Inserts recommended as suitable for use with $4\frac{1}{2}$ in thick concrete slabs.

Office Department specification requiring malleable iron inserts suitable for attaching $\frac{3}{4}$-in-diameter threaded rods. The deleted inserts were made from gray cast iron, would not receive a $\frac{3}{4}$-in-diameter threaded rod, or were special purpose inserts [4].

The specification requiring malleable iron inserts was probably designed to guard against the use of a brittle material such as gray cast iron. A ductile steel insert does not meet the strict wording of the specification although one was included in this investigation.

2.1.1. Type 1 Insert

The Type 1 insert used in this investigation is described in the catalogs as a malleable iron threaded insert, especially designed for use where impact or vibration is a factor. Figure 1 is a photograph of this insert, and it indicates the significant dimensions. These inserts were fastened to the plywood concrete form with 1-in roofing nails driven through the two side lugs visible at the bottom of the insert in figure 1.

2.1.2. Type 2 Insert

The Type 2 insert illustrated in figure 2, is called a threaded insert by the manufacturer. The closed end spool is machined from mild steel. The loop welded to the spool is 0.26-in-diameter steel wire, with an ultimate strength of about 65,000 psi. These inserts are set in the concrete form by using a plastic plug or cup which is nailed to the form prior to forcing the insert spool over the cup.

2.1.3. Type 3 Insert

The Type 3 malleable iron insert illustrated in figure 3 is also called a threaded insert by the manu-

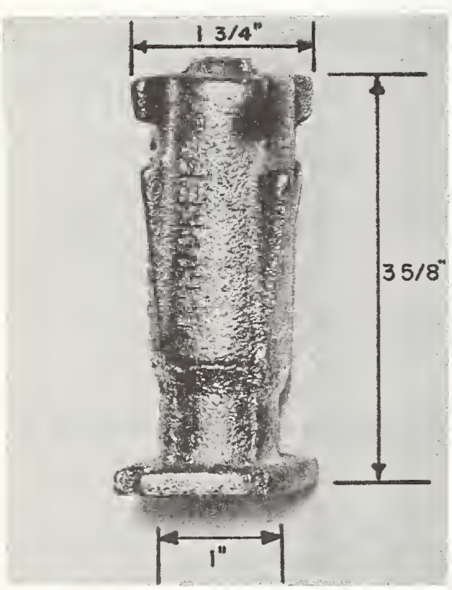

Figure 1. *Type 1 insert.*

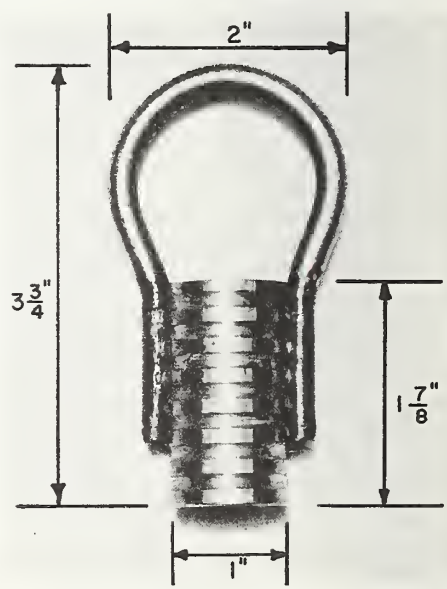

Figure 2. *Type 2 insert.*

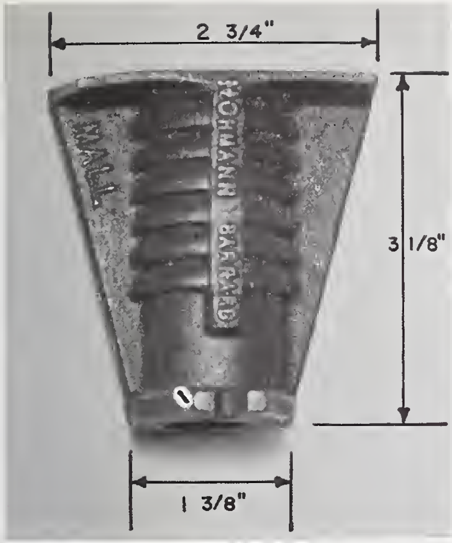

FIGURE 3. *Type 3 insert.*

facturer. These inserts were set by driving 1-in roofing nails into the form through the two side lugs visible at the bottom of the insert in figure 3.

2.2. Concrete

Table 2 describes the seven types of aggregates used in making the concrete. As indicated in this table, two of the coarse aggregates were normal weight and five were expanded-shale lightweight aggregates. All concretes were mixed in 6 to 10 cu yd commercial transit mixers in 3 cu yd batches or larger. The normal weight concretes were standard mixes for the supplier, as were the concretes made with the L-1 lightweight aggregates. For the concretes made with L-2, L-3, L-4, and L-5 lightweight aggregates, the readymix contractor supplied the cement and usually the sand. The lightweight aggregate was measured and placed in the mixer by NBS personnel. These concretes were proportioned as recommended by the aggregate producer, except that water was added until a suitable consistency was attained. A 4 to 6 in slump was the target consistency for the normal weight concrete, and a 2 to 4 in slump for the lightweight aggregate concrete.

Some problems were encountered in acquiring the desired unit weight and consistency with the readymix L-1 semi-lightweight concrete. These difficulties were probably a result of the rather small batch sizes in the large mixers. Control cylinders (6-in × 12-in) were cast from each batch of concrete for determining compressive and splitting tensile strengths. All test specimens were consolidated in the mold by internal vibration. After removal from the molds, all specimens were air-dried until tested. The specimens were tested at various ages, ranging from 5 to 42 days. Compressive strength determinations were made in accordance with ASTM Method C-39. The splitting tensile strength tests were made in accordance with ASTM C-496, except for the curing conditions.

2.2.1. Normal Weight Aggregate Concrete

Table 3 gives the properties of the concrete mixes made with normal weight aggregates. The sand to stone ratio was 45 to 55 for all mixes except H-2 and H-2a. For these two mixes the proportions were 40 to 60.

2.2.2. Lightweight Aggregate Concrete

Table 4 is a listing of the concretes made with the lightweight aggregates. These concretes were all semi-lightweight[3] except for L-4. The proportions recommended by the producer of the lightweight aggregate were used throughout this investigation.

2.3. Reinforcement

All principal reinforcement was No. 5 deformed, intermediate grade steel bars placed on bolsters to provide the required concrete cover. Temperature steel was generally No. 3 bars spaced at about 12 in on centers.

[3] Semi-lightweight concrete is a concrete containing the coarse lightweight aggregate, but with a natural sand replacing the lightweight aggregate fines.

TABLE 2. *Aggregate descriptions*

Aggregate designation	Max. size	Type of aggregate		Source of aggregate	
		Coarse	Fines	Coarse	Fines
	in				
H-1	$\frac{3}{4}$	Crushed stone	Natural sand	Md.	Md.
H-2	$\frac{3}{4}$	Gravel	Natural sand	Md.	Md.
L-1	$\frac{3}{4}$	Expanded shale	Natural sand	Va.	Md.
L-2	$\frac{3}{4}$	Expanded shale	Natural sand	Ga.	Md.
L-3	$\frac{3}{8}$	Expanded shale	Natural sand	Ill.	Md.
L-4	$\frac{1}{2}$	Expanded shale	Expanded shale	Calif.	Calif.
L-5	$\frac{3}{4}$	Expanded shale	Natural sand	Texas	Md.

Note: Although these lightweight aggregates are identified as being expanded shales, the actual raw materials could be either shale, clay, or slate.

TABLE 3. *Concrete mixes made with normal weight aggregates*

Concrete	Nominal cement content	Measured slump	Age at test	Splitting strength	Compressive strength
	sacks/yd³	*in*	*days*	*psi*	*psi*
H-1	5	...	7	...	3480
H-2	5	4	7	380	3150
H-1A	6	5	27	...	3950
H-2A	6	5	19	450	4110
S-1	5	6	21	...	3830
S-2	5	6½	15	...	3940
S-3	5	5	12	...	3110
S-4	4	7	18	...	2640
W-1	5	5	7	420	3330
F-1	5	2½	10	510	5280
F-3	5	...	13	340	3830
F-4	5	6	13	420	3500
C-1	5	5½	34	...	3630
C-3	5	5	40	440	5010
X-1A	5	4½	23	460	4560
X-1B	4½	5½	20	368	2990
X-2A	5	4½	24	445	4560
X-2B	5	4½	28	338	3670
X-3A	4½	6	7	340	3180
X-3B	7	3 and 6	14	434	5570
X-4A	4½	6	20	334	3440
X-20A	5	5	10	320	4090
X-20B	5	6	9	320	3100
X-20C	5	4	7	312	3200

Note: All normal weight concretes were made with the H-1 crushed stone coarse aggregate except for concretes H-2 and H-2A which were made with the H-2 gravel.

2.4. Test Specimens

Four general types of concrete test specimens were used for the inserts in this investigation. Except for one waffle slab all specimens were 4½ in thick and were designed as one-way slabs. The slab length and width were dictated by the purpose for which it was made as discussed below. All 4½-in thick specimens were cast in wood forms, with the inserts nailed to the bottom of the form. The specimens were turned over for testing convenience.

2.4.1. 4 × 4 Specimens

Ninety percent of the almost 400 specimens tested were nominal 4 × 4 specimens. The actual

TABLE 4. *Concrete mixes made with lightweight aggregates*

Concrete	Aggregate	Nominal cement content	Measured slump	Fresh unit weight	Age at test	Splitting strength	Compressive strength
		sacks/yd³	*In*	*lb/ft³*	*days*	*psi*	*psi*
L-1	L-1	6	6½	117	35	270	2640
L-2	L-2	5¾	1¾	114	8	350	3040
L-3	L-3	5½	1	115	17	400	5420
L-4	L-4	5¼	1½	96	5	280	3300
L-5	L-5	5½	2	120	8	330	3370
L-1A	L-1	8	2½	119	19	370	5050
L-2A	L-2	7	3	117	20	480	5200
C-2	L-1	6	2	119	31	...	2800
C-4	L-1	6	8½	118	42	230	2350
F-2	L-1	6	3½	120	11	...	3450
X-2C	L-1	6	4	121	31	440	4200
X-2D	L-1	6	3	119	35	360	3810

Note: All concretes were semi-lightweight (sanded) except for L-4 which was lightweight.

dimensions were 42 in × 45 in × 4½ in thick. This specimen size was arrived at after preliminary tests [4] indicated that a smaller specimen would not be satisfactory for single insert pull-out tests. A single insert was cast in the specimen, with the reinforcement placed symmetrically about the insert, and with the principal steel placed parallel to the long dimension, with ¾-in cover.

Figure 4 is a photograph of a static pull-out test on a 4 × 4 specimen and illustrates a typical failure of the concrete. It is obvious from this illustration that the failure zone would be changed if the test stand supports were closer together.

2.4.2. 4 × 22 Specimens

Seven 4 × 22 specimens were cast with a width of 42 in and a length of 22 ft. The principal reinforcement in all these slabs was No. 5 bars at 6 in on centers and with ¾-in cover. Four of the seven specimens were designed as one-way slabs to be continuous over three supports with two 10-ft spans. Negative moment reinforcement for these slabs was No. 5 bars at 8 in on center, placed with ¾-in cover from the top surface as cast. Nineteen inserts were cast in each continuous slab at about 12 in on center. These slabs were cast from concrete designated as S-1, S-2, S-3, and S-4. In addition to the long slabs, four companion 4 × 4 slabs with single inserts were cast with the S-1, S-2, and S-3 concretes.

Figure 5 illustrates a 4 × 22 continuous slab ready for a test to determine the effect of bending moment on the pull-out strength of the insert. The positions of the 19 inserts were indicated by the eyebolts. It should be noted that, although the specimens were cast in the orientation they would be on the job, all slabs were turned over for testing convenience.

Three additional 4 × 22 specimens, designed as simple span, one-way slabs, were tested with a 20-ft span to investigate the pull-out strength of inserts in long thin slabs. Five inserts were cast in each of these specimens. One insert was at midspan, two were at 30 in on either side of midspan, and two were

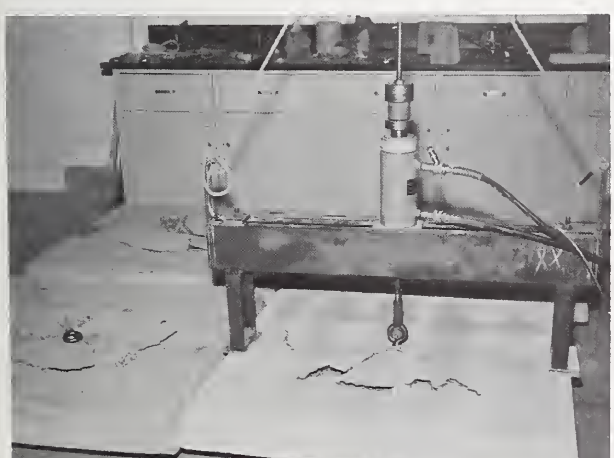

Figure 4. *Static test on 4 × 4 specimen.*

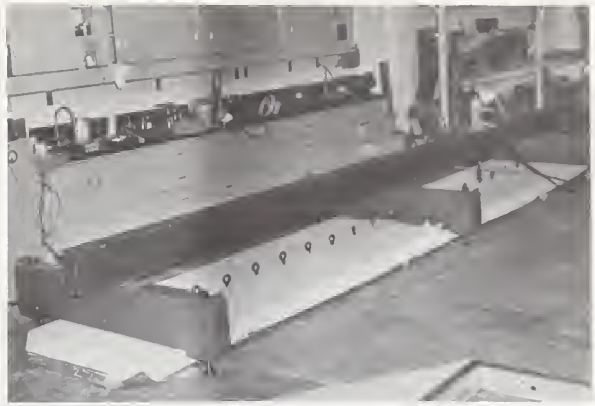

Figure 5. *Static test on 4 × 22 specimen.*

at 31 in from either end. The two inserts near the ends were tested as if they had been cast as separate 4 × 4 control specimens. These three specimens were cast from concrete designated as X-20A, X-20B, and X-20C.

2.4.3. 4 × 16 Specimens

Four 4 × 16 fatigue test specimens, with a width of 45 in and a length of 15 ft 9 in, were cast from each of the X-2A, X-2B, X-2C, and X-2D concretes. The inserts were spaced so that each specimen had two inserts, spaced at 64 in from either end, available for fatigue tests on a 10-ft span. In addition, most specimens contained inserts placed at 24 in from either end, for simulated 4 × 4 pull-out tests. Figure 6 is a photograph of a 4 × 16 specimen prior to

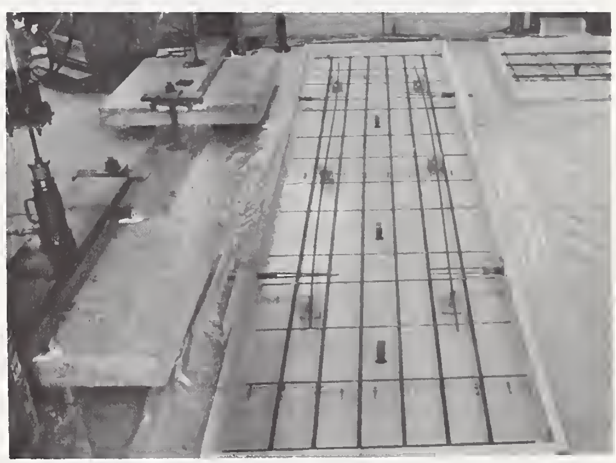

Figure 6. *Fabrication of 4 × 16 specimens.*

placement of the concrete. The two top bars visible in this picture were for prevention of damage while handling. These specimens were made in order to determine the effect of bending moment on the pull-out strength of inserts subject to cyclic fatigue loading.

2.4.4. Waffle Slab Specimen

A single waffle slab specimen with overall dimensions of 6 ft × 15 ft × 12 in was cast from the normal weight concrete designated as W-1, using 10-in deep 30-in × 30-in metal pans. Figure 7 is a photograph of the waffle slab prior to placement of the concrete. Two No. 5 bars were placed $\frac{3}{4}$ in from the bottom of each 6-in rib in the long direction, and two No. 5 bars were placed on top of these in the opposite direction. Welded wire fabric (6 × 6–10 × 10) was placed over the pans. Inserts were placed at each of the interior intersections (36 in on centers) of the ribs.

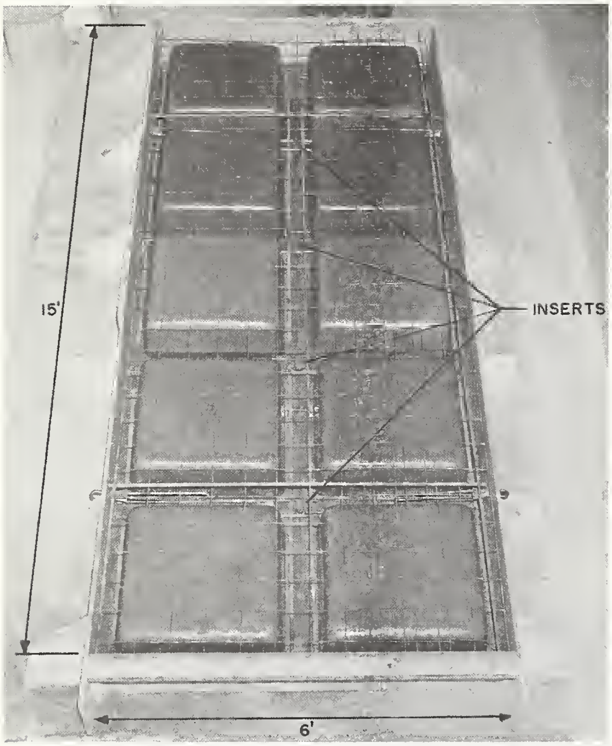

FIGURE 7. *Fabrication of waffle slab specimen.*

3. Test Equipment and Procedures

3.1. Static Tests

3.1.1. Static Test Equipment

Figures 4, 5, and 8 illustrate the equipment used for applying short term static pull-out loads to the inserts embedded in the concrete specimens. The apparatus in figure 4 was used for the 4 × 4 specimens, while that in figures 5 and 8 was used for the 4 × 22 specimens. The basic parts were:

a. a steel stand with supports spaced at the required distance;
b. a center-hole 60-kip hydraulic ram powered with a remote hand-operated pump;
c. a center-hole 60-kip load cell;
d. an X-Y plotter for recording the output of the load cell; and
e. a $\frac{3}{4}$ in high-strength steel pull-rod.

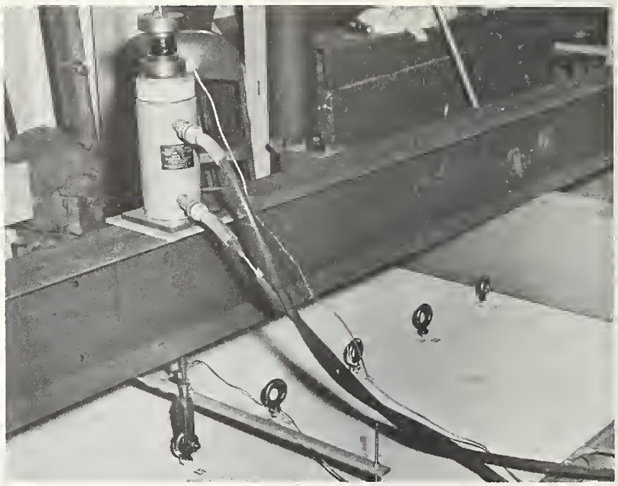

FIGURE 8. *Close-up of static test on 4 × 22 specimen.*

When testing the 4 × 4 specimens, the test stand had an effective span of 42 in. The effective span of the stand for the longer specimens was the same as the span of the test specimen. The test stand was always placed so that its span was in the same direction as the main reinforcement.

For some tests, the vertical movement of the insert was measured by using an LVDT displacement transducer. The LVDT, visible in figure 8, was mounted on the pull-rod and the core rested on a bridge, supported at the mid-span edges of the slab. The output of the LVDT was fed to the X-axis of the X-Y plotter used with the load cell so that a continuous plot of the load versus vertical movement was recorded.

3.1.2. 4 × 4 Specimen Test Procedure

The testing procedure was rather simple for the short-term static tests. The tensile pull-out load was applied to the insert at a uniform rate, until failure occurred. The maximum load attained during the tests was called the pull-out load of the insert in a 4 × 4 specimen (P_4). During preliminary tests [4], there were indications that the maximum load was a function of the rate of loading. For that reason, a standard loading rate of 2-kips per min was established for the static tests.

3.1.3. Continuous Span 4 × 22 Slab Specimen Test Procedure

The continuous 4 × 22 slabs were tested in the manner indicated by figures 5 and 8. Only partially visible in figure 5 is an air-bag system underneath the slab. Prior to the pull-out tests, the air pressure in the bag was adjusted to provide a uniform distributed load of 90 psf. This load is equal to a roof load of 30 psf plus 60 psf which is half the design load for inserts spaced at 5 ft on centers.

It should be noted that not all the 19 inserts shown in figures 5 and 8 were pulled. Generally, every other insert was pulled, until failure, and then the balance were pulled out. However, many times, the first pull-out damaged the specimen in such a manner that the neighboring inserts might have been affected. When this happened, the neighboring inserts were not pulled.

Companion 4 × 4 specimens were tested at the same age as the 4 × 22 specimens using the regular static test procedure.

3.1.4. Simple Span 4 × 22 Specimen Test Procedure

Three 4 × 22 specimens were tested with a simple span of 20 ft in a manner similar to that used for the continuous slabs. Due to the smaller load capacity no simulated live load was applied by an air bag system. Each of these slabs contained only five inserts. The two inserts near the end were tested as if they were in 4 × 4 specimens after the midspan inserts had been pulled.

3.1.5. Waffle Slab Specimen Test Procedure

Four Type 3 inserts were embedded in the waffle slab concrete at the four interior intersections of the 6-in ribs, as shown in figure 7. This test was to determine if the pull-out strength of inserts embedded in a waffle slab was affected by the relatively thin section of concrete around the insert. It should be noted that standard practice usually results in the placement of two reinforcement bars in both directions at the bottom of the ribs. This practice, which was followed for this test, results in the inserts being positioned so that the bars are close to the insert on the four sides. The inserts were tested using the 42-in test stand placed on the transverse ribs.

3.2. Sustained Load Tests

3.2.1. Sustained Load Test Equipment

The sustained load equipment was designed to hold a constant tensile load on inserts embedded in 4 × 4 specimens. Figure 9 is a picture of some of the specimens under sustained load. The sustained loading equipment was the same as the static load equipment, except that a 15-kip spring was used in place of the hydraulic ram. The spring, which was used to provide the required sustained load, was compressed by using a 60-kip ram. The load developed by the compressed spring was measured with the load cell and adjusted periodically. Movement of the pull-out relative to the transverse edges of each specimen was measured with 0.001-in dial gauges, which are visible in figure 9.

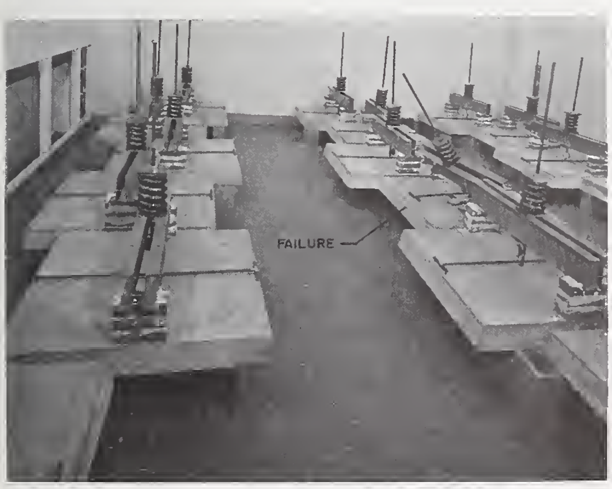

FIGURE 9. *Sustained load tests on 4 × 4 specimen.*

3.2.2. Sustained Load Test Procedure

Four batches of concretes, designed as C-1, C-2, C-3, and C-4, were used in preparing the 4 × 4 specimens used in the sustained load tests. C-1 and C-3 specimens were normal weight concrete, and C-2 and C-4 were semi-lightweight concrete. The purpose of the tests was to determine the maximum load which can be carried by an insert for an indefinitely long period of time. Two types of sustained load tests were attempted. The first type of test consisted of slowly increasing the load at a constant rate, until failure occurred.[4] The rate of loading was varied from 0.045 kips/h to 2.0 kips/min so as to get a relationship between failure load and rate of loading.

The second, and the more conventional type of sustained load test, consisted of maintaining a predetermined pull-out load on the insert. The load was maintained using the springs visible in figure 9. The magnitude of the load to be sustained was determined after ordinary short-term static pull-out tests were performed on companion 4 × 4 specimens. The sustained loads applied were 80, 85, and 90 percent of the short term pull-out loads. The movement of the pull-out rod relative to the edge of the specimen, which included some deflection of the slab, was measured at intervals of time, so that creep movement vs time could be plotted.

3.3. Fatigue Loading Tests

3.3.1. General

Two series of fatigue tests were made on inserts embedded in $4\frac{1}{2}$-in thick slabs. The first series of tests were on inserts embedded in 4 × 4 slabs, while the second series were on inserts embedded in 4 × 16 slabs.

One of the most important variables in fatigue tests is the ratio of the cyclic portion of the test load. This variable is usually expressed as the ratio (R) of the minimum to the maximum load. Murdock and Kesler [6] have shown that, for plain concrete in flexure, the fatigue strength (10 million cycles) is about 55 percent of the static short-term strength, when $R = 0.0$. When $R = 0.3$, the fatigue strength is about 65 percent, and when $R = 0.6$, the fatigue strength is about 80 percent. In this study R was varied from 0.3 to 0.43.

3.3.2. Fatigue Loading Test Equipment

Figure 10 is a general view of the fatigue tests on two 4 × 16 specimens. Alternating tensile loads of the required magnitude were applied by 10-kip servo-controlled hydraulic rams, reacting against a steel frame bolted to the laboratory tie-down floor.

[4] This loading scheme is a modification of the "Prot Method" [5] sometimes used in cyclic fatigue testing.

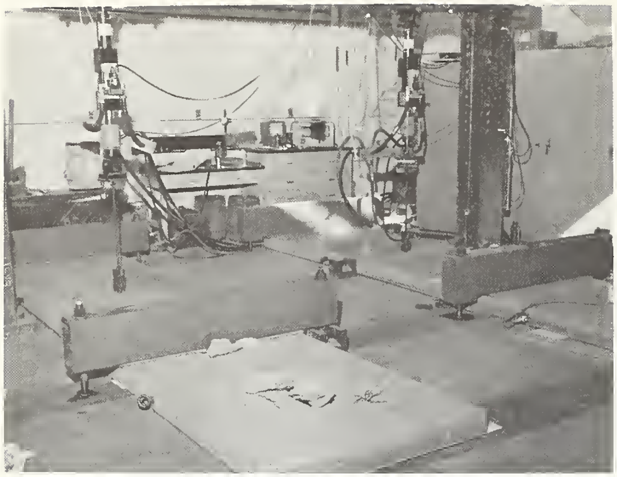

FIGURE 10. *Fatigue tests on 4 × 16 specimen.*

The test specimens were held to the tie-down floor at their reaction points. Both 4 × 16 and 4 × 4 specimens were tested under fatigue loading. The 4 × 16 specimen pictured in figure 10 was tested with a span of 10 ft. The test span for the fatigue tests on the 4 × 4 specimen was 42 in. Two fatigue tests were made on each 4 × 16 specimen, but only one on each 4 × 4 specimen.

3.3.3. Fatigue Tests on 4 × 4 Specimens

Twleve 4 × 4 fatigue test specimens were cast from each of the F-1, F-2, F-3, and F-4 concretes. Type 1 inserts were cast in the F-1, F-2, and F-3 specimens. Four inserts of each of the three types were cast in the F-4 slabs. The strength of the F-1 specimens was so great that the fatigue tests could not be made with the available equipment. The 4 × 4 specimens were fatigue loaded on a span of 42 in, with a minimum load ($P_{\min}$) of 3.0 kips for all specimens. The maximum load ($P_{\max}$) was varied so that $P_{\min}/P_{\max}$ (R) varied from 0.30 to about 0.43.

3.3.4. Fatigue Tests on 4 × 16 Specimens

Longer fatigue test specimens were cast from the X2-A, X2-B, X2-C, and X2-D concretes. Four specimens containing Type 3 inserts were cast from each concrete. The inserts were spaced so that each specimen had two inserts available for fatigue tests on a 10 ft span. Thus a total of eight fatigue tests were made on specimens from each concrete. These fatigue specimens also contained inserts at either end for determination of the static strength in a 4 × 4 specimen (figure 10). The maximum and minimum load values were varied, but their ratio (R) was held constant at 0.63 for these tests. It should be remembered that this ratio is less severe than that used for the 4 × 4 slabs of section 3.3.3.

4. Discussion of Results

4.1. General

The variables studied in this project are summarized here and in the flow chart of figure 11. Six

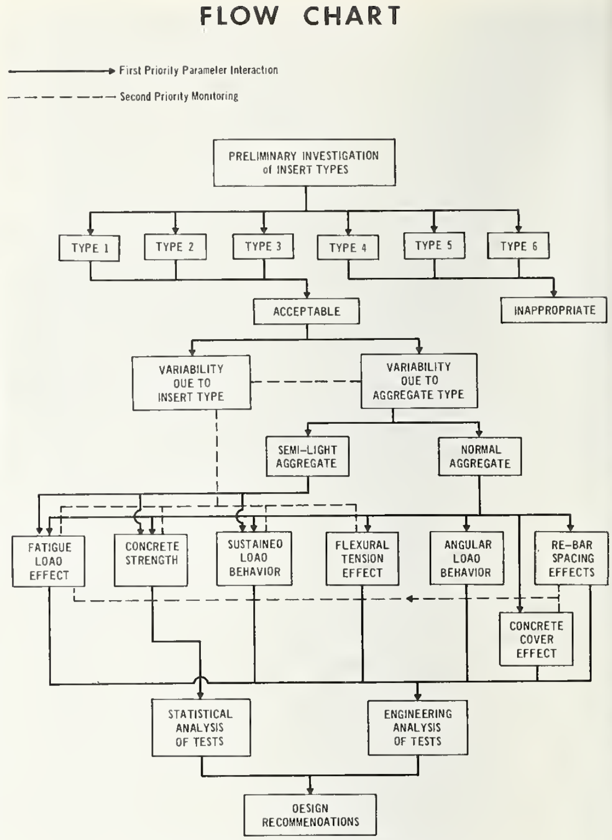

FIGURE 11. *Project flow chart.*

insert types were tested in a preliminary investigation [4]. Three of these were eliminated from this study because they did not meet a Post Office specification requiring malleable iron inserts suitable for attaching $\frac{3}{4}$-in-diameter threaded rods.

The three acceptable inserts were tested in 4 × 4 panels to determine if insert type or aggregate type was more critical. These tests are described in section 4.2. Based on these tests it was decided that aggregate type was more significant than insert type for the three inserts investigated. Subsequent tests were carried out with aggregate type as the major variable.

4.2. Effect of Aggregate and Insert Type

Table 5 lists the results of tests performed on 4 × 4 specimens during the early stages of the investigation. In order to minimize project complexity, these initial results were used to decide whether to emphasize aggregate-type or insert-type as the more significant parameter.

Since concrete L-4 was the only fully lightweight concrete (lightweight fines and lightweight coarse), it was deleted from the statistical observations used to compare normal weight concrete and semi-lightweight concrete (lightweight coarse only). Note that the results of tests on Type L-4 were significantly lower than the tests on semi-lightweight concrete of comparable strength. This suggests that

TABLE 5. *Tests for aggregate and insert variation*

Test set no.	Concrete compressive strength f_c'	Average pull-out load*		
		Type 1	Type 2	Type 3
	psi	*kips*	*kips*	*kips*
H-1	3480	12.0	10.0	12.0
H-2	3150	15.8	12.5	14.6
H-1A	3950	16.2	14.9	17.4
H-2A	4110	14.5	13.5	13.1
L-1	2640	11.6	10.2	11.5
L-2	3040	9.9	8.0	8.9
L-3	5420	11.5	10.9	10.3
L-4	3300	9.3	7.2	7.3
L-5	3370	11.5	9.2	10.6
L-1A	5050	15.8	15.3	14.1
L-2A	5200	13.4	11.8	11.5

* Average of 4 Insert Pull-Out Tests.

data for semi-lightweight concrete should not be directly extrapolated to fully lightweight concrete without additional testing. Table 6 summarizes some elementary statistics computed to show the relationship between the insert type and the concrete type. Referring to these statistics, it was decided that a two-part partitioning of the test distribution into normal and semi-lightweight concrete would be the most meaningful (as indicated in figure 11, section 4.1). Insert Type 3 was selected as the reference insert to which the other two inserts could be compared.

Typical specimen failures are illustrated in figures 4 and 12. In general, the base of the pull-out cone was about the same area for the Type 1 as for the Type 3 insert, but smaller in area for the Type 2. Type 1 and 3 inserts did not appear to be damaged by the tests. However, in about 70 percent of the tests, the wire loop of the Type 2 insert failed near the point where it was welded to the spool. Even when the loop did not fracture, the loop wires were visibly necked down.

TABLE 6. *Elementary statistics for concrete and insert type*

Insert type	Concrete type	Pull-out load		No. samples
		Sample mean	Range	
		kips	*kips*	
1	Normal	14.6	7.3	16
2	Normal	12.7	6.8	16
3	Normal	14.3	8.3	16
1	Semi-light	12.3	7.9	24
2	Semi-light	10.9	7.6	24
3	Semi-light	11.2	6.8	24
All	Normal	14.0	8.3	48
All	Semi-light	11.5	9.3	72
1	All	13.2	8.1	40
2	All	11.6	8.9	40
3	All	12.4	9.9	40

4.3. Concrete Strength-Weight Effects

Over 200 static tests on 4 × 4 specimens were about equally divided into normal weight and semi-lightweight concrete specimens. Within each concrete type the variables were concrete strength and insert type. The failure mode shown in figure 12 is primarily a concrete tensile failure. Since concrete tensile strength is approximately proportional to the square root of the concrete compressive strength, the pull-out loads were examined as a linear function of the square root of the concrete compressive strength for the normal weight and semi-lightweight specimens.

FIGURE 12. *Typical pull-out cone failure.*

The data for normal weight concrete specimens are shown in figure 13 along with two straight lines. The solid line is a linear least-squares fit [7, 8] to the data. The dashed line is proportional to the least-squares line and is intended to account for experimental scatter and other variables not accounted for by the least-squares equation. The least squares equation for the normal weight concrete is given by:

$$P_{4h} = 1.63 + 0.18\sqrt{f_c'} \qquad 4.3(1)$$

where P_{4h} is the insert pull-out strength in normal weight 4 × 4 specimens (kips).

It is evident in figure 13 that the least-squares line is not an ideal fit to the data. However, it is the simplest prediction equation for these data since there is no failure hypothesis available that considers all of the parameters affecting insert pull-out strength.

Scatter[5] was investigated by assuming that the pull-out load is a linear function only of the square root of the concrete compressive strength. Thus the effect of variable concrete strength can be reduced by using this relationship to normalize all pull-out loads to a reference concrete strength. Figure 14 shows the distribution of all normal weight concrete 4 × 4 tests. The solid line histogram is for actual pull-out loads. The dashed line histogram is for

[5] Scatter as used here includes all variations other than those due to concrete tensile strength.

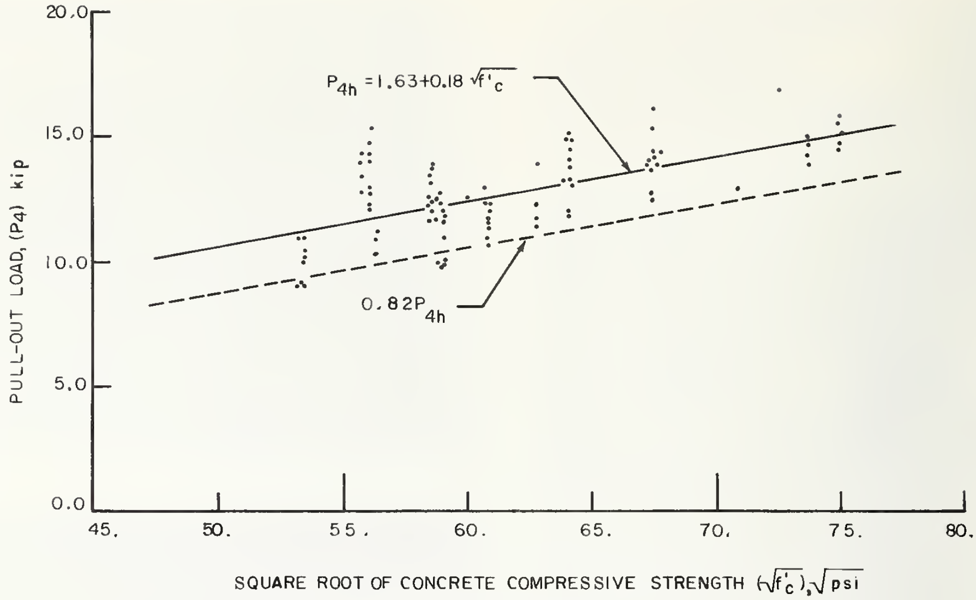

Figure 13. *Variation of pull-out load with the square root of concrete strength for normal weight concrete.*

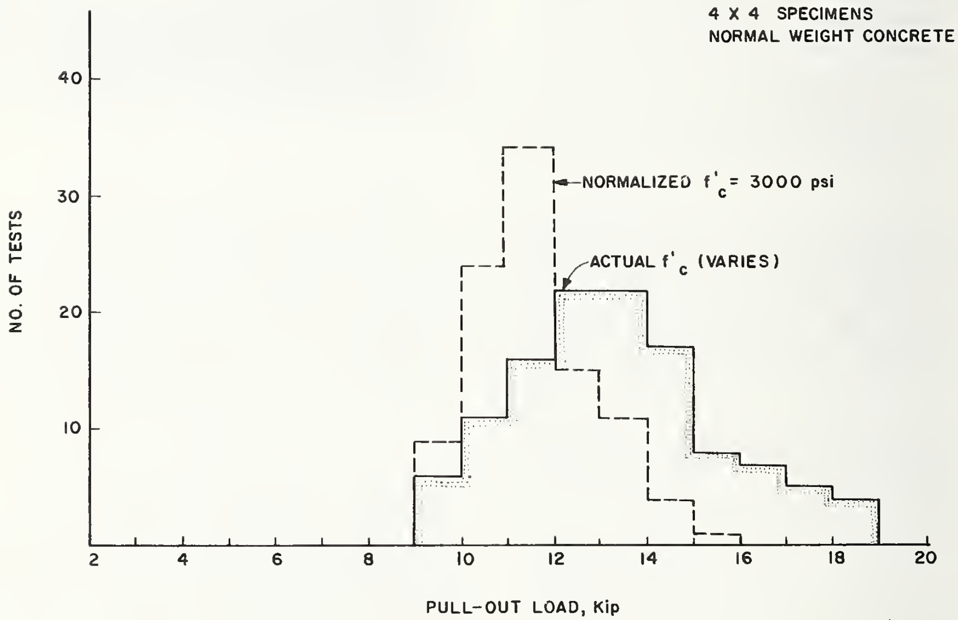

Figure 14. *Histogram of normal weight concrete pull-out loads in 4 × 4 specimens.*

normalized pull-out loads based on a reference concrete compressive strength of 3000 psi. The normalized load, P_n, is equal to the actual pull-out load, P, times the ratio of the square root of the reference concrete strength to the square root of the actual concrete strength

$$P_n = P\sqrt{f_c' = 3000 \text{ psi}}/\sqrt{\text{actual } f_c'}.$$

Referring to the normalized histogram, the average pull-out load is 11.5 kips with a standard deviation of 1.2. Using these data and assuming a normal distribution of pull-out loads, a pull-out load can be determined which most (say 95 percent) of the test results exceed. Thus it was determined that 95 percent of the pull-out loads exceed 9.4 kips which is 82 percent of the average pull-out load. The dashed line shown in figure 13 is the least-squares equation line multiplied by 82 percent. Note that most of the data are above this line. Thus the reduced least-squares line is a conservative approximation of the pull-out load and at the same time it makes allowance for increased pull-out load with increased concrete strength.

The semi-lightweight concrete data were handled in a similar manner. The data are shown in figure 15 along with the least-square fit and the reduced line to account for scatter. The least-squares line shown in figure 15 is given by:

$$P_{4s} = 3.06 + 0.13\sqrt{f_c'} \qquad 4.3(2)$$

where P_{4s} is the insert pull-out strength in semi-lightweight 4 × 4 concrete specimens.

The histograms for actual and normalized pull-out loads are shown in figure 16. Referring to the normalized histogram, the average pull-out load is 10.2 kips with a standard deviation of 1.5. Using these data it was determined that 95 percent of the test results exceed 7.7 kips which is 75 percent of the average pull-out load. This indicates that the semi-lightweight specimens exhibit slightly more scatter

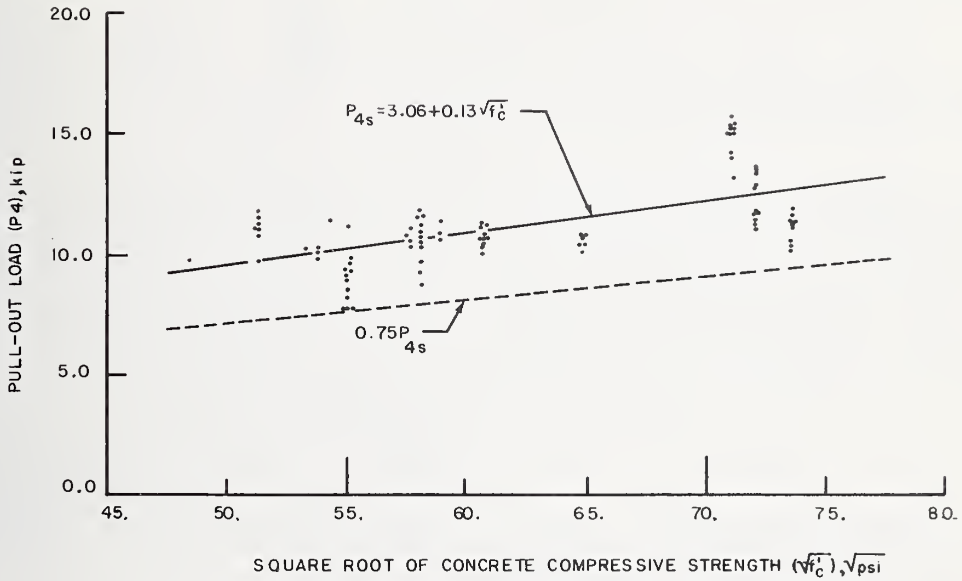

FIGURE 15. *Variation of pull-out load with the square root of concrete strength for semi-lightweight concrete.*

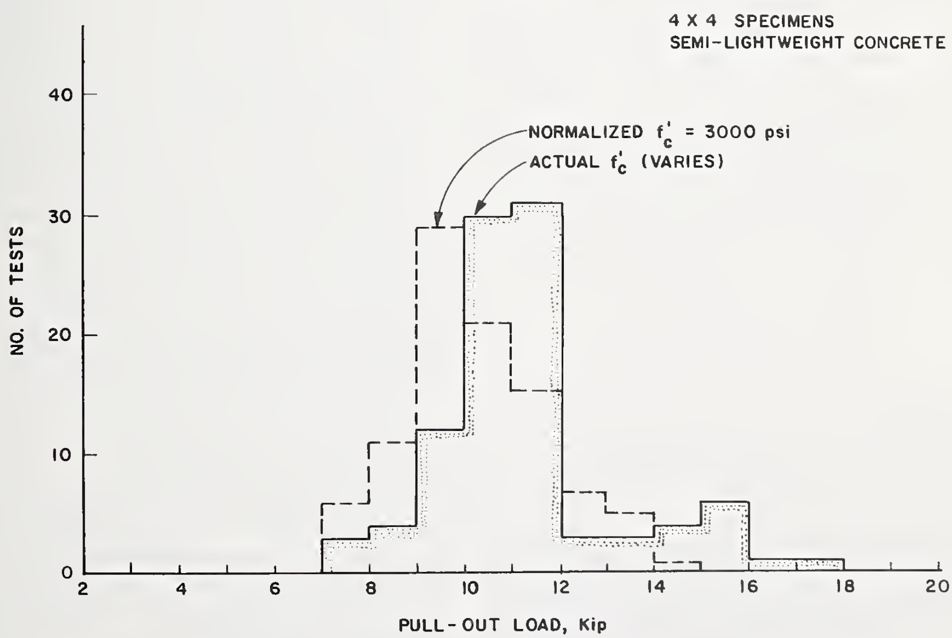

FIGURE 16. *Histogram of semi-lightweight concrete pull-out loads in 4 × 4 specimens.*

TABLE 7. *Comparison of empirical and test results—4 × 4 specimens*

	Nominal strength of concrete	Pull-out load (P_{4u}) by concrete type		
		Normal-weight (W_c = 142 lb/ft³)	Semi-light-weight (W_c = 118 lb/ft³)	Light-weight (W_c = 96 lb/ft³)
	psi	*kip*	*kip*	*kip*
Empirical*	3000	11.3	9.8	8.3
Test**	3000	11.5	10.2	8.1
Empirical*	5000	14.1	12.0	10.2
Test**	5000	14.3	12.3	....

* Empirical $P_{4u} = 2.0 + .0012\,W_c\sqrt{f_c'}$ (kips).
** Test results are the average of all samples within the range of the nominal concrete strength.

than the normal weight specimens. Note that most of the data lie above the reduced least-squares line.

In comparing figures 13 and 15 the relative effects of concrete unit weight may be introduced. The average unit weight of the normal weight concretes (W_h) was 142 lb/ft³. The average unit weight of the semi-lightweight concretes (W_s) was 118 lb/ft³. Thus the ratio of unit weights (W_h/W_s) is 1.20. For 3,300 psi concrete, the calculated regression equations give:

$$P_{4h} = 1.63 + 0.18\sqrt{3{,}300} = 12.0 \text{ kips}, \qquad 4.3(3)$$

$$P_{4s} = 3.06 + 0.13\sqrt{3{,}300} = 10.6 \text{ kips}. \qquad 4.3(4)$$

Since $P_{4h}/P_{4s} = 1.13$ is close to the unit weight ratio of 1.20, there may be a linear proportionality relationship between concrete unit weight and the insert pull-out strength. To investigate this possibility, consider the twelve tests on concrete L-4 (Table 5). For this lightweight concrete, the unit weight (W_l) was 96 lb/ft³, and the concrete strength was 3300 psi. If the unit weight ratio is a good approximation, then the average pull-out strength of these tests should be 8.3 kips.

The experimental average of the twelve tests was 7.9 kips. Although the results of twelve tests were insufficient to form a final conclusion for the lightweight, the available evidence points toward a generalized insert pull-out strength equation of the form:

$$P_{4u} = 2.0 + 0.0012 W_c\sqrt{f_c'} \qquad 4.3(5)$$

where:

P_{4u} = the insert pull-out strength in 4 × 4 specimen (kips),
W_c = the unit weight of concrete (lb/ft³),
f_c' = the compressive strength of concrete (psi).

Equation 4.3(5) was obtained by modifying the second term of Eqs. 4.3(1) and 4.3(2) to include the concrete unit weight. The constant 0.0012 was then obtained by averaging the constants from the second term of the two modified equations. The constant 2.0 was then selected somewhat arbitrarily so that the general equation does not produce pull-out loads larger than those obtained from equations 4.3(1) or 4.3(2). Loads obtained from the general equation range from two to four percent lower than those obtained from the equations 4.3(1) and 4.3(2).

Based on this empirical equation insert pull-out loads were calculated for 3,000 and 5,000 psi concrete. Table 7 compares these empirical results with experimental averages. As can be seen, the comparisons are well within acceptable limits. Since the equation is empirical it is limited to the range of the test data which is for concrete compressive strengths between about 3,000 to 5,000 psi. Scatter factors should be applied to the equation depending on the type of concrete.

4.4. Effect of Flexural Cracking

The pull-out strength of an insert is largely dependent on the tensile capacity of the embedding concrete. If this capacity is reduced by tensile cracking resulting from flexural action, then insert pull-out strength would decrease.

To study the effects of slab bending moment (flexural cracking) on insert strength, four long slabs (4 × 22 specimens) were constructed with inserts on 12-in centers. These slabs were supported at three reaction points on 10-ft centers to simulate the condition of a 4½-in thick slab continuous over two spans. Simulation of a uniform live load of 90 psf on three of the slabs (S-1, S-2, and S-3) was accomplished using air bags. Figure 5 shows the test setup. One additional continuous slab (specimen S-4) was tested with uniform live load of 150 psf.

Figures 17, 18, and 19 show the results of the insert pull-out tests performed on the three continuous slabs with 90-psf load. Slab S-1, shown by figure 17, contained Type 1 inserts. Slab S-2, shown by figure 18, contained Type 2 inserts, and Slab S-3 of figure 19 contained Type 3 inserts. Four control tests were performed on 4 × 4 specimens for slabs S-1, S-2, and S-3. The average of the control tests is represented on the respective figures by a horizontal line. Slab S-4, shown by figure 20, contained Type 1 inserts but was loaded with 150 psf

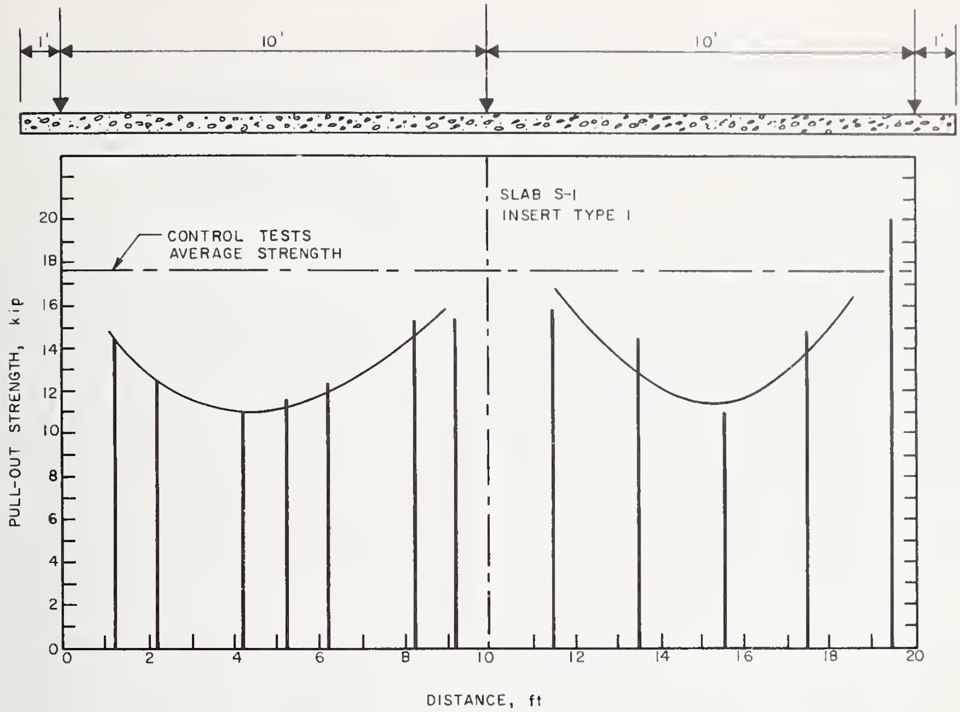

FIGURE 17. *Variation of pull-out strength with insert location in slab S-1.*

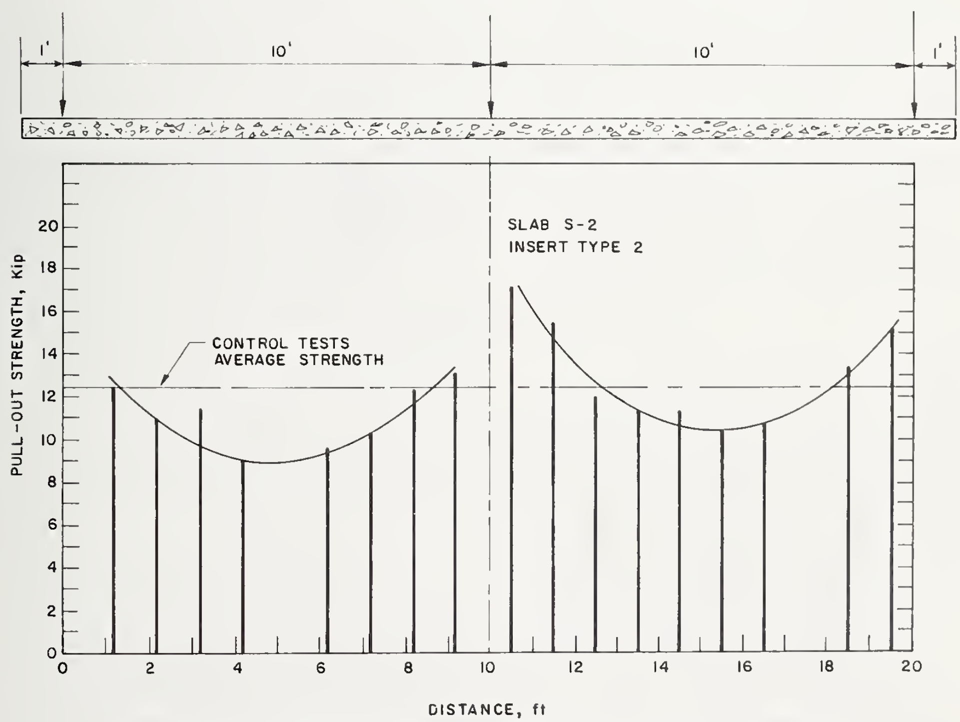

FIGURE 18. *Variation of pull-out strength with insert location in slab S-2.*

of simulated live load. No 4 × 4 specimen data are available for Slab S-4.

Each of figures 17, 18, 19, and 20 shows a positional effect on insert pull-out strength. This effect was investigated further in figure 21 by plotting normalized pull-out load versus insert distance from the nearest support. The pull-out loads were normalized to a common concrete strength of 3,000 psi for ease in comparing slab data (3,000 psi was the average compressive strength for the four slabs). The normalized load, P_n, is equal to the actual pull-out load, P, times the ratio of the square root of

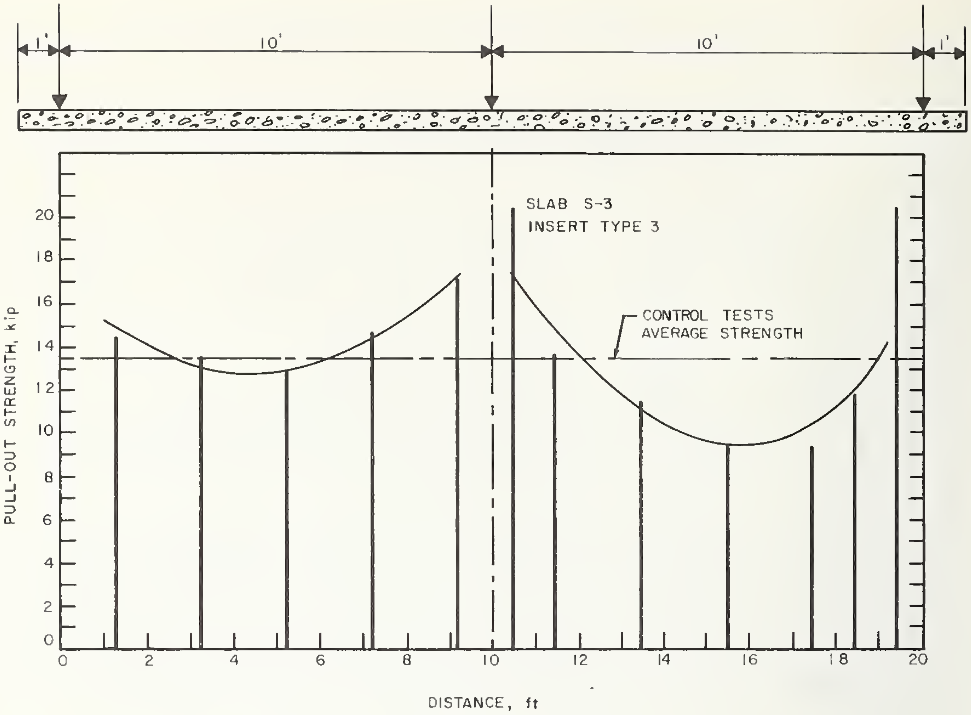

FIGURE 19. *Variation of pull-out strength with insert location in slab S-3.*

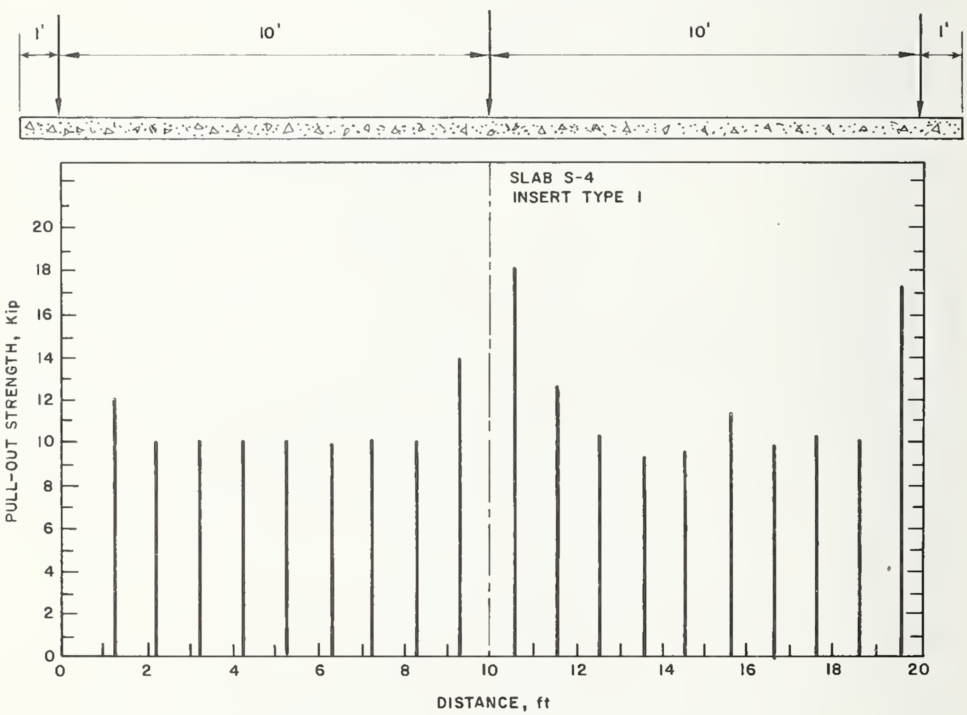

FIGURE 20. *Variation of pull-out strength with insert location in slab S-4.*

the reference concrete strength to the square root of the actual concrete strength

$$P_h = P\sqrt{f_c' = 3000 \text{ psi}}/\sqrt{\text{actual } f_c'}.$$

This is a reasonable procedure since pull-out load was shown to be a linear function of $\sqrt{f_c'}$ in section 4.3. This procedure assumes all insert types are equal although there are slight differences as indicated in section 4.2.

The large pull-out strengths for inserts close to

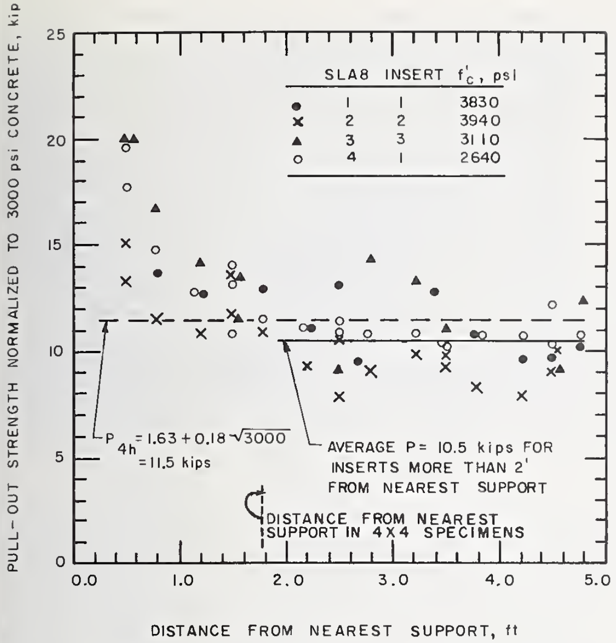

FIGURE 21. *Effect of distance from support on pull-out strength of inserts embedded in reinforced concrete slabs.*

the supports can be attributed largely to the containing influence of the support reactions. Since the insert pull-out cone varied from 12 in to 20 in diameter it is unlikely that the reactions affected inserts farther than about 2 ft from the supports. The data in figure 21 for inserts farther than 2 ft from the nearest support had an average pull-out load of 10.5 kips. This is shown in the figure by a solid line. Although the scatter is large (about 8 to 14.5 kips), there seems to be no definite variation in load with distances greater than 2 ft from the support. The average load of 10.5 kips compares to a computed load of 11.5 kips based on equation 4.3(1) for a 4 $\times$ 4 slab. This indicates that equation 4.3(1) may give results about 10 percent too large for slabs with spans greater than 4 ft. Bending moments cannot be computed exactly for slabs S-1 through S-4 due to some uncertainties in support conditions. Approximate calculations and test observations indicate that the tension steel probably did not yield.

Three 20 ft simply supported slabs (X-20A, X-20B, and X-20C) were tested in order to investigate the effect of reinforcement yield on the pull-out strength. Slab X-20A had an average pull-out strength of 7.1 kips (f_c' = 4090 psi), slab X-20B had an average pull-out strength of 6.8 kips (f_c' = 3,100 psi), and slab X-20C had an average pull-out strength of 6.4 kips (f_c' = 3,200 psi). The inserts slipped out of the concrete due to numerous large flexural cracks (steel had yielded) rather than pulling out a cone of concrete as shown in figure 12. These test results indicate that yielding of tension reinforcement should not be allowed since it permits the formation of large cracks which change the failure mode and reduce the pull-out strength by 30 to 40 percent. This is not a serious problem since usual design procedures prevent the main tension steel from yielding.

4.5. Effect of Reinforcement Cover and Spacing

4.5.1. Concrete Cover

Table 8 lists the results of tests performed to study the effects of reinforcement spacing and concrete cover. Test specimens were the 4 $\times$ 4 concrete slabs with Type 3 insert. Test No. X1B-1 through X1B-9 were performed as three sets of three tests, each set with a different amount of concrete cover ($\frac{3}{4}$ in, $1\frac{1}{2}$ in, and 3 in) over the reinforcing steel. Reinforcement for each set consisted of No. 5 bars at 12 in on centers, to simulate a condition of maximum reinforcement spacing. In addition, a fourth control set (X1B-10 through X1B-12) was included with No. 5 bars at 6 in on centers, to reference the maximum spacing condition with a more common design situation.

Within the test range ($\frac{3}{4}$-in to 3-in clear cover), the experimental results in figure 22 indicate a linear strength loss of about 1.4 kips per in of increased concrete cover over the reinforcing steel.

As indicated in figure 12 the typical pull-out mode of failure consists of an irregular cone of concrete pulled out with the insert. If the reinforcing steel intercepts this failure cone some of the insert load may be transferred to the reinforcing steel by dowel action. With increasing cover the reinforcing steel intercepts less of the failure cone. Hence the above loss in strength is probably due partly to the vertical location of the reinforcing steel with respect to the inserts. Some loss may also be due to the increased flexural cracking in the specimens with decreased moment capacity which occurs with increased cover.

4.5.2. Reinforcement Spacing

Referring again to table 8, tests No. X1A-1 through X1A-9 were designed to study the effects of reinforcement spacing. The tests were performed as three sets of three tests, each set with a different steel spacing ($3\frac{1}{2}$ in, 6 in, 12 in). The concrete cover and percent reinforcement were held approximately constant. From the test results shown in table 8, it is observed that up to the 12 in spacing the steel location does not significantly affect insert strength (average pull-out strengths of 14.7, 13.8, and 14.3 kips respectively).

4.5.3. Waffle Slab Ribs

The single waffle slab (W-1) test was made to determine if the pull-out strength of an insert would be affected by the relatively thin section of concrete around the insert. The average pull-out load for the four Type 3 inserts was 15.5 kips. From the equation 4.3(2) the expected pull-out load would be about 12 kips. These results indicate that the pull-out strength of inserts embedded at the intersection of 6-in ribs in similar waffle slabs

Table 8. *Effect of reinforcement cover and spacing*

Test no.	Reinforcement			Pull-out load	Average	Range
	Size	Spacing o.c.	Cover			
		in	*in*	*kips*	*kips*	*kips*
X1B-1	#5	12	$\frac{3}{4}$	10.3		
X1B-2	#5	12	$\frac{3}{4}$	9.9	10.1	0.4
X1B-3	#5	12	$\frac{3}{4}$	10.0		
X1B-4	#5	12	$1\frac{1}{2}$	9.0		
X1B-5	#5	12	$1\frac{1}{2}$	9.0	9.1	0.2
X1B-6	#5	12	$1\frac{1}{2}$	9.2		
X1B-7	#5	12	3	7.1		
X1B-8	#5	12	3	6.8	7.0	0.4
X1B-9	#5	12	3	7.0		
X1B-10	#5	6	$\frac{3}{4}$	10.4		
X1B-11	#5	6	$\frac{3}{4}$	10.9	10.7	0.5
X1B-12	#5	6	$\frac{3}{4}$	10.9		
X1A-1	#4	$3\frac{1}{2}$	$\frac{3}{4}$	13.8		
X1A-2	#4	$3\frac{1}{2}$	$\frac{3}{4}$	14.3	14.7	2.2
X1A-3	#4	$3\frac{1}{2}$	$\frac{3}{4}$	16.0		
X1A-4	#5	6	$\frac{3}{4}$	13.6		
X1A-5	#5	6	$\frac{3}{4}$	13.7	13.8	0.4
X1A-6	#5	6	$\frac{3}{4}$	14.0		
X1A-7	2-#5	12	$\frac{3}{4}$	13.5		
X1A-8	2-#5	12	$\frac{3}{4}$	13.9	14.3	1.9
X1A-9	2-#5	12	$\frac{3}{4}$	15.4		

Notes: 1. Concrete Type X1A was normal weight with $f_c' = 4560$ psi.
2. Concrete Type X1B was normal weight with $f_c' = 2990$ psi.
3. Insert Type 3 was used in all slabs.

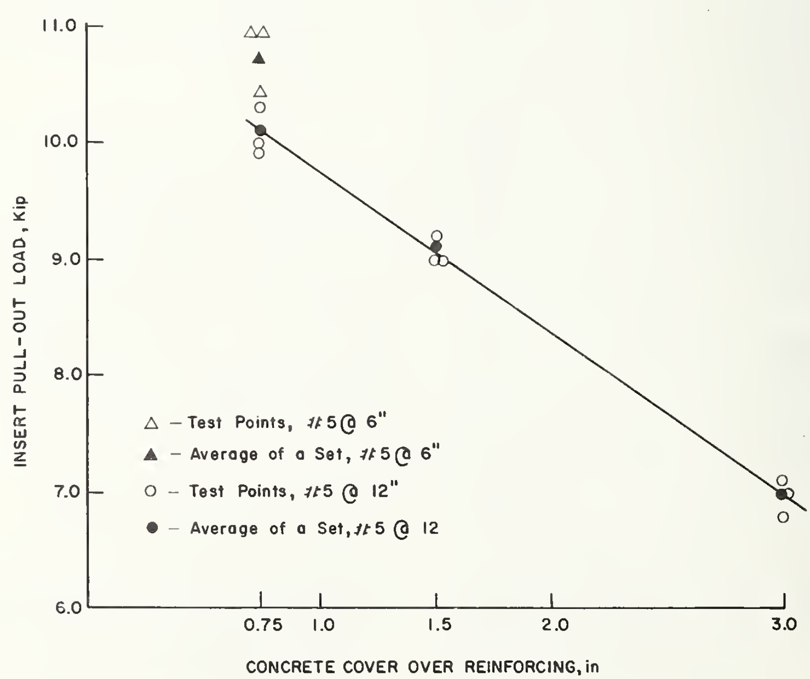

Figure 22. *The effect of concrete cover on pull-out load.*

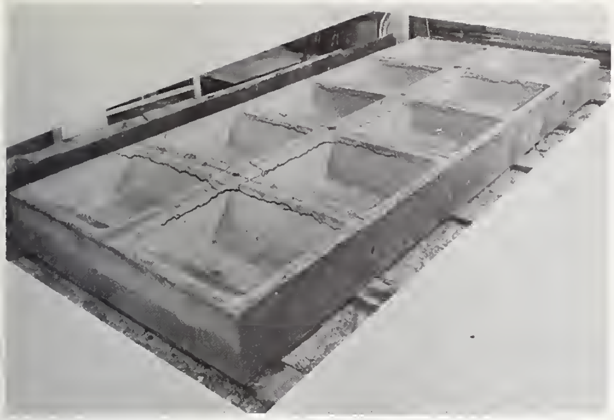

FIGURE 23. *Waffle slab after testing.*

FIGURE 24. *Sustained load failure on 4 × 4 specimen.*

would be as good or better than that for those embedded in the 4 × 4 specimens. Figure 23 is a photograph of the waffle slab after testing. The crack pattern, accentuated by felt pen markings, is easily visible in this figure and shows how the cracks tended to extend along the line just below the reinforcement. There is no doubt that the inserts were restrained by the reinforcement, as evidenced by the crack patterns in the ribs.

4.6. Sustained Load Behavior

Forty-eight 4 × 4 specimens were tested to study the effect of sustained load on insert pull-out strength. Twelve normal weight concrete specimens were cast from each of batches C-1 and C-3. Another twelve semi-lightweight concrete specimens were cast from each of batches C-2 and C-4.

Specimen groups C-1 and C-2 were tested by the modified Prot method described in section 3.2.2. Although inconclusive, the data indicated that the sustained pull-out strength would reach a minimum of about 90 percent of the short term strength, at a load rate of about 0.1 kip/h. At slower loading rates the data indicate that the sustained strength may become greater than 90 percent. This unexpected result requires further study for verification.

Specimen groups C-3 and C-4 were tested by the more conventional procedure also described in section 3.2. For each type of concrete, the twelve tests were subdivided into four groups of three tests each. The first group was tested to determine the static load capacity. Each of the remaining three groups was tested at different sustained load levels. These load levels were 80, 85, and 90 percent of the insert pull-strength obtained by performing the static load capacity tests on the companion 4 × 4 specimens. A typical sustained load failure is shown in figure 24.

The results of the sustained load tests are plotted on figure 25, as Deformation vs Time curves. Each of the six curves represents the average of the three

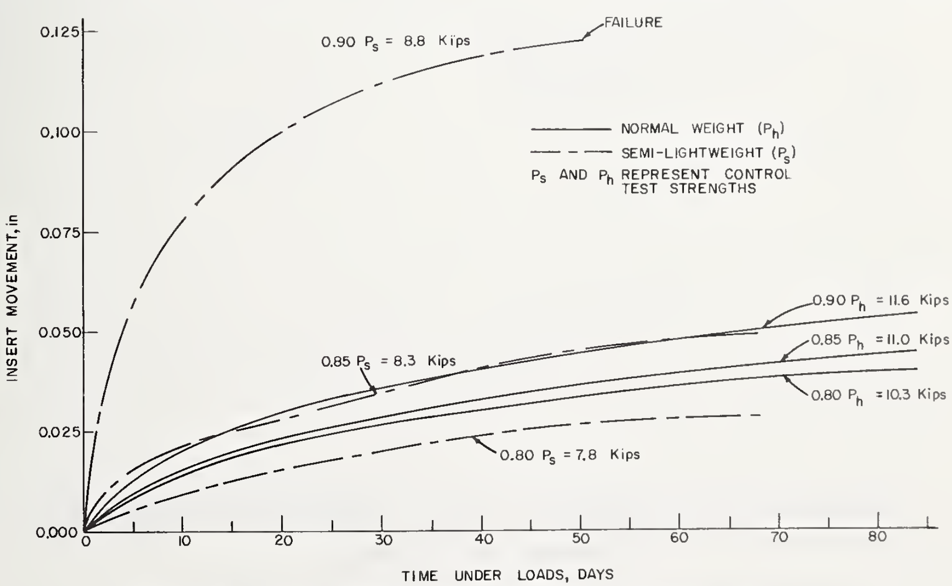

FIGURE 25. *Insert movement variation with time in sustained load tests.*

tests performed at the indicated load level. The dashed curves represent semi-lightweight concrete, while the solid lines apply to normal weight concrete. As shown on the figure, the only failure occurred for semi-lightweight concrete loaded to 90 percent of the equivalent static strength. For all other load levels, the deformations are relatively small, stable, and tending toward an asymptotic relationship with respect to some upperbound deformation. These other specimens were observed for a period of one year without any indication of failure.

4.7. Fatigue Load Behavior

4.7.1. General

Approximately 50 fatigue tests were performed to study the effects of fatigue loading on insert pull-out strength. Of these tests, 25 were performed on Type 3 inserts embedded in the 4 × 16 specimens and tested with a beam span of 10 ft. The remaining 25 tests were performed on the three types of inserts embedded in 4 × 4 specimens. Each type of fatigue test slab had 4 × 4 static control test specimens so that fatigue loads could be expressed as a ratio of the static strength (P_{max}/P_4).

4.7.2. Normal Weight Concrete

Figure 26 is a semi-log plot of P_{max}/P_4 vs number of cycles causing fatigue failure. These tests were performed on Type 3 inserts embedded in 4 × 16 slabs made from two batches of normal weight concrete. The ratio of the cyclic load ($R = P_{min}/P_{max}$) was held constant at 0.63. The averages of the 4 × 4 static control tests for the two batches were 11.6 kips and 13.7 kips, which are close to the mean load of all static tests at similar concrete strengths.

The fatigue test results shown in figure 26 indicate that the fatigue strength at 2 million cycles is about 65 percent of the static strength for the 4 × 4 control specimens. In short term static tests on companion specimens with a 10 ft span the insert strength was 87 percent of the strength for the 4 × 4 control specimens. This strength loss is in close agreement with the effects of flexural cracking (a strength loss of about 10 percent) described in section 4.4. This indicates that about one-third of the apparent strength loss in these fatigue tests was due to the increased span and two thirds was due to fatigue.

In general the fatigue test slabs were reinforced by No. 5 bars at 6 in on centers. To check the effect of reinforcement spacing on fatigue strength, one test slab from each batch was reinforced with No. 5 bars at 12 in on centers. As indicated on figure 26, the reduction in steel did not significantly reduce the insert fatigue strength.

4.7.3. Semi-Lightweight Concrete

The results of fatigue tests with 4 × 16 semi-lightweight concrete specimens are plotted in figure 27. These tests were similar to those of figure 26, except that the test specimens were made from two batches of semi-lightweight concrete. Type 3 inserts were embedded in the slabs.

The average pull-out strength of the 4 × 4 static control tests was 10.8 kips, which is close to the mean load of all 4 × 4 semi-lightweight tests. The fatigue test results shown in figure 27 indicate that for 2 million cycles, the fatigue strength of an insert is

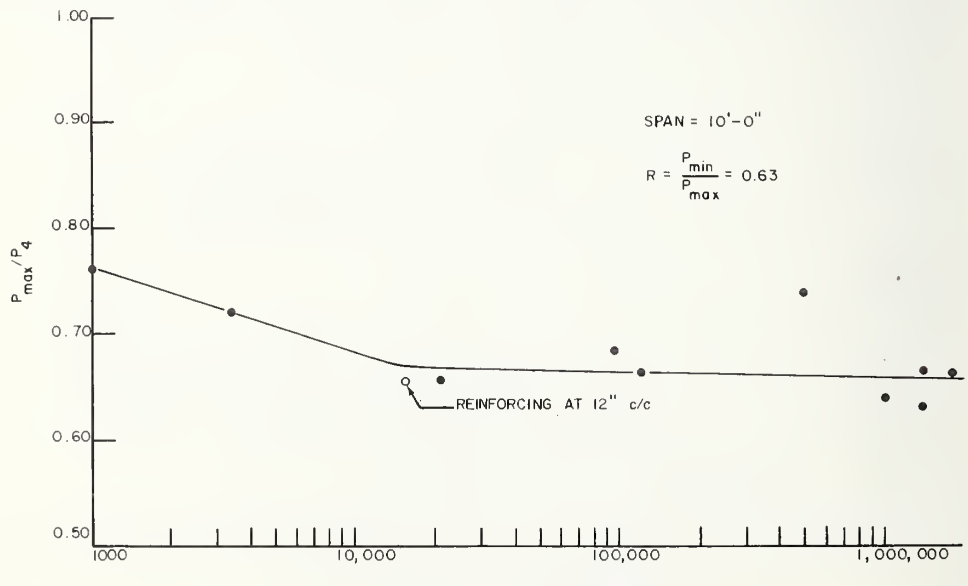

FIGURE 26. *Effect of fatigue on pull-out loads in 10 ft spans of normal weight concrete specimens.*

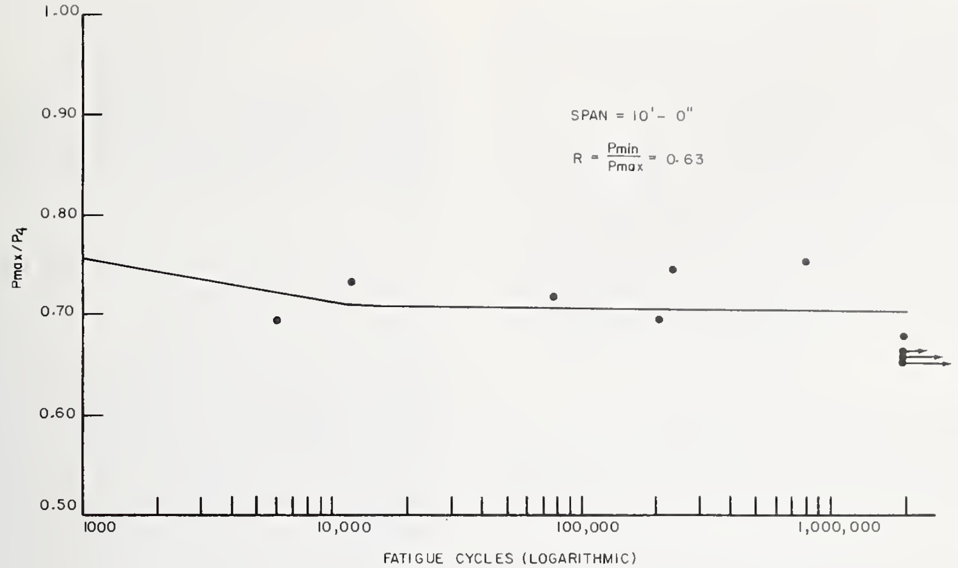

FIGURE 27. *Effect of fatigue on pull-out loads in 10 ft spans of semi-lightweight concrete specimens.*

about 70 percent of the static strength for the 4 × 4 control specimens.

In static tests on companion specimens with a 10 ft span the insert strength was 89 percent of the strength for the 4 × 4 control specimens. This strength loss is in close agreement with the effects of flexural cracking (a strength loss of about 10 percent) described in section 4.4 although those tests were on normal weight concrete. Again this indicates that about one third of the strength loss in these fatigue tests was due to the increased span and two thirds was due to fatigue.

4.7.4. Variation of Insert Type and Ratio, *R*

Figure 28 shows the results of 25 fatigue tests performed on inserts embedded in 4 × 4 specimens. These tests were used to investigate the effect of variation of insert type and variation of the ratio, *R*, on fatigue strength. However, there are insufficient data shown in figure 28 to permit reaching any positive conclusions.

4.7.5. Fatigue Failures in Connecting Hardware

An important adjunct to the fatigue testing on the inserts was the discovery that the fatigue limit

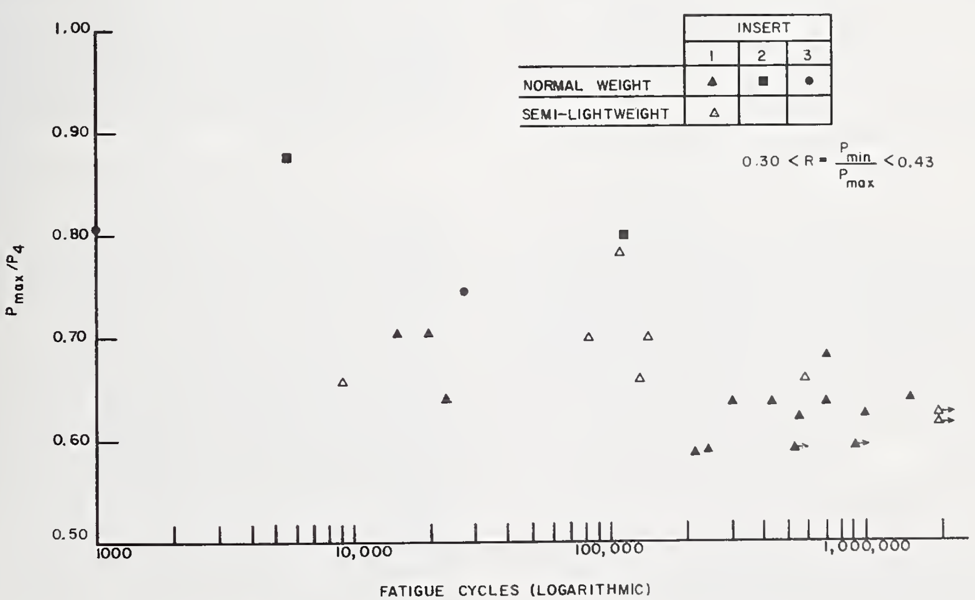

FIGURE 28. *Effect of fatigue on pull-out loads in 4 × 4 specimens.*

for some of the connecting hardware used in the tests was close to that of the actual specimens being tested. No record was kept of the individual bolts and clevis pins so that the full history of each is not known. However, the mortality rate for this hardware was around 50 percent of that for the insert specimens. This is not surprising when fatigue data, such as has been presented in [9], is considered. These data indicate that the fatigue strength of ordinary steel bolts can be as low as 20,000 psi for 10 million cycles. This means that for a safety factor of 2.0, the allowable load under cyclic loading conditions on a $\frac{3}{4}$ inch threaded steel bolt could be as low as 3.0 kips.

4.8. Angular Load Effect

Referring to table 9, Tests No. X4A-1 through X4A-12 were designed to investigate the effects of intentional or unintentional misalinement of an insert or its connecting hardware. Tests X4A-1 through X4A-4 were performed on well-alined inserts and loads, for reference purposes. Tests X4A-5 through X4A-8 were performed to determine if an angled load decreased the insert pull-out strength. The results show no reduction in strength for a load 20 degrees out of alinement with a plane perpendicular to the plane of the slab.

Tests X4A-9 through X4A-11 were performed to establish if an angled insert decreased the pull-out strength. The results show no reduction in strength. Slab X4A-12 was tested with the insert and load both angled (but in line with each other). Again, there was no reduction in strength.

TABLE 9. *Effect of Angular Pull on Insert Pull-Out Load**

Test no.	Load angle**	Insert angle**	Pull-out load	
			Individual	Average
	degrees	*degrees*	*kips*	*kips*
X4A-1	0	0	11.8	
-2	0	0	12.0	11.8
-3	0	0	11.7	
-4	0	0	11.7	
-5	20	0	12.4	
-6	20	0	12.2	12.4
-7	20	0	12.6	
-8	20	0	12.4	
-9	0	20	13.1	
-10	0	20	13.7	13.4
-11	0	20	13.2	
-12	20	20	13.8	13.8

* Concrete Strength, $f_c' = 3440$ psi.
** Measured from a plane perpendicular to the plane of the slab.

5. Summary and Conclusions

5.1. General

This investigation was limited to cast-in-place inserts with $3\frac{1}{8}$- to $3\frac{3}{4}$-in embedment lengths in $4\frac{1}{2}$-in thick reinforced concrete slabs. Reinforcement steel was No. 5 bars at 6 in on centers with $\frac{3}{4}$-in clear cover unless stated otherwise.

5.2. Aggregate and Insert Type

All three types of inserts can be considered similar for purposes of design load recommendations that encompass flexure, fatigue, and sustained load effects. Although the inserts behave similarly they are not equivalent. Types 1 and 3 were generally very close in performance, but with Type 2 testing about one to two kips lower for similar loading conditions. Nevertheless the inserts have been considered equal with differences being accounted for in section 5.9 on experimental scatter.

The pull-out strength of the inserts was lower in specimens made with semi-lightweight concretes than in specimens made with the normal weight concretes. This difference was slightly less for fatigue loading than it was for static loading.

5.3. Concrete Strength

From section 4.3, it is concluded that an increase in concrete compressive strength causes a predictable increase in the average pull-out strength. For a concrete strength between 3,000 and 5,000 psi, and a concrete unit weight of 115 to 145 lb/ft^3, the average static pull-out strength of the inserts in a reinforced concrete slab 4 × 4 specimen can be approximated by:

$$P_{4u} = 2.0 + 0.0012\, W_c \sqrt{f_c'} \qquad 5.3(1)$$

where:
P_{4u} = 4 × 4 specimen pullout strength (kips),
W_c = unit weight of concrete (lb/ft^3),
f_c' = concrete compressive strength (psi).

5.4. Concrete Flexural Cracking

Tests on slabs continuous over two 10-ft spans indicate that insert pull-out strengths based on simply supported 4 × 4 specimens may be 10 percent too high when the inserts are used in longer spans where more flexural cracking occurs. Tests on 20-ft simply supported slabs indicate that yielding of flexural reinforcement changes the insert mode of failure. That is, the inserts slip out of the concrete due to large flexural cracks rather than pulling out a cone of concrete which is the typical mode of failure. This is not a serious problem since usual design procedures prevent the main tension steel from yielding. Insert pull-out loads obtained from 4 × 4 specimens should be multiplied by a reduction factor $\phi_t = 0.90$ to account for the loss of insert strength due to flexural cracking.

5.5. Reinforcement Quantity and Cover

From the discussion of section 4.5.1, it is concluded that for concrete covers greater than $\frac{3}{4}$ in the insert capacity in a 4 × 4 specimen made with a 3,000 psi normal weight concrete would be reduced by 1.4 kips per in of cover for up to 3 in of cover. These tests did not include cover less than $\frac{3}{4}$ in or greater than 3 in.

The reinforcement spacing study of section 4.5.2 shows that the loss of insert strength with No. 5 bars spaced up to 12 in, is relatively small and may be neglected.

5.6. Sustained Load

It was shown in section 4.6 and figure 25 that a sustained insert load 90 percent of the static pull-out load can result in a failure in semi-lightweight concrete slabs. Thus, if sustained load is to be considered as a prime parameter, a reasonable reduction factor (ϕ_s) is 0.85. Since it is physically impossible to develop maximum sustained load and maximum fatigue load simultaneously, the maximum effects of sustained load and fatigue load are not cumulative. Since sustained load is not as detrimental to the insert behavior as is fatigue load, it may be unnecessary to consider this parameter, provided fatigue loading has been considered.

5.7. Fatigue Load

For inserts subjected to cyclic loads, such as those applied by vibrating mechanical equipment, a fatigue load reduction factor should be considered. Based on the results discussed in section 4.7, the semi-lightweight concrete should have a fatigue reduction factor (ϕ_{fs}) of 0.70 and normal weight concrete should have a 0.65 fatigue reduction factor (ϕ_{fh}). The fatigue tests were performed on 10-ft long concrete slabs, which means that these factors have incorporated in them a reduction due to flexural tension cracking. It is also important to recognize that the probability of getting a fatigue load and a sustained load large enough to adversely affect the insert strength in a cumulative fashion is extremely small.

5.8. Angular Load Effect

No load reduction was observed with inserts and/or loads out of alinement up to 20 degrees as measured from a plane perpendicular to the plane of the slab.

5.9. Experimental Scatter Factor

Based on the discussion in section 4.3 for normal weight and semi-lightweight concrete pull-out loads normalized to 3,000 psi, it was concluded that reduction factors are required to account for experimental scatter. In order to assume that 95 percent of the pull-out loads are as large as the average value in a 4 × 4 specimen an experimental scatter factor for normal weight concrete (ϕ_{eh}) of 0.82 should be used. A similar experimental scatter factor for semi-lightweight concrete (ϕ_{es}) is 0.75.

6. Design Criteria

6.1. Design Recommendations

6.1.1. Design Safety Margin

In sections 5.4 through 5.9 four reduction factors were defined. These are summarized as follows:

Experimental Scatter Factors,	ϕ_{eh}	= 0.82
	ϕ_{es}	= 0.75
Fatigue Reduction Factors,	ϕ_{fh}	= 0.65
	ϕ_{fs}	= 0.70
Flexural Cracking Factor,	ϕ_t	= 0.90
Sustained Load Factor,	ϕ_s	= 0.85

It is assumed that the individual reduction factors may be combined by cumulative multiplication to account for the effect of several variables in one factor. The combined fatigue reduction factor, ϕ_{cf}, should include the effect of experimental scatter and fatigue. Since the fatigue tests were performed on long slabs, the fatigue factors, ϕ_{fh} and ϕ_{fs} include a flexural cracking effect. Sustained load does not need to be considered since this would not ordinarily occur with a fatigue load. Thus the combined fatigue reduction factor for normal weight concrete, ϕ_{cfh}, is:

$$\phi_{cfh} = \phi_{eh} \times \phi_{fh} = 0.53$$

The similar factor, ϕ_{cfs}, for semi-lightweight concrete is:

$$\phi_{cfs} = \phi_{es} \times \phi_{fs} = 0.53$$

Since these factors are equal, only one combined fatigue reduction factor, $\phi_{cf} = 0.53$, needs to be considered in design. This is the most severe combination of reduction factors.

It is desirable to select a design safety margin to account for strength variations and occasional structural overloads normally expected. In the case of live loads, one commonly used load factor is 1/1.8 = 0.56 (See ACI 318-63). Combining this factor with the most conservative of the reduction factors makes the total reduction factor (ϕ) 0.29.

6.1.2. Design Equations

The recommended design equation given below is limited to inserts of the type tested when embedded in $4\frac{1}{2}$-in thick concrete with $\frac{3}{4}$-in cover over reinforcing steel. The equation is limited to concrete ranging in compressive strengths from 3,000 to 5,000 psi. Additional limitations are listed in section 6.2. Subject to the above restrictions, the recommended equation for the design load on an insert is:

$$P = \phi(2.0 + 0.0012 W_c \sqrt{f_c'}). \qquad 6.1(1)$$

For $\phi = 0.29$, this becomes:

$$P = 0.58 + 0.00035 W_c \sqrt{f_c'} \qquad 6.2(2)$$

where:

P = allowable design load per insert (kips),
W_c = unit weight of concrete (lb/ft^3),
f_c' = compressive strength of concrete (psi).

Some typical values of P have been tabulated in table 10 for several different concrete strengths.

TABLE 10. *Allowable design loads*

Concrete strength, f_c'	Design loads	
	Normal-weight concrete	Semi-light-weight concrete
psi	*kips*	*kips*
3000	3.3	2.8
4000	3.7	3.2
5000	4.1	3.5

Notes:

1. Design loads computed for inserts embedded in concrete with unit weights of 142 lb/ft^3 for the normal weight and 118 lb/ft^3 for the semi-lightweight concrete.
2. Semi-lightweight concrete with lightweight coarse aggregates and fine aggregate of normal-weight sand.
3. These are allowable design loads for only the inserts when embedded in $4\frac{1}{2}$ in thick reinforced slabs made with $\frac{3}{4}$ in cover over reinforcing steel and made with the specified concrete. The effect of the fatigue loading on the connecting hardware must also be considered. In general the maximum allowable load on an insert may be fully controlled by the allowable load on the connecting hardware when cyclic loading is expected.

6.2. Precautions and Limitations

6.2.1. Connecting Hardware

The recommendations presented in this paper are for inserts, similar to those tested, embedded in properly placed reinforced concrete slabs. However, the critical factor in the insert suspension system subjected to fatigue loading may not be the pull-out strength of the insert but the properties of the connecting hardware, specifically, the threaded rods. It was pointed out in section 4.7.5 that approximately 50 percent of the hardware used in the fatigue tests failed before the inserts. No specific recommendations concerning connecting hardware can be made since no records were kept of the individual bolts used in the fatigue tests. But it is apparent that allowable design insert loads as computed from the equation of section 6.1.2 and as presented in table 10 should be used only for moderate fatigue or static loading conditions unless the fatigue properties of the connecting hardware are fully considered.

6.2.2. Insert Spacing

If it is desirable to have inserts spaced closer than 3 ft on centers the allowable load on each insert should be reduced. In light of the absence of data on this factor it seems advisable to limit insert loading so that the total design load on the inserts within any 3 ft diameter area is not greater than the allowable load on a single insert. This needs research because there could be situations where it would be advantageous to group a number of inserts close together.

6.2.3. Inserts Other Than Those Tested

It is recommended that inserts, different from those tested, be evaluated using the 4 × 4 reinforced slabs and the static test procedures described heretofore.

6.2.4. Inserts in Lightweight Aggregate Concretes

When designing slabs to be made from either lightweight or semi-lightweight concretes the allowable insert load may be less than the 3.0 kips, normally used by many designers. If this 3.0 kips insert load is considered to be the lower limit, the design compressive strength for the lightweight aggregate concrete may have to be increased. This increase presents no problem for most of the aggregates produced at the present time. An alternate to this increase in the concrete strength would be to require an increase in the unit weight of the concrete.

6.2.5. Installation of Inserts

During this investigation a number of inserts were "lost" while placing the concrete. This can easily happen in the construction of an actual structure since the method of holding the insert is not accident-proof. The concrete handlers should be warned to keep their tools and vibrator spuds away from the inserts. In the laboratory good results were obtained by assigning one man to be fully responsible for the inserts and for placing the concrete around the inserts.

If the loss of inserts is a major problem in the field it may be necessary to design a better method of holding the inserts while placing the concrete. In the event of a missing insert some type of a drilled-in anchoring device would have to be established. The design recommendations for inserts in this paper do not apply to these other types of anchors.

The work described herein was carried out in the structural laboratories of the Building Research Division at the National Bureau of Standards. The program was sponsored by the U.S. Post Office Department. Liaison with the Post Office Department was provided by W. J. Werner who is now with the U.S. Department of Housing and Urban Development. Frank Erskine of the Expanded Clay and Slate Institute, Washington, D.C., arranged for the Institute's donation of the expanded shale lightweight aggregates.

Appreciation is extended to J. E. Breen, Professor of Civil Engineering at the University of Texas at Austin, who reviewed the paper and made many helpful suggestions.

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15. SUPPLEMENTARY NOTES

16. ABSTRACT (A 200-word or less factual summary of most significant information. If document includes a significant bibliography or literature survey, mention it here.)

Detailed test procedures are presented for a research program on cast-in-place inserts embedded in reinforced concrete. Three types of inserts, two of malleable iron and one of ductile steel, capable of receiving a 3/4 inch threaded rod were tested. Other variables included concrete aggregate type, concrete strength, reinforcement cover and spacing, angular loading, flexural cracking, sustained load and fatigue loading.

It was found that the pull-out load for an insert could be approximated by a linear function of the concrete unit weight and square root of the compressive strength in a statically loaded reinforced concrete slab. The effect of other variables is related to the insert pull-out loads in these slabs. Design recommendations are presented.

17. KEY WORDS (Alphabetical order, separated by semicolons)
Anchors; concrete slabs; design loads; fatigue; inserts; pull-out loads; sustained load.

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