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MATERIAL AND DIMENSIONAL PROPERTIES OF AN

ELEVEN-STORY REINFORCED CONCRETE BUILDING

by

R. A. Gardiner and D. S. Hatcher

Final Report to the National Science Foundation

on NSF Grant ENV 76-08244

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Full-scale destructive testing of an eleven-story reinforced concrete structure was conducted. Objectives were to investigate the in-situ structural dimensions and material properties, to determine the dynamic characteristics of the structure by small amplitude shaking tests, and to observe the structural damage and the degradation of the dynamic characteristics due to large amplitude shaking tests. This investigation concerns only the study of the in-situ structural dimensions and material properties. The methods used in the determination of the strength of the materials in the structure and the results obtained thereby are reported. Included are both the yield and ultimate strength of the reinforcing steel and the compressive strength of the concrete. An enumeration is provided of the variations of the structural dimensions from their specified values and the manner in which these variations were determined. The effect of all the above variations on the flexural strength of representative sections of members in the structure is shown. Also included is a comparison of these results with the capacity reduction factors specified by the A.C.I. Code (318-77). Conclusions state that variations of dimensions and properties of the structure investigated are generally similar to those of other buildings, and that the average strength of the members exceeds the design strength.

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ABSTRACT

MATERIAL AND DIMENSIONAL PROPERTIES

OF AN ELEVEN-STORY REINFORCED

CONCRETE BUILDING

by Ronald A. Gardiner

ADVISOR: Professor D. Hatcher

August, 1978

Saint Louis, Missouri

This paper describes the comparison between the in-situ structural properties and dimensions and the specified properties and dimensions of an eleven story reinforced concrete building. In addition, the effect of the structural variations on the flexural strength of the members is investigated.

The conclusions of this paper are that variations of dimensions and properties of the structure investigated are generally similar to those of other buildings, and that the average strength of the members exceed the design strength.

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MATERIAL AND DIMENSIONAL PROPERTIES OF AN ELEVEN-STORY REINFORCED CONCRETE BUILDING

1. INTRODUCTION

During the summer of 1976 a rare opportunity to perform full scale destructive testing of an eleven-story reinforced concrete structure occurred (1)*. The building, although still structurally sound, was nonfunctional socially and was therefore slated for demolition. A testing procedure was proposed, and permission was received to carry out the project on the building prior to its destruction. The objectives of this project were to investigate the in-situ structural dimensions and material properties, to determine the dynamic characteristics of the structure by small amplitude shaking tests, and to observe the structural damage and the degradation of the dynamic characteristics due to large amplitude shaking tests.

The last two of these objectives are described elsewhere (1). This investigation concerns only the first of the objectives, the study of the in-situ structural dimensions and material properties.

*The numbers in parentheses in the text indicate references in the Bibliography.

This inquiry can be divided into three areas of interest. First, the comparison of the material properties in the structure with the specified properties. Second, the comparison of the in-situ structural dimensions with the specified dimensions. Finally, the determination of the effect of the variations in material properties and structural dimensions on the flexural strength of the members in the structure and the detection of any relationship between this effect on the strength and the capacity reduction factors specified by the A.C.I. Code [318-77] (2).

Chapter two of this paper is a report of the methods used in the determination of the strength of the materials in the structure and the results obtained thereby. This includes both the yield and ultimate strength of the reinforcing steel and the compressive strength of the concrete. Chapter three is an enumeration of the variations of the structural dimensions from their specified values and the manner in which these variations were determined. In chapter four the effect of all the above variations on the flexural strength of representative sections of members in the structure is shown. Also included is a comparison of these results with the capacity reduction factors specified by the A.C.I. Code [318-77] (2). Chapter five is a summary of the results of this investigation while chapter six contains the conclusions drawn from these results.

The investigation does not provide the final answers concerning the in-situ variation of structural properties and dimensions or their effect on the strength of structures. Others (3-8) have studied these problems and many of the investigations on which they have reported have been of larger scope or broader data base than the project reported here. However the body of data concerning structural variations and their effects is still relatively small and the results determined in this investigation are important as they will add at least a small amount of additional information in this vital area.

2. STRENGTH OF MATERIALS

In any building design, one of the most important parameters with which the designer has to work is the strength of the materials to be used in the construction. The strength of the members constructed may vary greatly from the design value if the materials used have different strengths from the values specified. During the process of designing the building investigated in this study, values were specified for the compressive strength of the concrete and the tensile strength of the reinforcing steel. The determination of the actual strength of the structure in reality is compared to the design idealization. Given in this chapter are the details of the determination of the strength of the reinforcing steel from tests of samples taken from the structure and the determination of the in-situ concrete strength from the results of non-destructive tests performed on the material. In both cases, comparisons are made between the actual strength of the material and the specified value of that strength.

2.1 STRENGTH OF REINFORCING STEEL

Since there is no non-destructive test to measure the strength of the reinforcing steel in the structure, samples were tested in the laboratory. Because of the desire to perform the shaking tests on the building in an undisturbed state, these samples were obtained, for the most part, after all testing was completed. The only exception to this was three #3 bars obtained from the roof slab that was cut open for the installation of the shaking equipment (1). The majority of the specimens were obtained by taking samples of bars after the building was razed and before the debris was carried offsite to be dumped. Those samples that were reasonably straight after the demolition and could be freed from the rubble were mostly slab reinforcement bars. Only two beam reinforcing bars were obtained because of the difficulty of extricating these from the debris.

The specimens were marked with an 8-inch gage length and then tested in tension in a universal testing machine. The yield point of the steel was noted by a drop-off in the load on the bar. The test continued until fracture occurred. The yield and ultimate stresses were obtained by dividing the yield load and ultimate load by the nominal cross-sectional area of the bar tested. The ultimate strain was calculated by dividing the difference in gage length before and after testing by the initial gage length. The results are presented in Table 1. The average yield stress of the eight #3 bars tested was 60910 p.s.i. The small number of #4 bars and #5 bars tested do not give results that can be considered statistically significant. However, the yield stress of the #5 bars is approximately 90% of the yield stress of the #3 and #4 bars. The specified value of the yield strength of the reinforcing steel was 50,000 p.s.i. Thus the average yield strength of the #5 bars is 10% higher than the design value while the average yield stress of the #3 and #4 bars is 20% higher than the design value. These values are within the range determined by other investigators.

2.2 STRENGTH OF CONCRETE

In order to assess the strength of concrete in a structure, no matter what method is used, a sample of the strength at a statistically significant number of unbiased locations is required. For this building, the roof and the ground floor were eliminated from study

TABLE 1. RESULTS OF REINFORCING BAR TESTS

	Yield Stress (p.s.i)	Ultimate Stress (p.s.i.)	Ultimate Strain (in./in.)
		#3 bars	
	60460	100900	0.031*
	62900	102400	0.043*
	62000	101500	0.080*
	62550	101700	0.136
	60910	101900	0.158
	58460	101700	0.165
	59090	100800	0.145
	60910	100400	0.159
Average	60910	101400	0.115
		#4 bars	
	61000	112800	0.106
	64000	116000	0.126
Average	62500	114400	0.116
		#5 bars	
	56450	93390	0.023*
	54840	105200	0.042*
Average	55650	99270	0.042

*Bar broke outside of the gage length

because of inaccessibility to the slab. The remaining floors were assigned a random number equivalent (Figure 1). Each floor was then divided into a 5 ft x 5 ft grid (Figure 2) which could be defined by a two-digit number, the first representing the east-west coordinate, the second the north-south coordinate. This gave a total of 90 locations per floor on ten floors, or a total of 900 locations in the structure. The concrete strength was determined at 99 points, which was slightly greater than 10% of the total. The locations to be tested were chosen by picking three-digit numbers, the first corresponding to the floor and the last two to the grid location, from a random number table. For example, the random number 629 corresponds to the point 7I2 (Figure 2) which is 10 feet west of the northeast corner column on the seventh floor.

The determination of the compressive strength of the concrete could have been accomplished in several ways. Destructive tests in the laboratory of samples cored from the structure could have been used exclusively. However, this would have been expensive and is not necessarily the best method, as the results obtained from the testing of cores are subject to some interpretation. For these reasons, non-destructive testing methods were preferred. Such methods include, 1) Surface hardness tests, by the use of an impact-rebound hammer or a Windsor Probe, or 2) Sonic velocity tests. The impact-rebound hammer test and a sonic velocity test were used. These tests give results which are also subject to interpretation. To help in interpreting the results, cores were taken at a small number of points to calibrate the non-destructive tests.

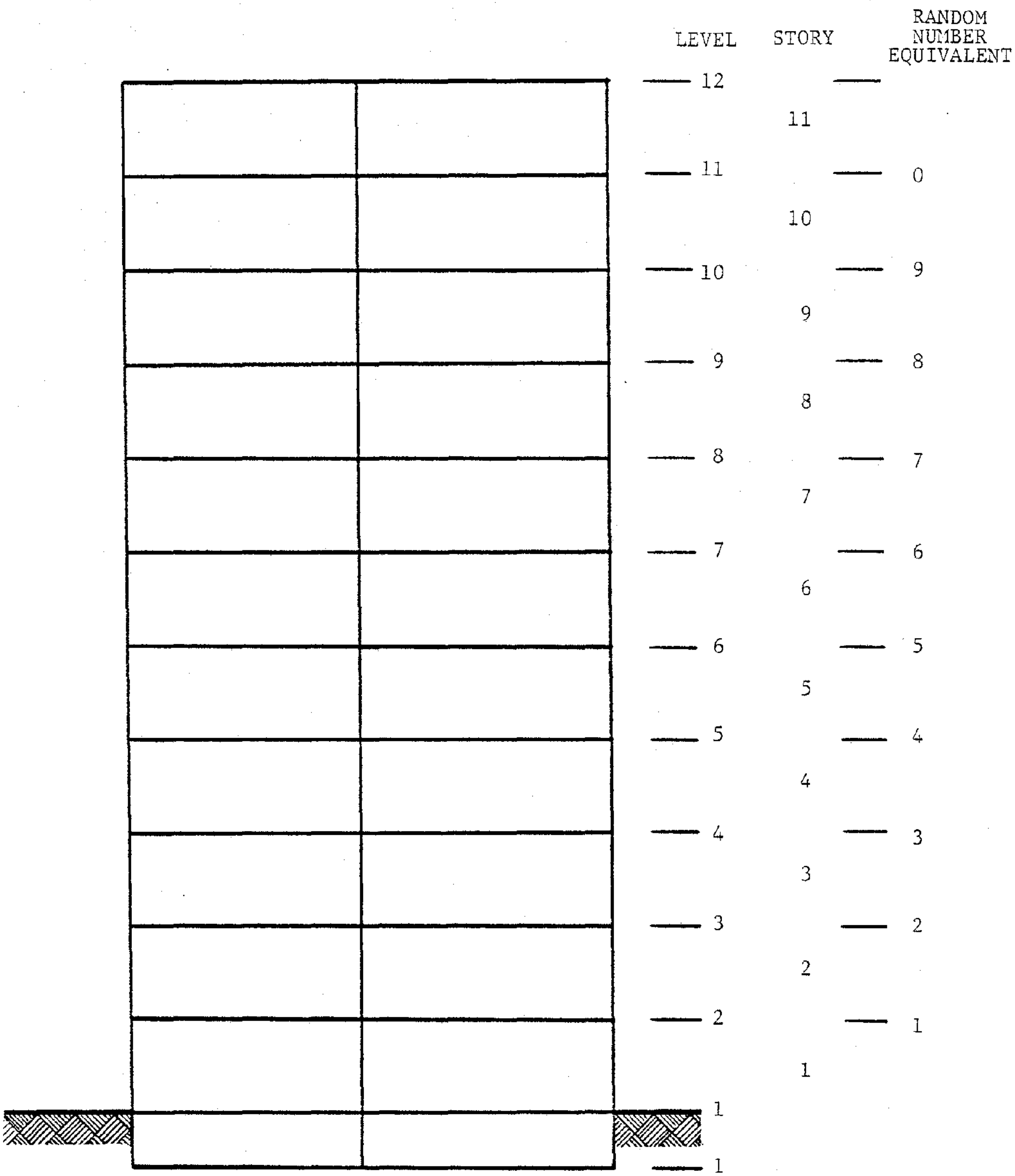


FIGURE 1. DESIGNATION OF VERTICAL COORDINATES

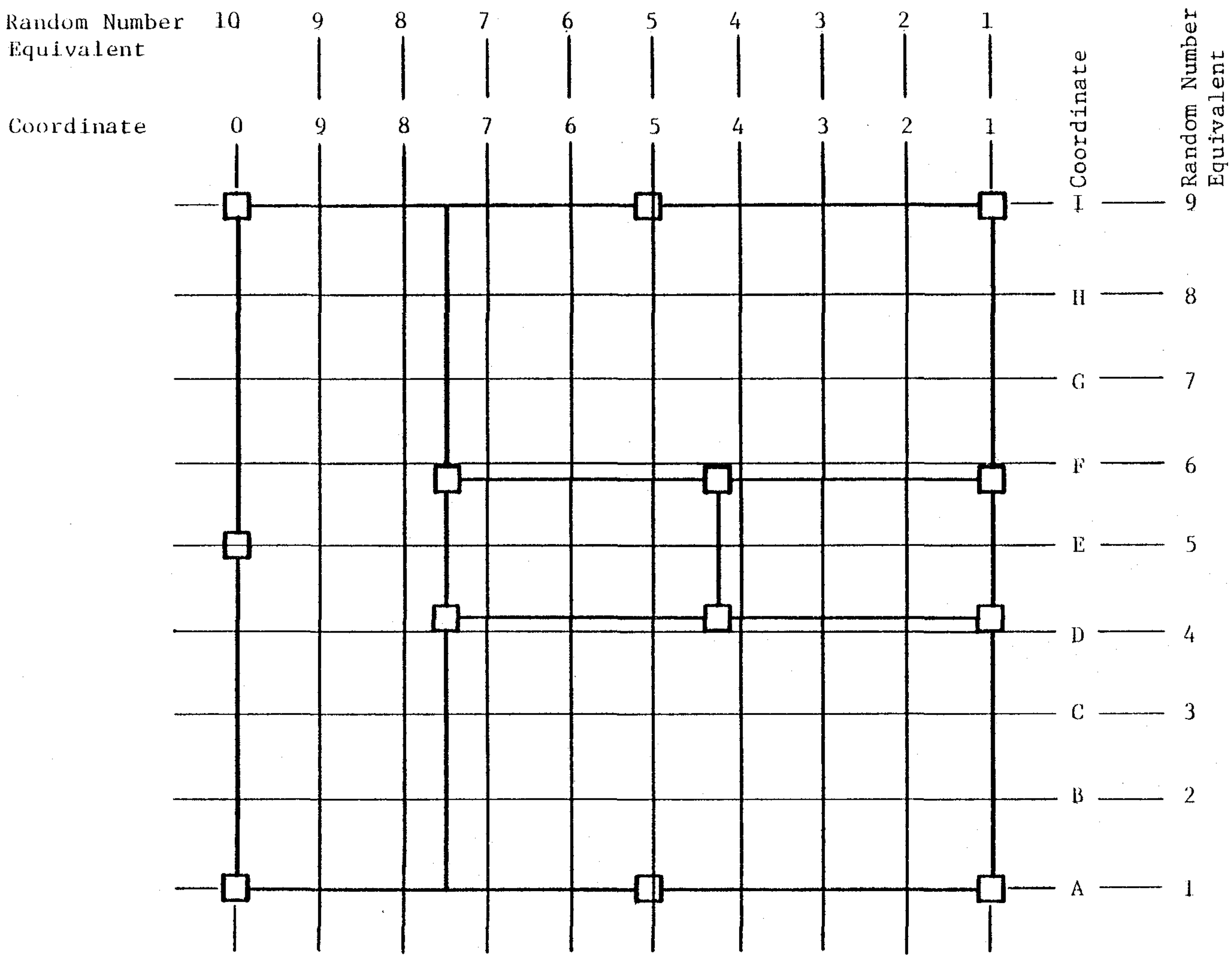


FIGURE 2. FLOOR GRID

The surface hardness test was the first of the non-destructive tests to be performed. This was done with the use of a Concrete Test Hammer, Model CT-320, purchased from the Soiltest Company. This device is similar to others known as "Schmidt hammer", "Swiss hammer", or "rebound hammer." The first two names are traced to the invention of the device by Ernest Schmidt while working in Switzerland. The last name implies the principle on which the hammer works. The rebound of an elastic mass is related to the modulus of elasticity, E, of the material against which it impinges. For concrete, the compressive strength is also related to the modulus of elasticity. Consequently, for concrete the rebound reading of the test hammer is related to the compressive strength. A strong concrete will cause a greater rebound than a weaker concrete when struck by the hammer.

The test hammer is subject to some errors if care is not taken in its operation. For instance, a test specimen must be rigidly supported or part of the hammer impact energy will be lost in displacing the mass of the specimen and a true rebound reading will not be obtained. The test hammer measures surface properties. Direct comparison cannot be made of readings taken of two different types of surfaces such as a surface formed by a steel mold and one formed by a wood mold. For good comparison of two surfaces of the same type, both surfaces should be smooth, clean, and flat. In addition, this device is subject to variations due to local surface anomalies. A large piece of aggregate just beneath the surface of the concrete may cause an extremely large rebound reading, just as a void just below the surface may cause a very low rebound reading. For these reasons the recommended procedure (9,10) is to take a large number of readings in a small area, discard the highest and lowest and average the remaining reading to obtain the rebound number. The procedure followed in this investigation was to take seven rebound readings on both the top and the bottom of the slab at each location. One reading was taken at each point of an 8-inch hexagon and one at the center of the hexagon (Figure 3). The top readings were handled separately from the bottom readings. The readings were averaged and any reading that differed more than two standard deviations from the mean was rejected. The adjusted average was taken as the rebound number. By the above procedure, two rebound readings were obtained for each location tested, one for the top of the slab and one for the bottom of the slab.

There has been a great deal of discussion regarding the interpretation of the results obtained when using the test hammer to predict concrete strength. Some authors (10,11) express great optimism about the validity of the predicted strength no matter what type of concrete the device is used on. For example, the Soiltest Company supplies a set of curves for predicting compressive strength of concrete for a given rebound reading and angle of impact (Figure 4). Most authors (12,13,14,15) feel that the prediction of concrete strength on the basis of rebound reading is very reliable if done with a calibrated curve, or batch curve, obtained for each type of concrete to be tested. This batch curve would be derived by destructively testing samples of the concrete to determine the compressive strength corresponding to a given rebound reading. Despite the use of a batch curve, Bloem (4) objects that a single rebound reading may be obtained from concrete samples with a range of strength of up to 2 k.s.i. Greene (11) states, however, that the coefficient of variation for a rebound test on a

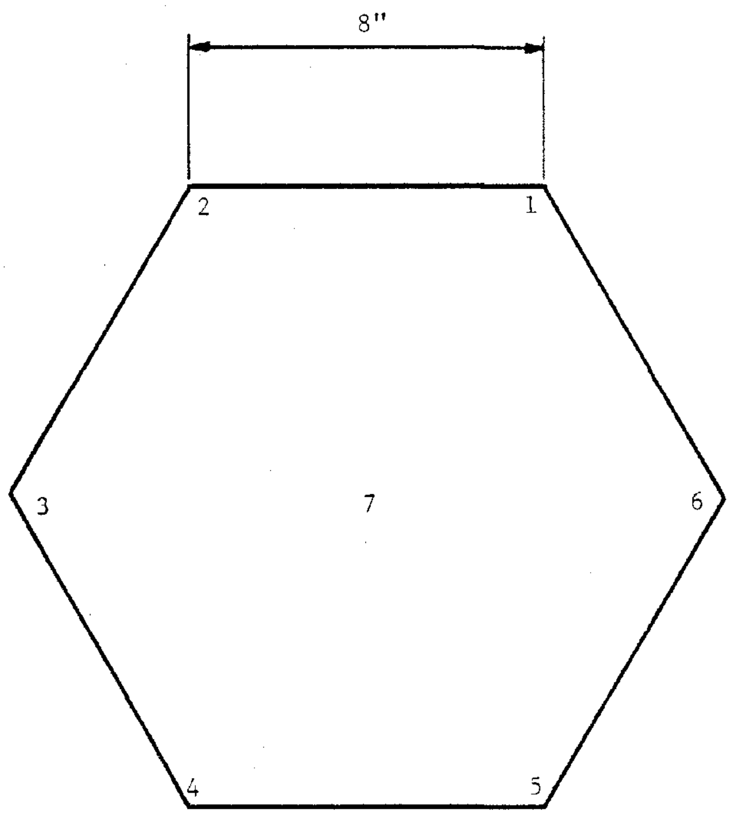
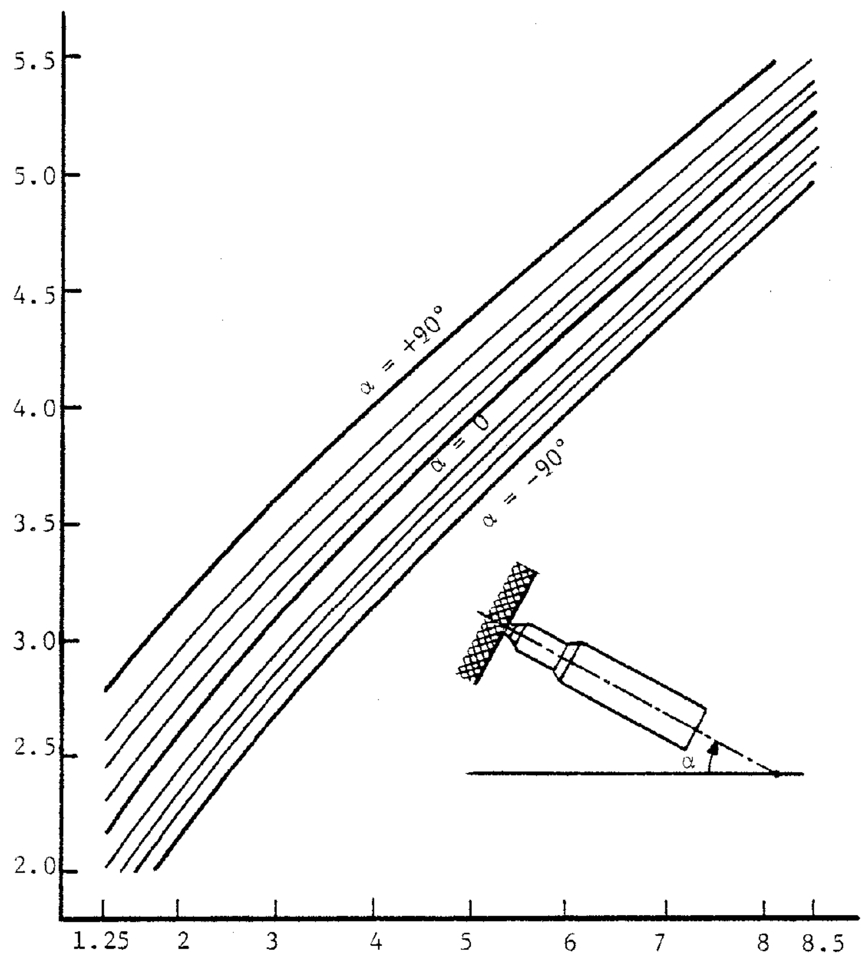


FIGURE 3. REBOUND HAMMER OPERATION PATTERN



Cylinder Compression Strength (k.s.i.)

FIGURE 4. REBOUND HAMMER CALIBRATION CURVES

given sample of concrete is only slightly higher than the coefficient of variation obtained when testing that sample in a testing machine. It was considered possible then to use the test hammer to predict at least a range of the values of the compressive strength of the concrete in the structure.

The second non-destructive test involved a sonic-velocity meter, also known as a v-meter. The main components of the v-meter (Figure 5) are transducers (left and right), a transmitter and receiver, and a high speed electronic clock. As the transmitter sends an ultrasonic pulse through the sending transducer into the test specimen, the timer is activated. When the pulse reaches the receiving transducer, it is converted to an electrical signal and turns off the timer. Transmission time divided by the distance between transmitting and receiving transducers gives pulse velocity. There is discussion of the manner in which the pulse velocity through a material may be related to any other property of the material. Of particular interest with regard to concrete is the possible relationship between the pulse velocity and either the compressive strength, f'_c, or the elastic modulus, E_c. Nwokoye (16) shows several curves that demonstrate a strong relationship between pulse velocity and cube crushing strength. However, more widely accepted is the relationship (10)

$$V = \sqrt{E/\rho}$$

where V is the sonic velocity through the material

E is the elastic modulus

and ρ is the density.

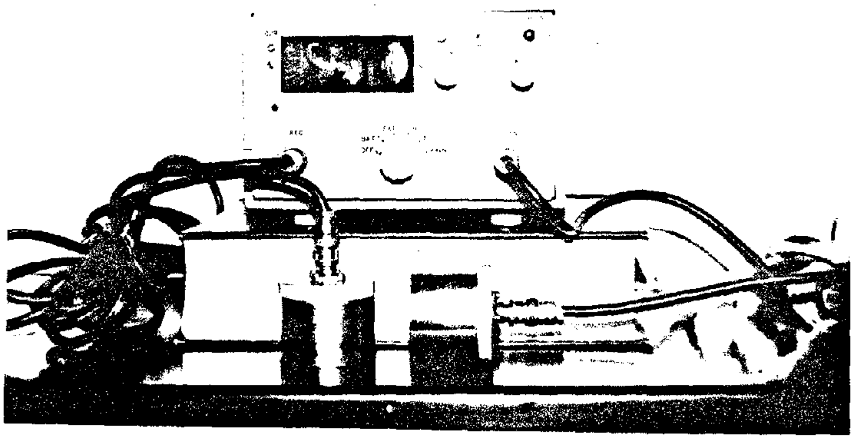


FIGURE 5. SONIC VELOCITY METER

This can be written as

$$E = \rho \ V^2$$

For concrete, the relationship would be

$$E_c = \rho \ V^2 \qquad [1]$$

Using this relationship and that suggested by Pauw (17) and used in the A.C.I. Code (2) for E_c, it was considered possible to relate the pulse velocity and the compressive strength of the concrete in the structure. In the A.C.I. Code the expression for E_c is

$$E_c = 57000 \ f'_c \qquad [2]$$

where E_c is the elastic modulus in p.s.i.

and f'_c is the compressive strength in p.s.i.

As stated earlier, cores were obtained from the structure for the purpose of calibrating the two non-destructive tests. A four-inch core was to be taken from one location on each floor with the exact positioning of the drill determined, by a method to be discussed later, so as to avoid interference with the reinforcement in the slab. The cores were obtained by a commercial testing firm. The firm purchased a new four-inch bit which was expected would complete this job and be used on others. However, because of the extreme hardness of the concrete, the bit was worn out before half of the desired cores had been cut. Since no other four-inch bit was available, the remaining cores were cut using three-inch bits. An extra three-inch core was taken approximately six inches from the location of a four-inch core. This was done in an attempt to isolate any differences due to L/d effects that might occur in testing the two sizes of cores.

Rather than rely entirely on the relationships between compressive strength and the results of the two non-destructive tests developed by others, tests were conducted on samples in the laboratory. A series of six batches of thirty concrete cylinders were cast. These included two batches of each of three different strengths of concrete which were labeled 3, 4, and 5, respectively. One of the two batches, designated A, of each strength of concrete consisted of ten 6 x 12-inch cylinders, ten 4 x 8-inch cylinders, and ten 4 x 4-inch cylinders. The other batch, designated B, consisted of ten 6 x 12-inch cylinders, ten 4 x 8-inch cylinders, and ten 4 x 5-inch cylinders. The specimens with shorter lengths, and thus non-standard L/d ratios, were included to study the effects of the L/d ratio on the results of compression tests. This was necessary since the cores taken from the slabs in the structure, with thicknesses of either four inches or five inches, would also have non-standard L/d ratios. The tests with the concrete test hammer and the v-meter were performed on each cylinder 28 days after the specimens were cast. The data was recorded and the specimens were then tested to failure in compression.

Before the relationship between the non-destructive tests and the compressive strength could be determined, the true compressive strength had to be defined. The strength of a 6 x 12-inch cylinder tested under standard conditions (18) was defined as the true compressive strength. Hereafter, the term 'compressive strength' will be used only to refer to the strength determined by testing a 6 x 12-inch cylinder. The strength determined by testing any other size cylinder will be referred to as 'apparent compressive strength.' In order to determine the compressive strength of the concrete in any non-standard specimens tested it was necessary to determine the effect of the non-standard dimensions on the apparent strength. In particular, it was required to determine the ratio of apparent strength to compressive strength, hereafter referred to as R_s, obtained when testing a cylinder with:

1) a smaller diameter (3 or 4 inches) than standard (6 inches) and

2) a smaller L/d ratio (1.0 or 1.25) than standard (2.0).

The effect of testing a cylinder with a diameter smaller than standard, as reported by Malhotra (19), is quite varied for different types of concrete. The strength ratio, R_s, for 4 x 8-inch cylinders may be from 0.84 to 1.32. Price (20) reports an average R_s of 1.04 for 4 x 8-inch cylinders. The results of the compression tests of the cylinders cast in the laboratory in this investigation are shown in Table 2. The average value of R_s for the 4 x 8-inch cylinders was 0.99. Consequently, no correction factor was applied to the apparent strength of the cylinders with diameter smaller than standard to account for this size difference.

The effect of L/d on the strength ratio is handled in several different ways. The A.S.T.M. standard C-42 (21) contains a table of correction factors to be applied to the apparent strength of cylinders with various L/d ratios to obtain the compressive strength of the concrete in the cylinders. The values in this table represent points on the curve shown in Figure 6. Price (20) presents similar method for calculating compressive strength from apparent strength and L/d ratio. The only variable in these corrections is the L/d ratio. Kesler (22), however, presents information indicating that both the

TABLE 2

RESULTS OF COMPRESSION TESTS ON

4 x 8 and 6 x 12 IN. CYLINDERS

Concrete Batch	Average Cylinder Strength of Size (p.s.i.) 4 x 8	6 x 12	R_s
3A	5480	5550	0.989
4A	7590	7470	1.020
5A	8960	8440	1.060
3B	5740	6010	0.954
4B	8170	8330	0.981
5B	6120	6230	0.982
		Average	0.997

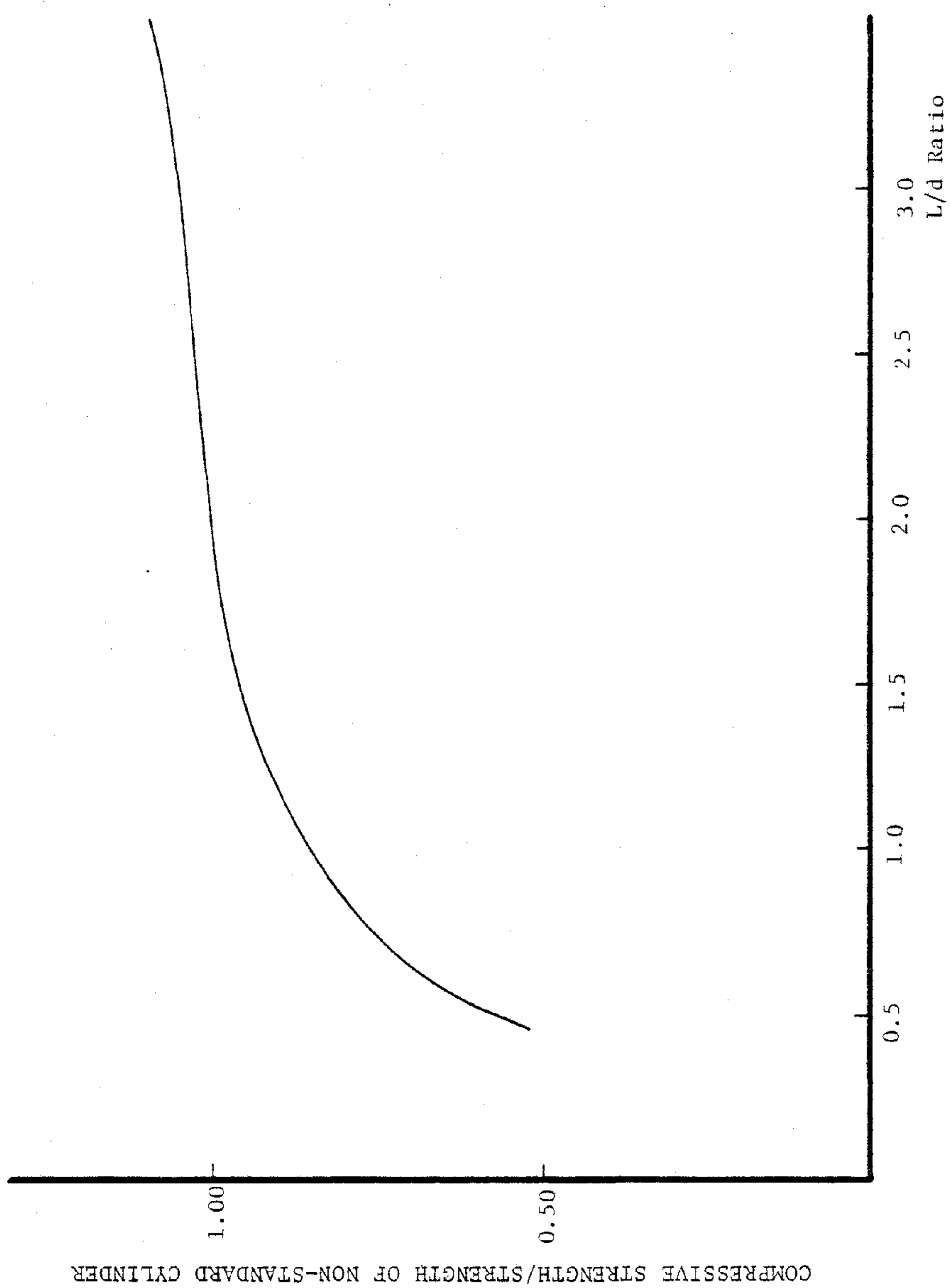


FIGURE 6. STRENGTH RATIO OF NON-STANDARD L/d RATIO CYLINDERS

L/d ratio and the compressive strength should be variables in determining the required correction factors. For example, for a given L/d ratio lower strength concretes require greater corrections than do higher strength concretes. The results of Kesler's work are presented in Figure 7. This figure was used to obtain the compressive strength of the concrete in the cylinders with non-standard L/d ratios tested in this study. This resulted, as shown in Table 3, in a strength ratio of 0.88. Although the average value of R_s before Kesler's corrections were applied was closer to 1.0 (R_s = 0.97), the corrected values based on Kesler's work were used for two reasons. First, the results were conservative, if in error. Second, it was not felt that the results obtained on the basis of the smaller number of cylinders tested in this investigation should be preferred to Kesler's results.

With the compressive strength of the concrete in the cylinders established as described above, an attempt was made to determine the relationship between that strength and the concrete test hammer rebound readings. That attempt was not very successful. The compressive strength of the concrete in each of the 4 x 8 and 6 x 12-inch cylinders was plotted against the average of the rebound readings obtained from tests on those cylinders. The results were widely scattered as shown in Figure 8. A regression line through the points showed a very low correlation coefficient (Table 4). Seeking an explanation for these results, the data from the 6 x 12-inch cylinders was analyzed separately from the data from the 4 x 12-inch cylinders. Analysis of the results from the 6 x 12-inch cylinders indicated a strong linear relationship as shown by the correlation coefficient listed in Table 4. Similar analysis of the results from the 4 x 8-inch cylinders showed a similar

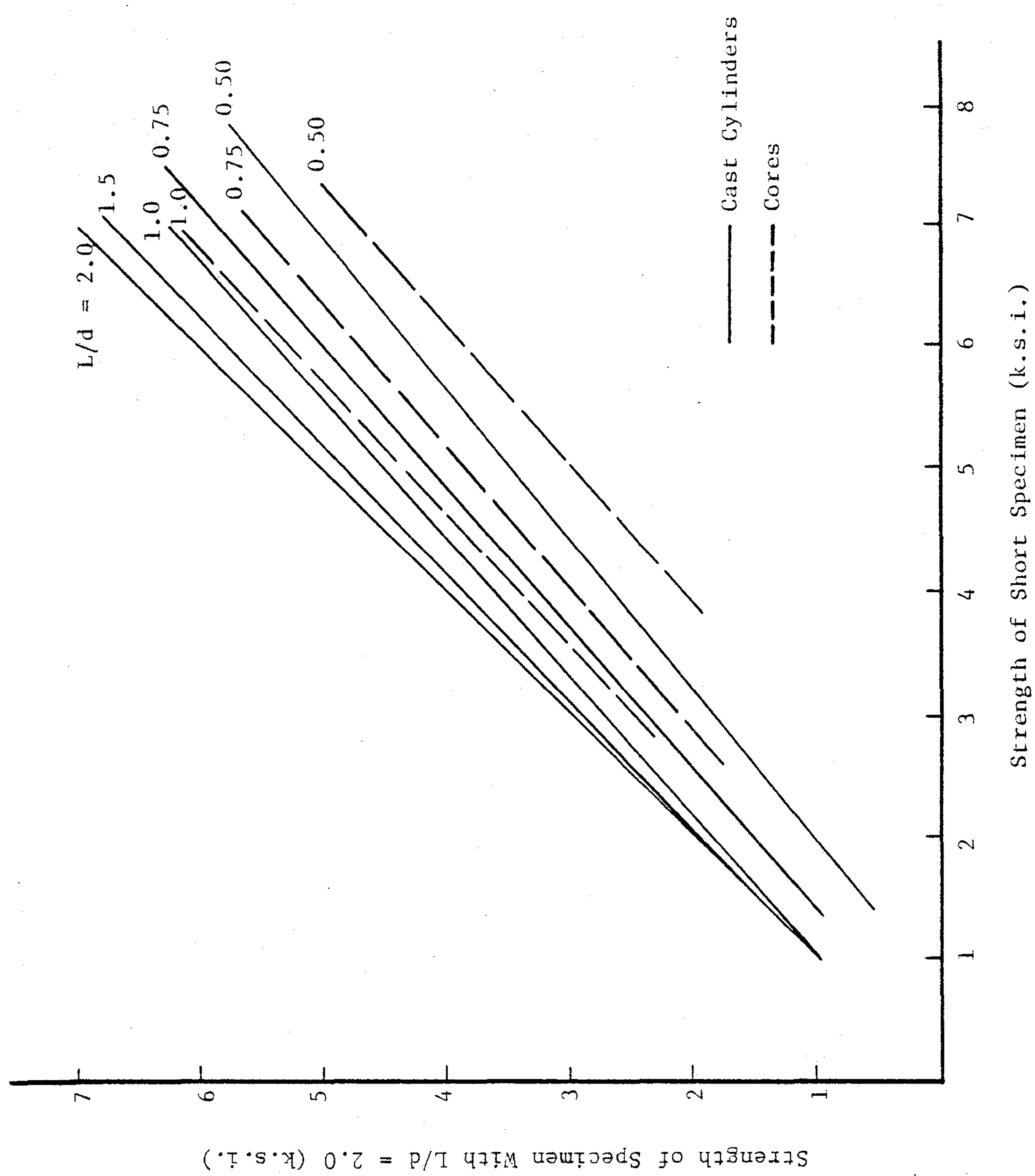


FIGURE 7. KESLER'S CORRECTIONS FOR NON-STANDARD L/d CYLINDERS

TABLE 3

RESULTS OF COMPRESSION TESTS ON SHORT CYLINDERS

Concrete Batch	L/d	Apparent Strength	R_s	Corrected Strength	R_s
3A	1.00	4930	0.888	4380	0.789
4A	1.00	7240	0.970	6430	0.860
5A	1.00	8120	0.963	7200	0.853
3B	1.25	6180	1.030	5650	0.940
4B	1.25	8000	0.961	7500	0.901
5B	1.25	6240	1.000	5700	0.915
		Average	0.969	Average	0.876

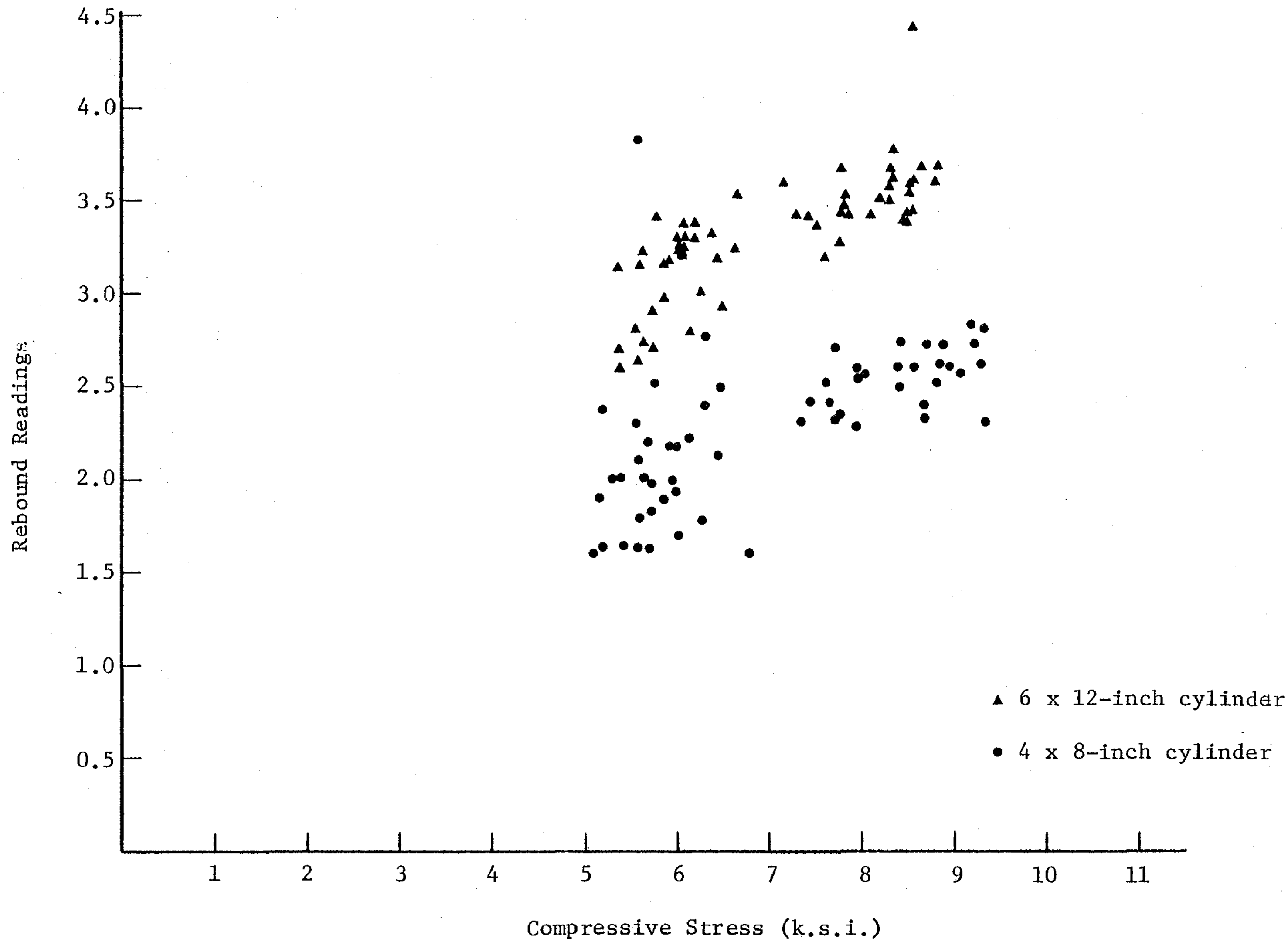


FIGURE 8 REBOUND READINGS VERSUS COMPRESSIVE STRENGTH FOR CYLINDERS

TABLE 4

REGRESSION ANALYSIS OF STRESS VERSUS REBOUND READINGS

X = Compressive Strength
Y - Rebound Reading

Equation: $X = A + B(Y)$

Cylinder Data Groups	A	B	R^2
4 x 8 in. and 6 x 12 in.	4819.0	781.7	0.147
6 x 12 in.	-2041.0	2731.0	0.565
4 x 8 in.	2368.0	2033.0	0.361

linear tendency although the correlation coefficient was not as strong. The regression lines given in Table 4 for the 4 x 8 and 6 x 12-inch cylinders are shown in Figure 9. The calibration curve supplied by Soiltest with the test hammer is shown for comparison. There is a great difference between the lines generated from the results of the different size cylinders and between the Soiltest curve and either of the two lines. For a given rebound reading, the regression line for the 6 x 12-inch cylinders would indicate a much higher compressive strength than the Soiltest curve and the regression line for the 4 x 8-inch cylinder would indicate an even larger compressive strength. The reason for the variation in the regression line was determined to be the method of operation of the rebound test. Supports were made for each size cylinder (Figure 10a) so that the cylinders would be held as shown in Figure 10b. It was intended that the rebound tests would be performed with the hammer pointed downward, $\alpha = -90$, on the side of the cylinders (Figure 10c). Instead, the tests were performed with the hammer in a horizontal position, $\alpha = 0°$, on the bottom of the cast cylinders. The supports constructed did not provide adequate support against movement for the specimens with the tests performed in the horizontal direction. There was movement of the cylinder upon impact of the hammer, with more movement of the base for the 4 x 8-inch cylinders than for the one for the 6 x 12-inch cylinder. This movement absorbed part of the energy of impact leaving less energy to rebound the plunger. This resulted in a lower than correct rebound reading, with increasing movement causing greater reduction in rebound reading. Because of these incorrect rebound readings, the relationships determined from the results of the cylinder tests could

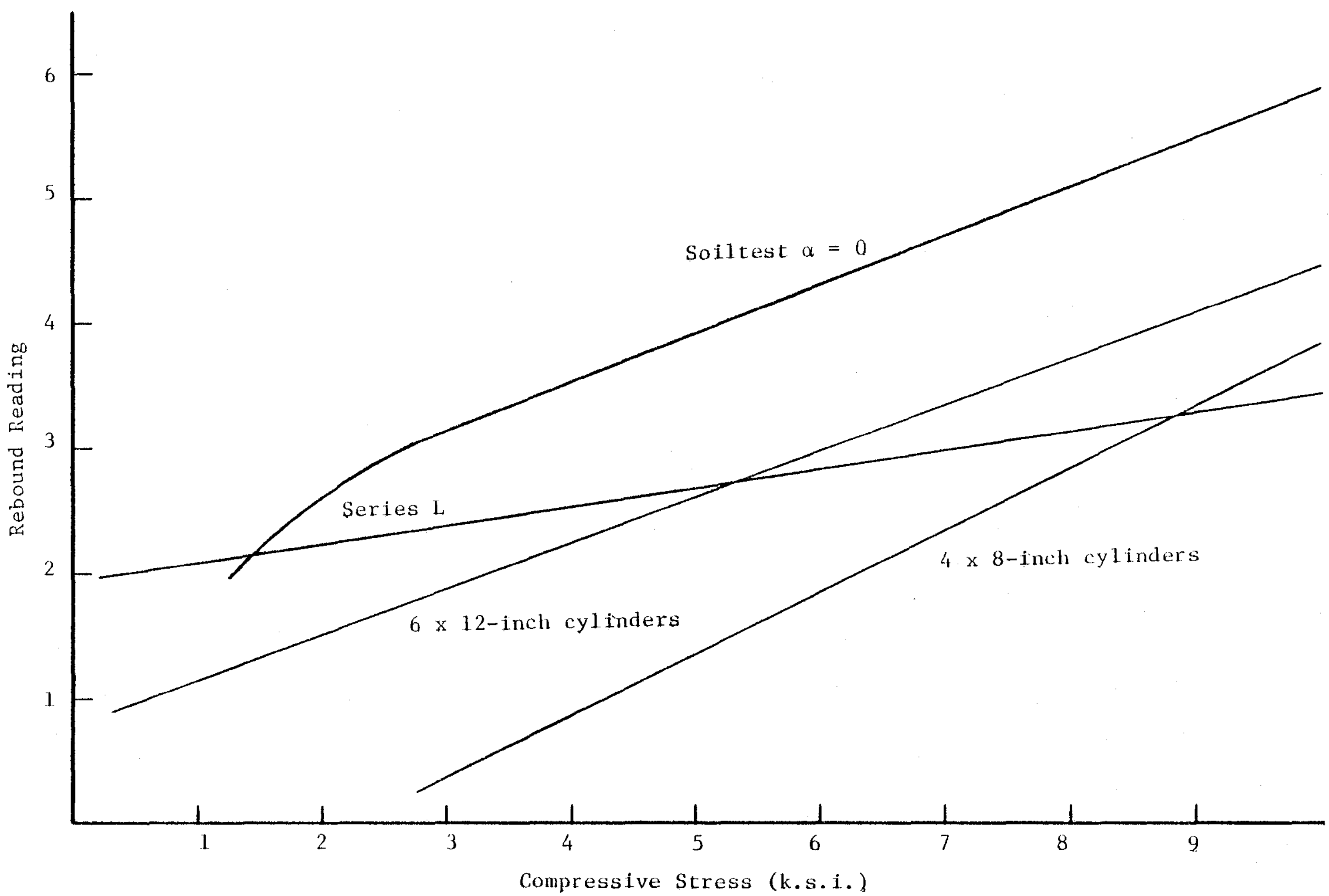


FIGURE 9. RELATIONSHIPS BETWEEN REBOUND READING AND COMPRESSIVE STRENGTH

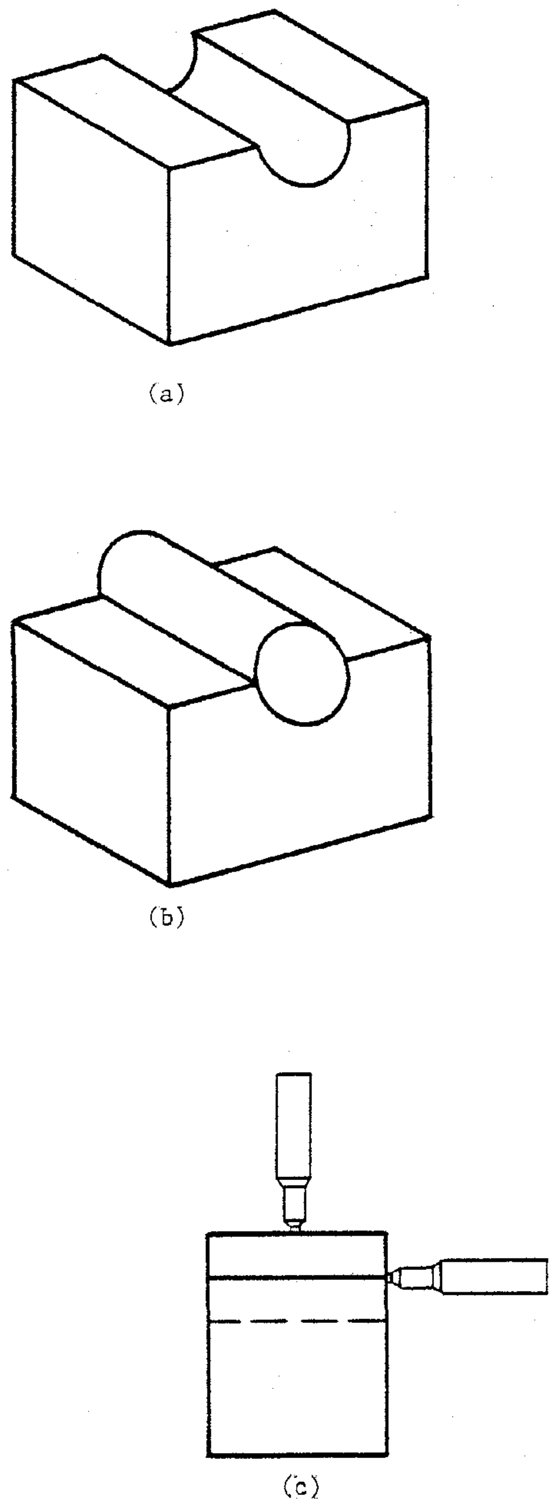


FIGURE 10. CYLINDER SUPPORTS FOR REBOUND TESTS

not be applied to the in-situ rebound readings. However, several valuable trends could be noted as a result of the cylinder tests. First, a strong linear relationship between rebound reading and compressive strength was evident. This relationship became statistically stronger as the specimen became more rigidly supported. Second, the slopes of the lines determined from the cylinder data were similar to the slope of the Soiltest curve. As a result, a general form of the relationship between these two variables could be predicted. The line representing the relationship would have a similar slope to the straight line portion of the Soiltest curves. Because of 'full fixity', it would lie somewhere above the regression line of the 6 x 12-inch cylinders. However, the exact placement would have to be determined by further testing of "fixed" specimens, or by destructive testing of cores from the structure to determine the compressive after using the concrete test hammer in-situ.

A second series of thirty 4 x 8-inch cylinders of varying strengths was cast to use in a new set of concrete test hammer tests. Much more fixity was obtained for this series. Cylinders were placed in the testing machine and loaded to 3,800 pounds (approximately 300 p.s.i.). The impact hammer was then used on the side of the cylinders. Ten rebound readings were taken and the average determined. This is similar to the procedure recommended by Zoldners (13). Linear regression was performed on the data from these cylinders. The resulting line is shown on Figure 9 labeled Series L. As can be seen, the results are poorer than those which the first series provided. At fault was the attempt to perform the rebound test on a non-flat surface. If the hammer is not held exactly perpendicular to the tested specimen, the impact delivered by the plunger is glancing. A glancing blow results in an incorrect reading. The likelihood of this type of glancing blow by the plunger is much greater when the rebound test is performed on the curved face of a cylinder than when the test is run on a flat surface. Since incorrect rebound readings were obtained in this series of tests, the end result was that no additional useful information was obtained from the tests.

The determination of the relationship between the compressive strength of the cylinders and the sonic velocity through the cylinders was obtained by relating each of these quantities to the elastic modulus, E_c. This was accomplished using equations [1] and [2] already given. It was not expected that complete agreement between the above equations would be achieved; however, it was expected that some constant, Q, could be obtained such that

$$Q(\rho V^2) = 57{,}000 f'_c. \qquad [3]$$

On the basis of the cylinders tested, the constant, Q, was determined to be approximately 0.7 (Table 5). Using that value and ρ=150 lbs/ft^3 the following relationship was obtained.

$$f'c = 7.6114 \times 10^{-18} V^4 \qquad [4]$$

This relationship showed a good statistical correlation, with R^2=0.52. Unlike the concrete test hammer tests, there were no operational differences between these tests whether carried out on cylinders in the laboratory or performed in-situ. Consequently, it was felt that this relationship was applicable in its present form to readings taken from the structure. Verification of this depended on calibration by the cores that were taken.

TABLE 5

VALUES OF THE RATIO Q

Concrete Batch	Group	Q
3A	1	0.614
	2	0.658
	3	0.684
4A	1	0.693
	2	0.715
	3	0.758
5A	1	0.709
	2	0.728
	3	0.696
3B	1	0.671
	2	0.631
	3	0.669
4B	1	0.688
	2	0.693
	3	0.677
5B	1	0.681
	2	0.714
	3	0.734
	Average	0.696

Before the cores taken from the structure could be used in calibrating the non-destructive tests, the true strength of the concrete in the cores had to be determined. Just as with the samples cast in the laboratory, the apparent strength had to be adjusted to account for size differences from a 6 x 12-inch cylinder. As with the samples cast in the laboratory, it was decided that no correction need to be applied to account for the difference in the diameter of the specimen. Also, the variation in L/d ratio was accounted for through the use of Figure 7. In question was whether any additional correction would be necessary to account for any damage to the cores due to the vibration and rotating friction of the coring process. The possibility of this type of damage was enhanced by the previously discussed difficulties encountered in obtaining the cores. For the purpose of ascertaining if such damage occurred, the sonic velocity through the core after coring was compared with the velocity through the same sample in-situ. The results, given in Table 6, show a consistently lower sonic velocity after coring. Clearly there was some damage to the cores. This damage must have been in the form of micro-cracking throughout the specimen, since it was reflected in a reduced sonic velocity through the center of the core. The manner in which to determine the magnitude of the effect of this damage on the apparent strength is not clear. Campbell and Tobin (23) have investigated this problem and have shown that cores cut from a slab test at lower strengths than cylinders cured under exactly the same conditions. There was no attempt on their part to explain this phenomenon. However, on the basis of their data, a model was developed in this investigation that would explain the difference in apparent strengths. This model takes the form of

TABLE 6

COMPARISON OF SONIC VELOCITY BEFORE AND AFTER CORING

Point	V_{core}	$V_{in-situ}$	Ratio
2 B-1	1.701	1.773	0.9596
3 A-9	1.660	1.764	0.9408
4 C-4	1.608	1.714	0.9383
5 E-9	1.636	1.778	0.9202
6 H-9	1.721	1.778	0.9679
3 A-9	1.722	1.794	0.9597
7 H-10	1.498	1.563	0.9587
8 H-4	1.538	1.684	0.9136
9 B-9	1.739	1.802	0.9649
10 C-4	1.706	1.735	0.9833
11 H-9	1.698	1.774	0.9574
		Average	0.9510

a damaged layer of concrete on the exterior of the core (Figure 11) which does not contribute to the strength of the core. The actual strength of the core would be computed by dividing the failure load by the area of concrete inside the damaged ring. Using the data from Campbell and Tobin which shows that four inch diameter cores test at 0.76 of the strength of identical cylinders, a damage layer (r_n-r_a in Figure 11) of 1/4-inch was determined to be the appropriate size. After the apparent strength values of the cores were corrected for non-standard L/d ratios, this damage theory was used to compute the compressive strength of the concrete in the cores (Table 7). It should be noted that this method produced estimates of the compressive strength at point 3a-9 of 7970 p.s.i. and 8640 p.s.i. with the use of the four-inch and three-inch cores taken from that point in structure. This a difference of only 8%. Variations of that size, or larger, are obtained when testing cylinders cast from a single batch of concrete in a testing machine.

The calibration of the non-destructive tests was carried out with the compressive strength of the concrete in the cores determined as described above. Using in Equation [4] the value of the sonic velocity determined from the in-situ tests at the locations that were cored, an estimate of the compressive strength at each of those points was determined. This estimate is designated f_v. The compressive strength of the cores determined in the laboratory and corrected as previously described is designated f'_c. These two values and their ratios are shown in Table 8, and the frequency distribution for these two variables is shown in Figure 12. The average ratio of f_v/f'_c was 0.94

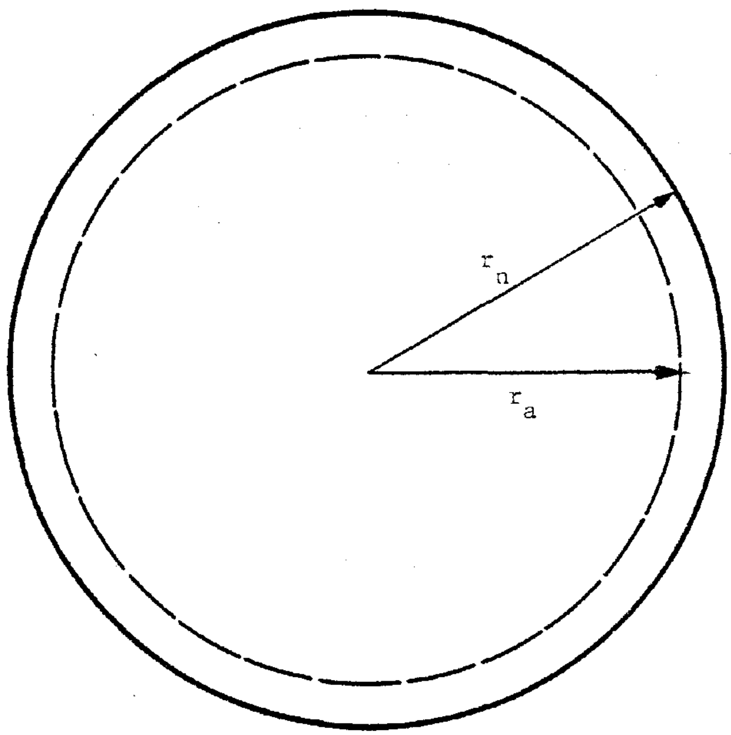


FIGURE 11. DAMAGED CORE

TABLE 7

COMPRESSIVE STRENGTH OF CORES

Point	Uncorrected Strength (p.s.i.)	Corrected for L/d (Kesler's) (p.s.i.)	Corrected for Damage Layer (p.s.i.)
	4 inch diameter cores		
2 B-1	6850	6400	8360
3 A-9	7030	6100	7930
4 C-4	4600	4350	5680
5 E-9	5630	4900	6400
6 H-9	6710	5800	7580
	3 inch diameter cores		
3 A-9	6440	6000	8640
7 H-10	5530	5200	7490
8 H-4	4050	4030	5800
9 B-9	7750	7300	10500
10 C-4	5780	5700	8210
11 H-9	5720	5500	7920

which means that the compressive strength predicted by this method is slightly conservative. Therefore, this method is satisfactory for the determination of a meaningful estimate of the compressive strength in-situ.

As stated earlier, the approximate form of the relationship between rebound readings and the compressive strength of the sample was known. However, the determination of the exact relationship depended on calibration by determining the strength of the cores and comparing the values with the in-situ rebound numbers. Plotting this compressive strength versus the rebound numbers revealed that in fact the points lie very close to the appropriate Soiltest curves. Therefore the Soiltest curves were accepted as the "batch curves" for the concrete in-situ. Using the rebound reading taken on the underside of the slab with the curve for $\alpha = +90$ a predicted value of the compressive strength was generated. Similar use of the curve for $\alpha = -90$ with the rebound readings from the top of the slab gave another prediction of the compressive strength. These two predictions were averaged to obtain f_H which is shown in Table 8 for each of the cores. The ratio of f_H/f'_c as also shown. The average ratio was 1.08. The frequency for f_H is also shwon in Figure 12. As can be seen this value has less spread than either f'_c or f_H, but as stated the average is close to the average of f'_c. Consequently, the Soiltest curves can be used to determine a good approximation of the concrete strength in-situ.

It would appear that both non-destructive tests could be used to determine the compressive strength in-situ. The average of the values

TABLE 8

COMPRESSIVE STRENGTH OF THE CORES BY DIFFERENT METHODS

Point	f'_c	f_v	f_v/f'_c	f_H	f_r/f'_c
		4 inch diameter cores			
2 B-1	8360	7520	0.899	7790	0.931
3 A-9	7970	7380	0.926	7240	0.909
4 C-4	5680	6570	1.160	8100	1.430
5 E-9	6400	7600	1.190	8500	1.330
6 H-9	7580	7610	1.000	8090	1.080
		3 inch diameter cores			
3 A-9	8640	7900	0.913	7250	0.839
7 H-10	7500	4540	0.606	8280	0.110
8 H-4	5800	6110	1.050	8340	1.440
9 B-9	10500	8030	0.765	8260	0.786
10 C-4	8210	6900	0.840	8250	1.010
11 H-9	7920	7530	0.951	8100	1.020
Average	7690	7060	0.937	8017	1.080
Standard Deviation	1380	1010	0.167	422	0.227

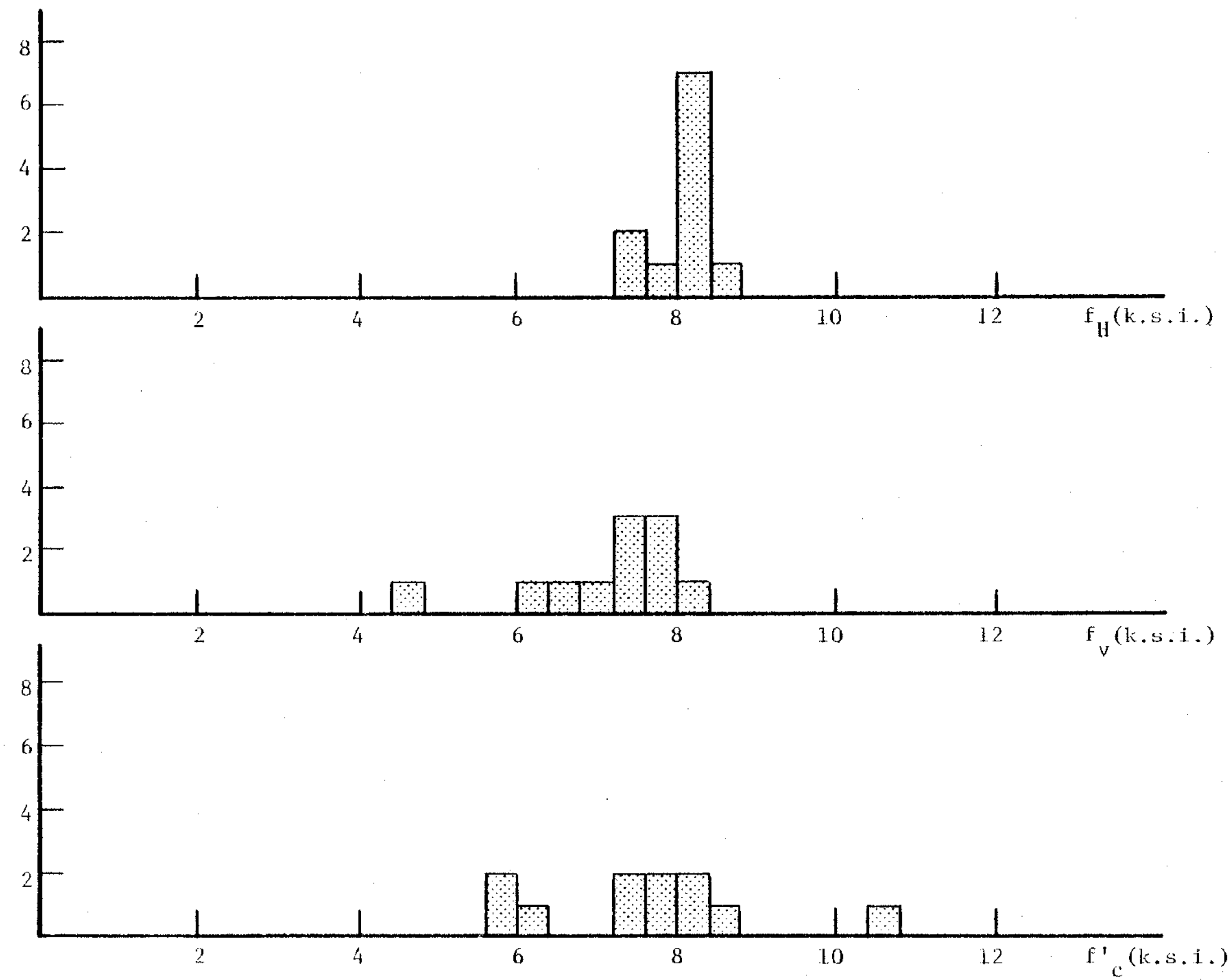


FIGURE 12. FREQUENCY DISTRIBUTION OF f, f_v, AND f'_c FOR THE CORES

predicted by the two tests would be an even better estimate of the compressive strength because it would not be as subject to errors that might occur in the performance of one of the other of the tests. There are problems, however, with this idea. The use of the v-meter does not give the sonic velocity through the slab, but rather the transmit time of the ultrasonic pulse through the slab. At those points that were cored, the distance traveled by the pulse could be accurately measured and the sonic velocity calculated with equal accuracy. However, at the other points in the structure the exact thickness of the slab was not known. As will be shown in a later section, the thickness of the slab varied greatly. Because the relationship is not a linear one, but rather of fourth order, a small error in assumed thickness and thus a small error in assumed sonic velocity results in a much larger error in calculated compressive strength. Examples of these errors are shown in Table 9 and depicted in Figure 13. Since any assumed thickness of the slab, for instance the specified thickness, may easily be in error by 10%, the compressive strength as determined by this method would be unacceptably in error. Therefore, the v-meter test results could not be used throughout the structure to determine the compressive strength. Fortunately, the concrete test hammer test data does not suffer from a similar problem. The compressive strength of the concrete in the structure can be determined from the rebound readings.

Using the rebound readings from the top and bottom of the slab, two estimates of the compressive strength of the concrete at each of the 99 points in-situ were generated. The two estimates were averaged

TABLE 9

RELATED ERRORS IN VELOCITY AND COMPRESSIVE STRENGTH

Percentage Error Velocity	Percentage Error Compressive Strength
0.2	1.0
0.5	2.0
1.0	4.1
1.2	5.0
2.0	8.2
2.4	10.0
4.7	20.0
5.0	21.6
10.0	46.4
10.7	50.0
18.9	100.0
20.0	207.0
50.0	506.0
100.0	1600.0

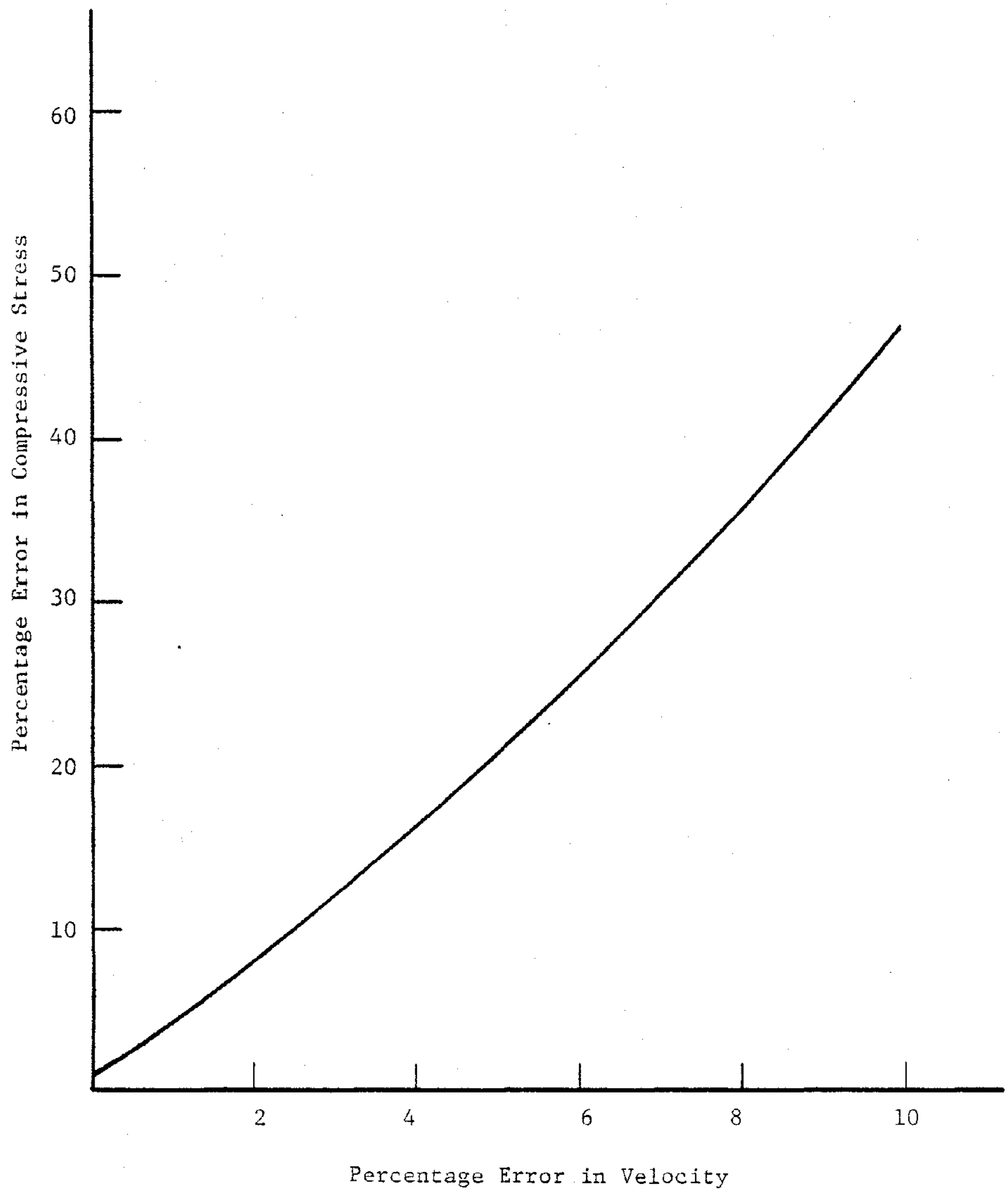


FIGURE 13. RELATED ERRORS IN VELOCITY AND COMPRESSIVE STRENGTH

to give one value at each point. The distribution of the resulting values of the concrete compressive strength is shown in Figure 14. It appears that a normal distribution would fit the distribution well. The average value predicted was 8050 p.s.i. with a standard deviation of 500 p.s.i. The range of predicted values was from 6280 to 9190 p.s.i. There was no one floor that was of greatly different strength than the others. The average of the concrete strengths of each floor ranged only from 7820 to 8430 p.s.i. Data from cylinders cast during the construction of Phase 2 of the original building project, which included the structure tested in this investigation, showed that the average 28 day strength of the concrete was 3780 p.s.i. The design specified a 28 day strength of 3000 p.s.i. The age of the structure at the time of these tests was 22 years. The ratio of the average strength at the time of the tests to the average 28 day strength was 2.13. Washa and Wendt (24) have investigated long term strength gain of concrete and reported an average strength ratio of 2.39 when comparing the strength at age 25 years to the strength at age 28 days. The result obtained from this investigation is in excellent agreement with their results.

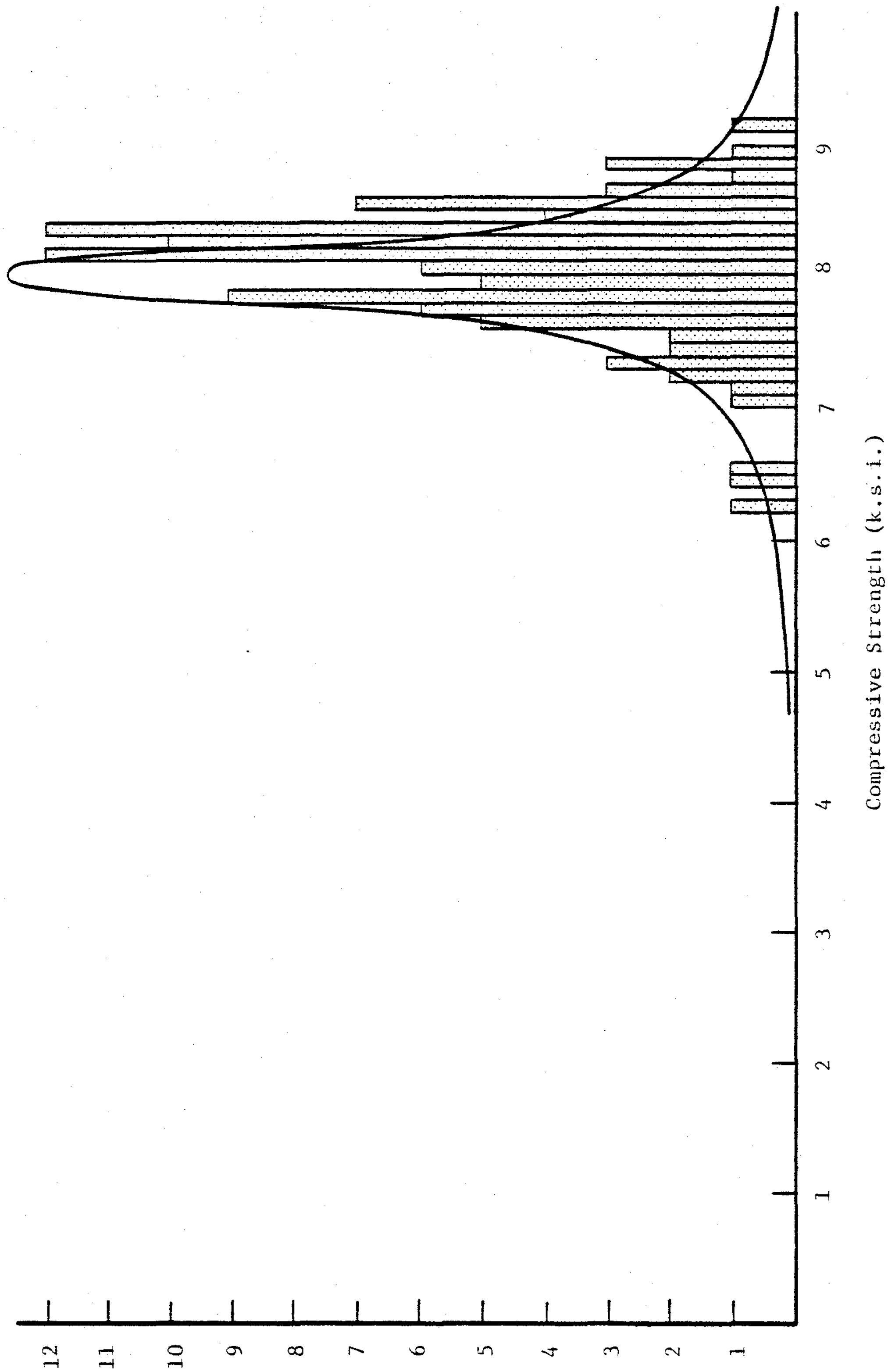


FIGURE 14. FREQUENCY DISTRIBUTION OF COMPRESSIVE STRENGTH IN-SITU

3. DIMENSIONAL VARIATIONS

An important goal of this investigation was to gain information concerning the variation of actual structure dimensions from their specified values. These variations are the subject of this chapter. The first section of this chapter is concerned with the variation of overall dimensions. These include centerline-to-centerline distances between columns and floor-to-floor heights. The second part of this chapter deals with variations of dimensions relative to the slabs in the structure. Information is presented concerning the thickness of the slabs as well as the reinforcement spacing and coverage. The final section of this chapter details variations in the dimensions of the beams and columns in-situ, including cross-sectional variations and variations of the concrete cover over the reinforcement.

3.1 OVERALL DIMENSIONS

As stated in the introduction, comparison of overall dimensions in-situ included a study of the plan dimensions of the floors in the structure, such as the centerline-to-centerline dimensions between the columns, and an investigation of the floor-to-floor heights.

Information concerning plan dimensions was to be gathered by measuring dimensions on several of the floors. To facilitate these measurements, the interior partitions had to be removed. Since this removal had already been effected, for other reasons (1), on the first and eleventh floors, these two floors were selected for measurement. The measurements were made with a 100 ft. steel tape to an accuracy of $\pm$ 1/8 inch. Table 10 lists the specified dimension, L_s, and the measured dimension, L_m, from the two floors, where the notation refers

TABLE 10

In-Situ Measured Floor Dimensions

		First Floor			Eleventh Floor		
Dimension	L_s	L_m	L_m/L_s (in)	L_m-L_s	L_m	L_m/L_s	L_m-L_s (in)
A	15'-7 1/4"	15'-7 3/8"	1.001	0.125	15'-5 1/2"	0.991	-1.75
B	8'-11 1/2"	9'-1/4"	1.007	0.75	9'-0"	1.005	0.5
C	15'-7 1/4"	15'-7 1/4"	1.000	0.0	15'-9"	1.009	1.75
D	19'-9 1/4"	19'-9 1/2"	1.001	0.25	19'-9 1/2"	1.001	0.25
E	24'-6 1/4"	24'-6 1/2"	1.001	0.25	24'-6 1/2"	1.001	0.25
F	19'-9 1/4"	19'-9 1/2"	1.001	0.25	19'-9 1/4"	1.000	0.0
G	24'-6 1/4"	24'-6 1/2"	1.001	0.25	24'-6 3/4"	1.002	0.5
H	16'-9 1/2"	16'-9 1/2"	1.000	0.0	16'-9 1/2"	1.000	0.0
I	15'-7 1/2"	15'-7 1/2"	1.000	0.0	15'-8"	1.0003	0.5
J	16'-6 1/4"	16'-6 1/4"	1.000	0.0	16'-7 1/2"	1.006	1.25
K	7'-1 1/2"	7'-1 1/2"	1.000	0.0	7'-1 1/2"	1.000	0.0
L	16'-6 1/4"	16'-6 1/4"	1.000	0.0	16'-5"	0.994	-1.25
M	20'-1"	20'-5/8"	0.998	-0.375	20'-1"	1.000	0.0
N	20'-1"	20'-7/8"	0.999	-0.125	20'-1"	1.000	0.0
	Mean		1.0006	0.098		1.0021	0.143

to Figure 15. The ratio of L_m/L_s as well as the difference between the two measurements is also given in Table 9. The average ratio was 1.001 which corresponds to an average variation of 0.14 inch. More important is the mean of the absolute value of the variations which was 0.4 inch. The maximum variation was $\pm$ 1.75 inch. Birkeland and Westhoff (7) have studied dimensional variations in many concrete structures. They state that the usual variation in spacing between parallel column lines is $\pm$ 1 inch with maximum variation of $\pm$ 2 inches. Their usual variation is twice as large as that found in this investigation; however, the maximum variation found is well described by their findings.

The measurement of the floor-to-floor heights were made using the steel tape in the open area of the stairwell. The specified height of each story was 8'-6" (102 inches). The measurements obtained are shown in Table 11 along with the difference between the measured and specified heights. The average difference from the specified dimension was + 0.15 inch and the maximum variation was $\pm$ 1 inch.

3.2 SLAB DIMENSIONS

In addition to the spans of the slabs, which were described in the last section, several other slab related dimensions were investigated to determine the variations of the in-situ dimensions from the specified values. Those dimensions that were studied were the thickness of the slab, the spacing between the reinforcement, and the concrete cover over the reinforcement.

The thickness of the slab could have been determined by drilling a hole through it and measuring the thickness of the slab at the hole.

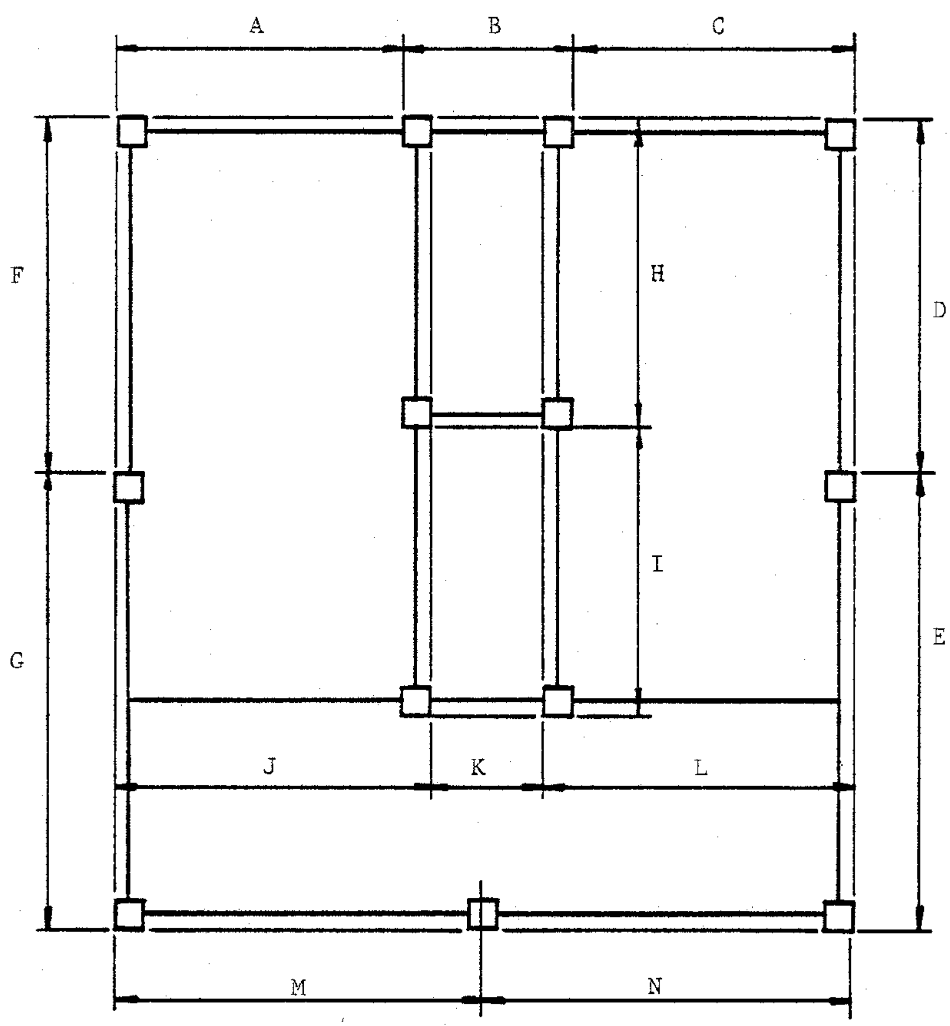


FIGURE 15. STANDARD FLOOR WITH LABELED DIMENSIONS

TABLE 11

FLOOR-TO-FLOOR HEIGHTS

Story	Specified Height (in.)	Measured Height (in.)	Δ (in.)
10	102.0	102.5	0.5
9	102.0	102.0	0.0
8	102.0	102.25	0.25
7	102.0	102.0	0.0
6	102.0	102.0	0.0
5	102.0	102.0	0.0
4	102.0	103.0	1.0
3	102.0	101.0	-1.0
2	102.0	102.25	0.25
1	102.0	102.5	0.5
		Average	0.15

However, this method is expensive and time-consuming. Therefore, it was desired to use a non-destructive test that would give a reliable measure of the thickness of the slab. As previously described in the section on the strength of the concrete, there is a relationship between the sonic velocity through concrete and the compressive strength of the concrete. Equation [4] can be rewritten

$$V = [(1.3138 \times 10^{17})\ f'_c]^{0.25} . \qquad [5]$$

The v-meter displays the time, t_a, required for a sonic pulse to traverse the slab. The thickness of the slab can then be computed by

$$\text{thickness} = V \times t_a \qquad [6]$$

or, substituting in for the velocity from Equation [5]

$$\text{thickness} = t_a\ [(1.3138 \times 10^{17})\ f'_c]^{0.25} . \qquad [7]$$

The veracity and utility of this equation can be determined by comparing the predicted thickness at the points that were cored to the measured thickness at those points. This comparison is shown in Table 12. This method for determining the slab thickness works well, as can be seen by the fact that the average error of the predicted thickness at those points was only 1.5% or 0.07 inch. In fact the maximum detected error was only 4% inch, which is about 0.2 inch. Therefore, a great deal of confidence can be placed in the value of the thickness of the slab determined by this method. Using Equation [7] with the strength of the concrete at each of the 99 points which was determined as previously described and the transit time measured by the v-meter, the

TABLE 12

Comparison of height of cores to predicted height

P+	Predicted thickness (in.)	Actual thickness (in.)	Error p-a (in.)	Percentage error
2B-1	4.924	4.875	0.0490	0.010
3A-9	3.672	3.6875	-0.0155	-0.004
4C-4	5.532	5.3125	0.2195	0.041
5E-9	4.119	4.0000	0.119	0.030
6H-9	4.192	4.125	0.0670	0.016
7H-10	4.001	4.000	0.001	0.000
8H-4	5.098	5.000	0.098	0.020
9B-9	3.906	3.875	0.031	0.008
10C-4	5.431	5.375	0.056	0.010
11H-9	4.012	3.9375	0.0745	0.019
		Mean	0.0700	0.015
		Standard Deviation	0.0665	0.013

thickness of the slab at each of those points was calculated. The thicknesses computed were averaged to determine the mean thickness of the five slabs on each floor (Figure 16). The specified thickness of Slabs A, B, C, and D was 5 inches and of Slab E was 4 inches. The average thickness of each slab on each floor is given in Table 13, where the n value is the number of points in that region for which a thickness was calculated. Several interesting items can be noted from this table. 1) Most slabs exceeded the specified thickness. In fact, only rarely was a slab thinner than specified. The worst case detected of this kind was Slab D on the ninth floor where the actual thickness was 3/8 inch less than the specified dimension. This greatest error was on a variation of - 8%. 2) The thickness of Slab C exceeded the specified thickness of 5 inches by more than 0.5 inch (10%) on four of the six floors on which any point in that region was investigated. Similarly, the thickness of Slab B exceeded 5.5 inches on two of the four floors on which data was taken from that region of the slab. 3) All the specified 5-inch slabs on the fourth floor exceeded 5.5 inches in thickness. The average, standard deviation and range of values determined for both the specified 4-inch and 5-inch slabs are given in Table 14. As can be seen the thinner slabs in-situ were much nearer the specified dimension than were the 5-inch slabs. However, the results obtained are not consistent with those presented by Mirza and MacGregor (3). As determined in this investigation, they report that the average measured slab thickness is slightly greater than the specified thickness. However, the average variation that they report is 0.04 inch and the standard deviation is 0.46 inch, whereas the mean variation of the 5-inch slabs in this investigation 0.33 inch and the

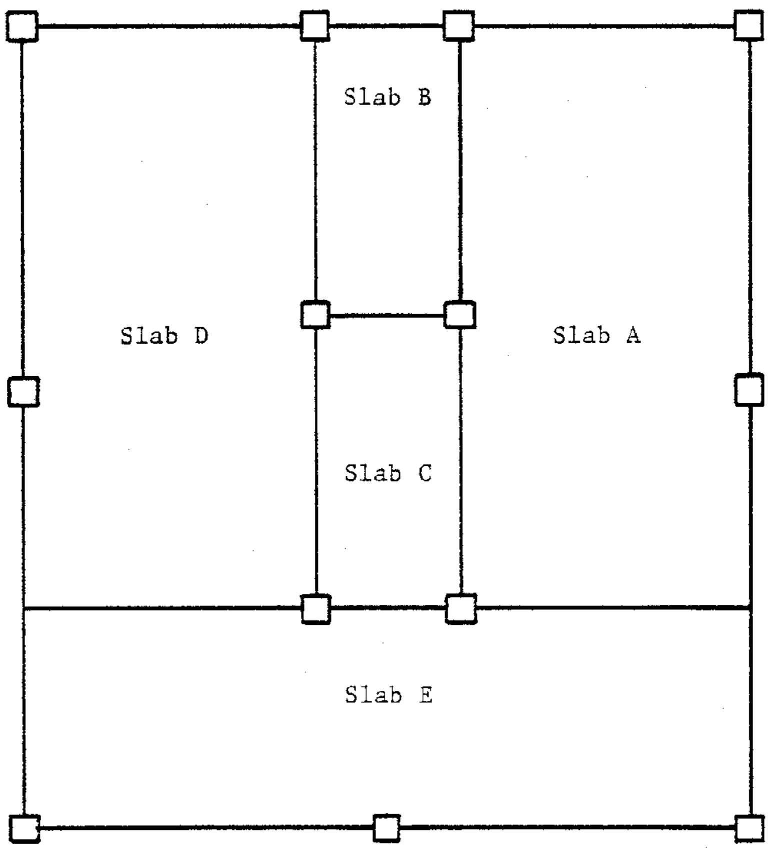


FIGURE 16. STANDARD FLOOR DIVIDED INTO FIVE REGIONS

TABLE 13

Thickness of Various Slabs In-Situ

Floor	SLAB A		SLAB B		SLAB C		SLAB D		SLAB E	
	# of Points	Thickness (in.)	# of Points	Thickness (in.)	# of Points	Thickness (in.)	# of Points	Thickness (in.)	# of Points	Thickness (in.)
2	3	5.130	0	-----	1	5.072	2	5.016	5	4.043
3	1	4.997	0	-----	1	5.632	3	5.046	2	3.745
4	6	5.523	1	5.897	1	5.540	4	5.640	3	3.977
5	4	5.092	0	-----	1	5.872	2	5.495	2	3.923
6	0	-----	1	5.571	1	5.211	0	-----	5	4.196
7	2	5.308	0	-----	0	-----	4	5.782	4	4.315
8	3	5.199	0	-----	0	-----	3	5.085	2	3.993
9	4	5.238	0	-----	0	-----	1	4.625	1	3.906
10	4	5.134	1	5.002	1	5.844	4	5.103	2	3.987
11	5	5.689	1	5.184	0	-----	2	5.254	3	4.348
Mean		5.260		5.410		5.530		5.230		4.043
Standard Deviation		0.220		0.40		0.330		0.360		0.189

TABLE 14

Average Thickness of Slabs in Structure

Slab Designation	Nominal Thickness (in.)	Actual Mean (in.)	Standard Deviation (in.)	Range (in.)
A,B,C or D	5.0	5.331	0.330	4.625-5.897
E	4.0	4.043	0.189	3.745-4.348

standard deviation is 0.33 inch. On the contrary, the mean variation of the 4-inch slabs, 0.043 inch, was nearly equal to the value given by Mirza and MacGregor, but the standard deviation, 0.19 inch, is significantly less than the value that they reported.

The second slab-related dimension that was of interest was the lateral spacing between reinforcing steel in the slab, which included the spacing between adjacent bars of positive moment steel, the spacing between bars of negative moment steel, and the spacing between the temperature reinforcing bars. These measurements were made with a magnetic inductance meter also known as a pachometer (Figure 17). The location of reinforcing bars is accomplished by moving the probe along the face of the concrete. The meter pointer will indicate a maximum deflection when the axis of the probe is parallel to and directly over the axis of a reinforcing bar (Figure 18). In-situ the procedure used was to watch the meter pointer pass through a maximum deflection, reverse the direction of probe movement, stop at the maximum deflection, and mark the axis of the bar on the concrete. This process was carried out with an accuracy of $\pm$ 1/4 inch, as determined by practice on samples in the laboratory. Using this procedure, all the reinforcement in a three-foot by three-foot region around each of the 99 points was studied in-situ. The resulting "maps" of the reinforcement were used to pick the exact position at a point at which to drill the cores in order to avoid interference by the reinforcement. Reinforcing "maps" are shown in Figures 19 and 20 for points 8 H-4 and 10 G-1, respectively. The specified spacing at these two points is exactly the same but, as can be seen, the spacing in-situ is quite different.

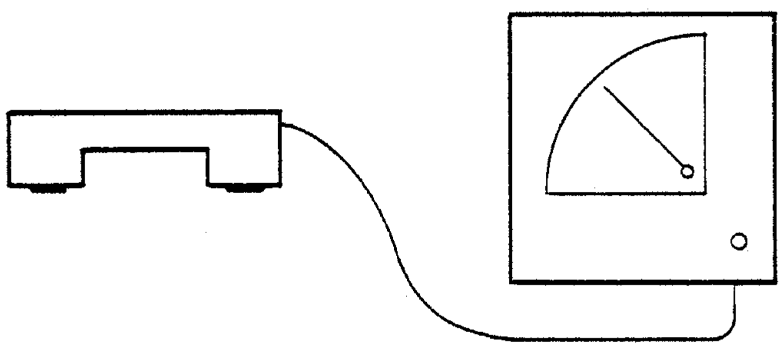


FIGURE 17. PACHOMETER

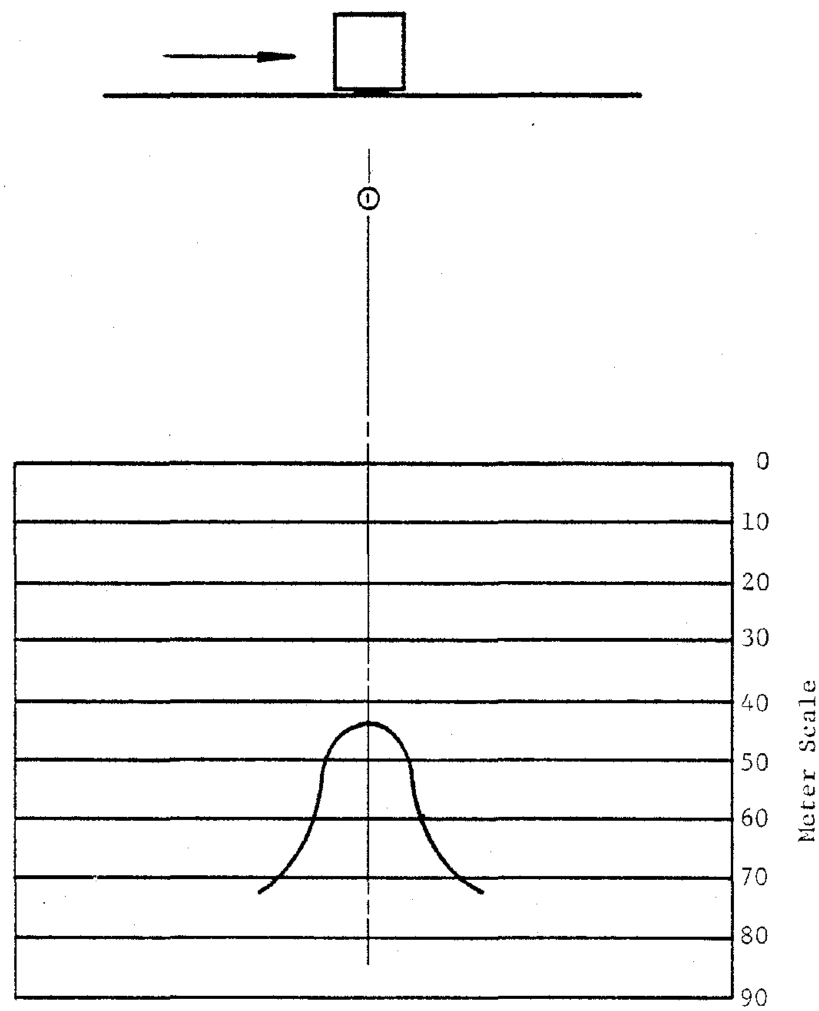


FIGURE 18. PACHOMETER PASSING OVER A BAR

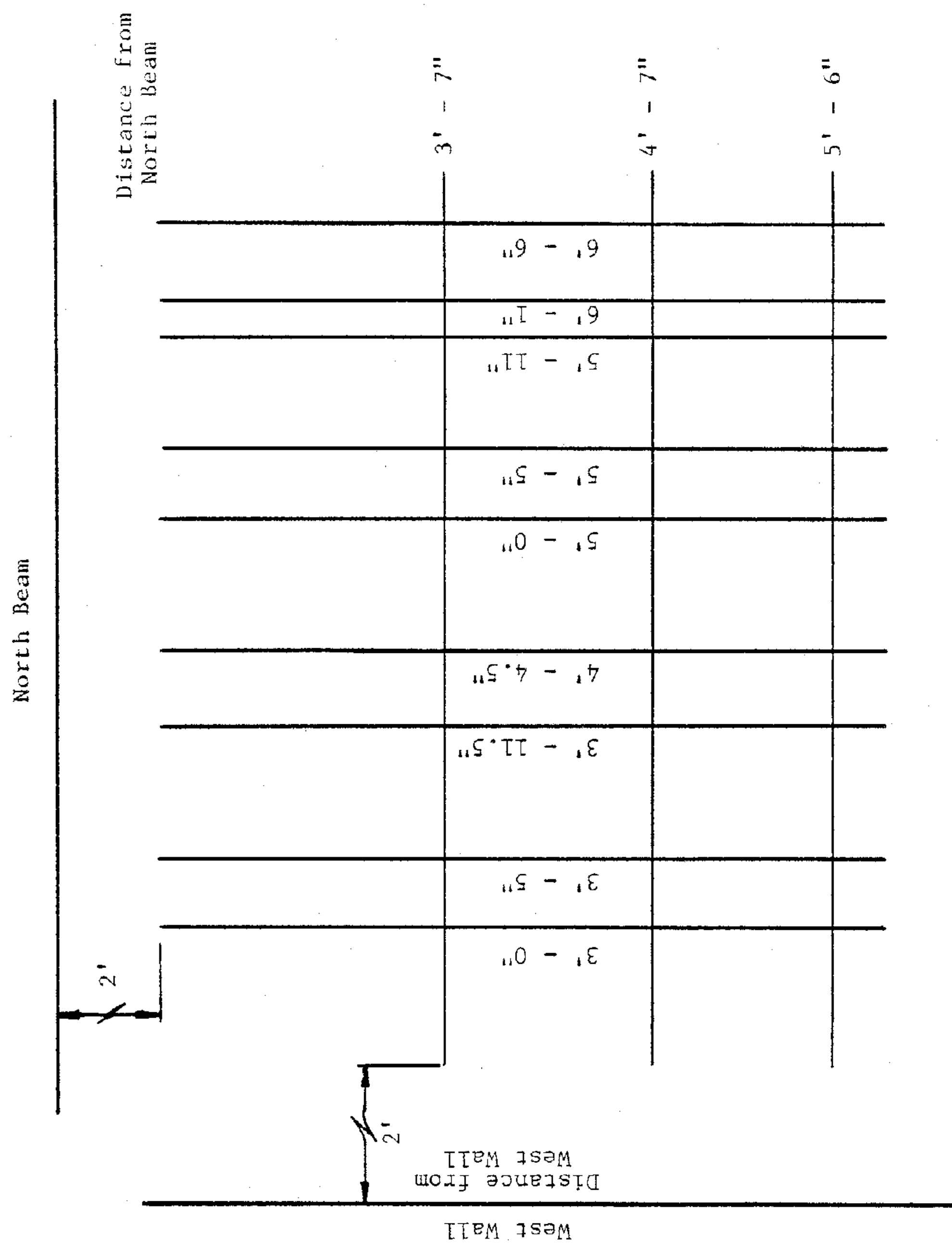


FIGURE 19. REINFORCEMENT AT POINT 8 H-4

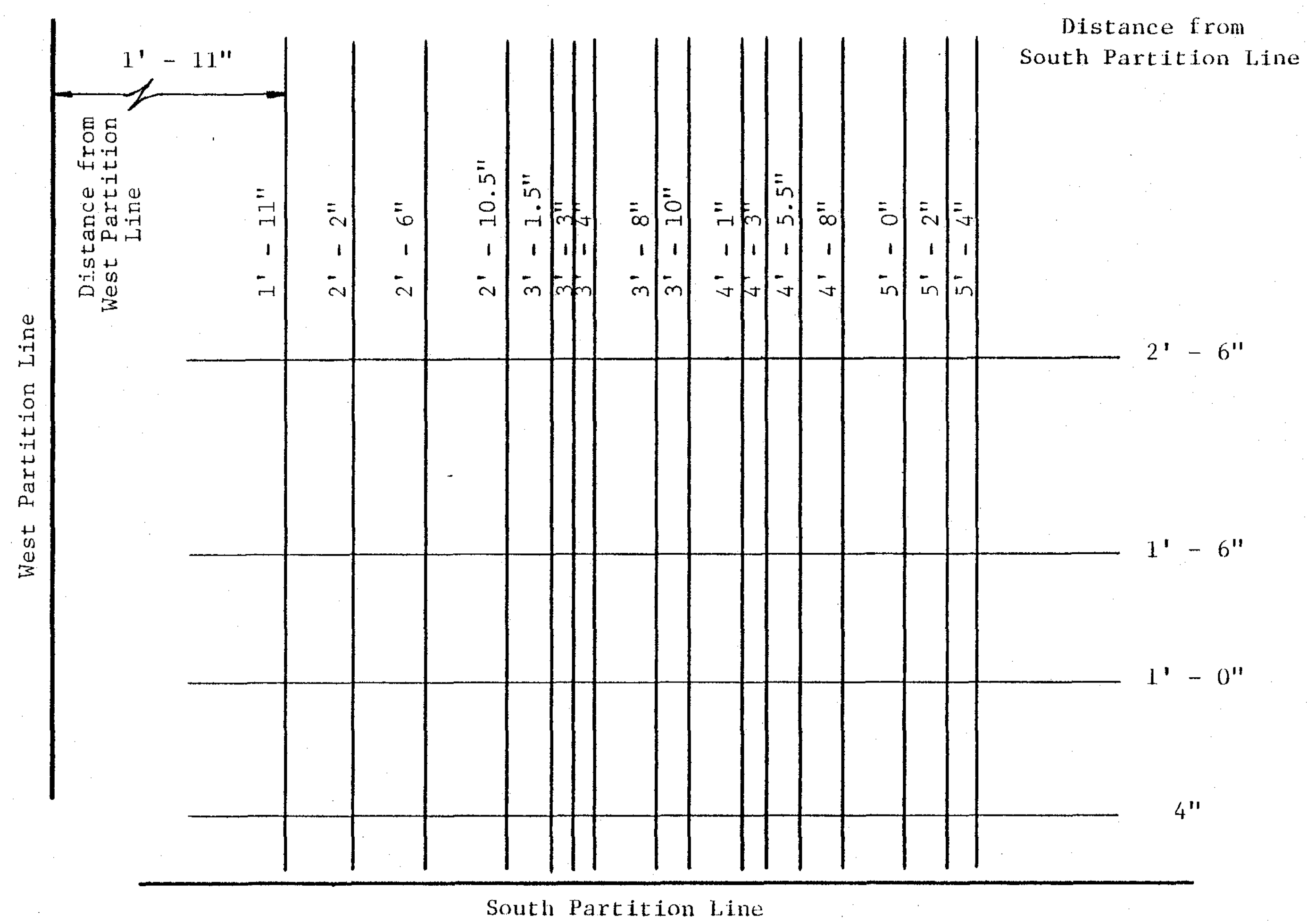


FIGURE 20. REINFORCEMENT AT POINT 10 G-1

This extreme variability of spacing was typical of the results obtained throughout the structure. Table 15 contains the results of averaging the spacings measured in-situ for the primary, secondary and temperature steel as well as the specified values of these dimensions.

No general statement can be made regarding the average spacing of the reinforcement. The spacing in-situ of the positive moment steel, negative moment steel at the edge beams, and transverse steel is approximately two-thirds the specified spacing. However, the spacing of the negative moment steel at the interior beams is slightly larger than the specified value. This inconsistency is inexplicable. The large standard deviation of the negative moment steel at the exterior beams, 2.9 inches, is also disturbing. The remainder of the data is much better balanced.

The final slab-related dimension of interest was the concrete cover over the reinforcement. The measurements of concrete cover were also made with the pachometer. The deflection field of the meter indicator is divided into 90 divisions. A set of curves relating the meter deflection to bar size and concrete cover is given by the manufacturer (Figure 21). With these curves, the concrete cover can be determined if the size of the bar and the maximum meter indicator deflection are known. If the depth of cover and the maximum deflection are known, the size of the bar can be determined. However, neither of these approaches is practical in-situ because both the size of the bar and the depth of cover are unknown. In this case another procedure must be followed. This method consists of recording the maximum indicator deflection on each of several passes of the probe over the

TABLE 15

Comparison of Specified and Actual Spacing
Between Reinforcing Bars

Spacing Between	Specified Mean (in.)	Actual Mean (in.)	Standard Deviation (in.)
Principal Reinforcement	6	3.77	0.95
Principal Reinforcement	8	4.26	1.19
Negative Moment Reinforcement	6	6.25	0.85
Transverse Reinforcement	12	7.98	4.10

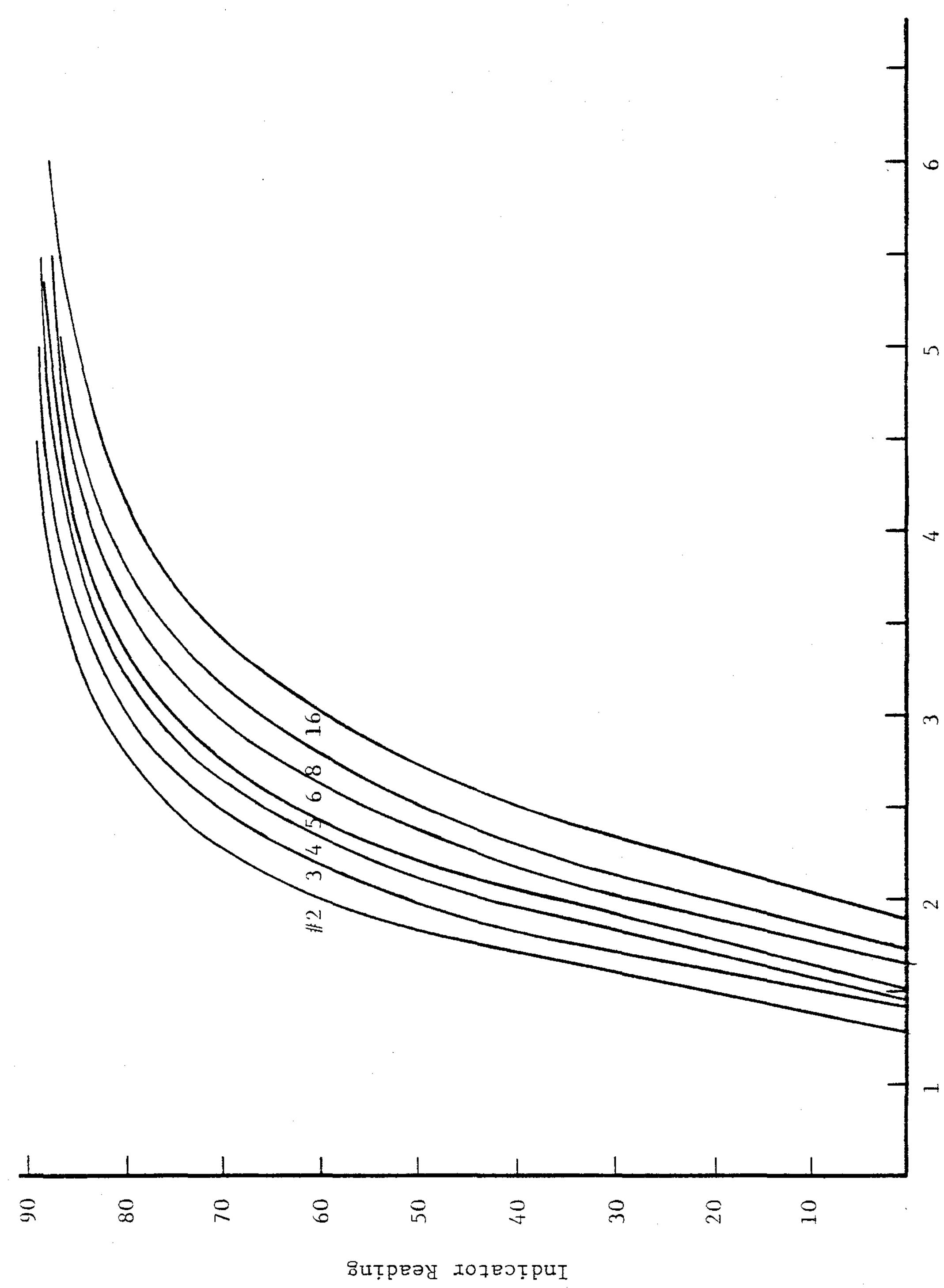


FIGURE 21. PACHOMETER CALIBRATION CURVES

bar. The first pass is made with the probe on the face of the concrete and the remainder with various known-thickness non-metallic shims beneath the probe. By comparing the differences in predicted coverage for different size bars with the known change in spacing between the probe and the reinforcing bar, the shim size, the correct bar size can be determined. This procedure is well outlined in the operating manual (25). This method and the manufacturer's curves were accepted after practice on samples in the laboratory showed that the correct bar size and depth could be determined in over 90% of the cases.

At each of the 99 points studied in the structure, one positive moment bar was analyzed for depth of cover by the above method. In addition, at all points that contained negative moment steel the cover over one of those bars was also studied. The average concrete cover determined in-situ, as well as the standard deviation, for both the top and bottom steel are given in Table 16. As is also shown there, the specified cover, by both the building plans and the A.C.I. Code (318-56), was 0.75 inch. The average variation from this value was + 0.25 inch for the negative moment steel bars and - 0.05 inch for the positive moment steel bars, both of which are within the tolerance set by the A.C.I. Committee 301 of $\pm$ 0.5 inch. The differences in concrete cover from the specified values are the type that would be expected after observing that the workmen walk on the steel after it has been placed, thus increasing the cover over the top and decreasing the cover for the bottom bars. As also might be anticipated, the effects of this are more strongly seen on the top steel. The data given by Mirza and MacGregor (3) appears to follow a similar trend. They state, "the mean deviation of the top reinforce from the nominal

TABLE 16

Comparison of Specified and Actual
Concrete Cover Over Reinforcing Bars

Cover for	Specified Cover (in.)	Mean Actual Cover (in.)	Standard Deviation (in.)
Top Bars	0.75	0.997	0.257
Bottom Bars	0.75	0.699	0.233

effective depth is approximately 2 1/2 times the mean deviation of the bottom steel, but the standard deviations of both steels are about the same." In this investigation, the standard deviations of both steels were nearly equal but, the variation of the effective depth of the top steel from the specified value is five times larger than the same variation for the bottom steel. In addition, the variations in this investigation were less than one-half of the variations presented by Mirza and MacGregor.

3.3 BEAM AND COLUMN DIMENSIONS

The final objective of the structural survey was to gain information concerning the columns and beams in-situ. For this purpose a sample of three beams, or one beam at three points along its length, and three columns on each floor was chosen for study. Each element was first measured to an accuracy of $\pm$ 1/8 inch and then the concrete cover of the reinforcement of each element was examined using the pachometer.

The outside dimensions of the elements measured proved to be equal to the specified dimension in the vast majority of the cases studied. For instance, in the large number of columns investigated, in no instance did any side exceed or fall short of the design dimension by as much as 1/8 inch. The widths of the beams investigated also were remarkably exact in dimension. In only two cases out of the twenty-four examined was the width 1/8 inch greater than the specified dimension. All others measured exactly as specified. The depth of the beams could not be measured (Figure 22). Instead the depth of the beam projecting below the slab, L_1, was measured and the thickness of the slab as determined by the use of the v-meter was added to it. Using this technique it was seen that the depth of the beam was not as well controlled as the width. On the basis of the 24 beams examined, the actual depth of the beams exceeded the specified depth by an average of 0.81 inch with a coefficient of variation of 0.55 inch. The major portion of this large increase in depth comes from the increased slab thickness over the beam. The average increase in L_1 is only 0.3 inch. Even this variation is larger than reported by other investigators. Mirza and MacGregor (3) report an average variation in beam depth of only 0.11 inch. On the contrary, the data concerning beam widths and column dimensions is in good agreement with that presented by others. Several sources (3,5,18,20) report generally less than 1% variance in column size. Drysdale (8) found a maximum variation of 0.25 inch in column size. Mirza and MacGregor report an average variation of 0.1 inch in beam width. All of which agree with the in-situ measurements in this investigation.

After the structural elements were measured, the pachometer was used to determine the concrete cover over the reinforcement. On the beams, the measurements were made from above of the slab and from below of the bottom of the beam. The columns (Figure 23) were analyzed on all accessible sides (four sides for interior columns, but only three sides for exterior columns except corner columns where only two sides were measured). Thus, there were two methods of measuring the concrete cover, C_c, on the column steel. One method was locating the center of the bar from the maximum needle deflection while passing along side AB and the second was by calculating the cover depth from repeated readings on side BD. The dimensions determined by the two methods rarely

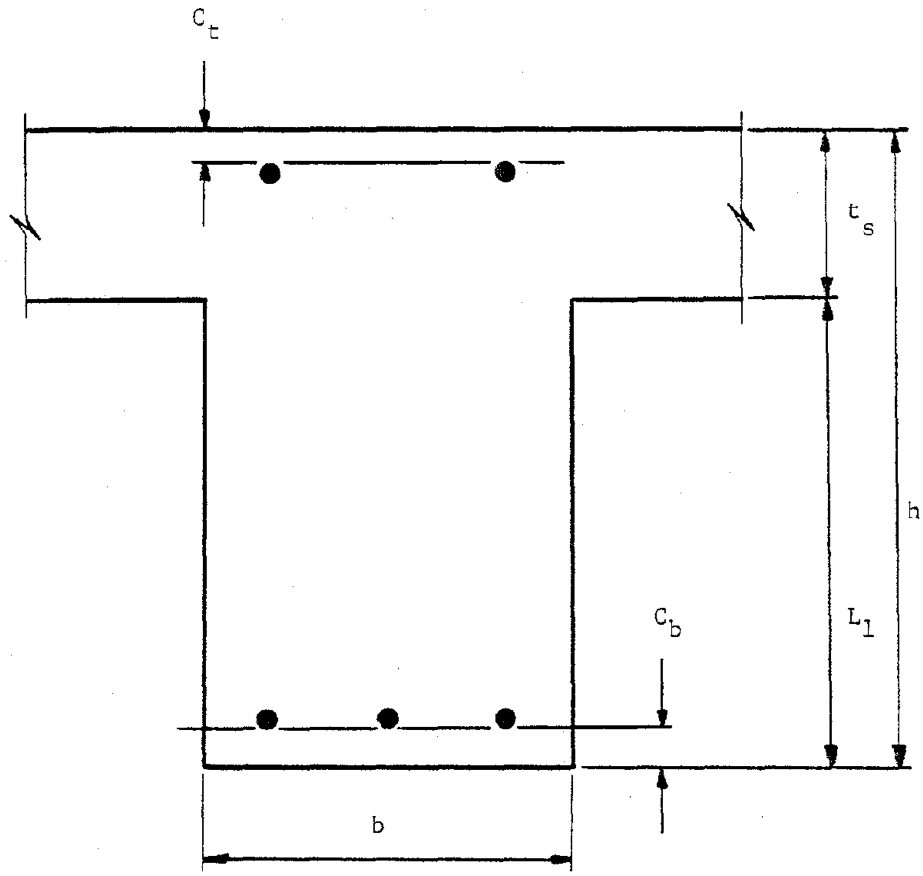


FIGURE 22. TYPICAL BEAM

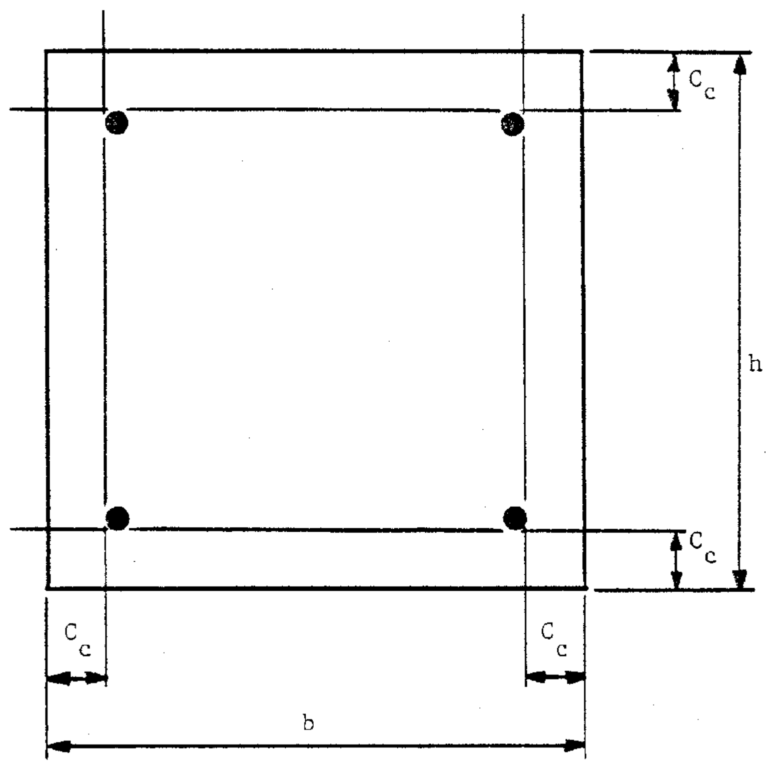


FIGURE 23. TYPICAL COLUMN

differed; but when they did, the determination of coverage by the repeated readings on side BD was used because it was felt that that method was the more exact of the two. For both beams and the columns the location of the stirrups and the ties were determined in order to avoid measurement directly over them. Spiral ties could not be avoided in this manner and therefore spiral tied columns had to be excluded from this investigation. All columns below the sixth floor had spiral ties, thus only columns above the sixth floor were analyzed. The specified cover for the principal reinforcement in the beams was 1 1/2 inch and for the principal reinforcement in the columns was 2 inches. With the measured concrete cover minus the specified cover defined as Δ, the results of the survey of the beam and column steel are shown in Table 17. The top beam steel has slightly more cover than specified, while the bottom steel has significantly less. If not for the large increase in beam depth the top steel cover would have been significantly less than specified. The variations of cover over the beam and column steel in this investigation are much larger than those presented by Mirza and MacGregor. This may be due to very close tolerances on the project studied by them. The value r may be defined as the ratio of the measured concrete cover to the specified cover. The mean value for the column steel was $r = 1.11$ with the standard deviation of the ratio r, also given in Table 16, are less than those presented by Drysdale (8). This may be due to the fact that the sample in this investigation is not as varied as that studied by Drysdale.

TABLE 17

Results of Survey of Beam and Column Steel

Concrete Cover for	Symbol	Δ (in.)	Standard Deviation Δ (in.)	γ	Standard Deviation γ
Top Beam Steel	C_T	0.033	0.347	1.02	0.230
Bottom Beam Steel	C_B	-0.355	0.278	0.76	0.190
Average		-0.201	0.347	0.87	0.230
Column Steel	C_C	0.210	0.760	1.110	0.380

4. EFFECTS OF VARIATIONS ON STRENGTH

The variations in material strengths and structural dimensions from their specified values are not, in and of themselves, very informative or important. What is informative, or important, is the effect that these variations have on the strength of the structure. In this chapter is reported the analysis of the flexural strength of the slabs and beams, and the combined axial-flexural strength of the columns, based on the member properties and dimensions in-situ. In addition, comparison is made between these results and the strengths of the members with properties and dimensions as specified. The ratio of the strength in-situ to the strength of the specified number is compared to the capacity reduction factors specified by the A.C.I. Code [318-77] (2).

Rather than compute the strength of each member that was studied in the structure, a range of probable strengths was computed by using the statistical data determined for each variable investigated. A summary of this information, which includes the in-situ mean and standard deviation as well as the specified value of each variable, is given in Table 18. Advantage was made of the fact that each variable could be represented by a normal distribution. The probability that a value of a variable which can be represented by a normal distribution will differ from the mean, $\bar{x}$, by more than two standard deviations, 2s, is only five percent. The values $\bar{x}$ - 2s and $\bar{x}$ + 2s are thus reasonable upper and lower limits for the value of the variable. These reasonable extremes were computed for all of the variables. Since each of the variables considered has an effect on the strength of a section, an

TABLE 18

Summary of In-Situ Variations

Variable	Specified Value	In-Situ Mean	In-Situ Standard Deviation
Slab steel yield strength	50 ksi	60 ksi	2.5 ksi
Beam & column steel yield strength	50 ksi	55 ksi	2.5 ksi
28 day concrete strength	3 ksi	3.7 ksi	0.1 ksi
Concrete strength at time of test	---	8.0 ksi	0.5 ksi
Slab thickness	5.0 in.	5.3 in.	0.3 in.
Slab thickness	4.0 in.	4.1 in.	0.2 in.
Average slab steel spacing			
Positive steel	6.0 in.	4.0 in.	1.0 in.
Positive steel	8.0 in.	4.5 in.	1.2 in.
Negative steel	6.0 in.	6.25 in.	0.85 in.
	12.0 in.	8.70	2.9
Concrete cover over slab steel			
Positive steel	0.75 in.	0.70 in.	0.25 in.
Negative steel	0.75 in.	1.00 in.	0.25 in.
Beam width	12 in.	12 in.	0.0 in.
Beam depth	18.375 in.	19.125 in.	0.3 in.
Concrete cover over beam steel			
Top steel	1.5 in.	1.5 in.	0.35 in.
Bottom steel	1.5 in.	1.15 in.	0.30 in.
Column exterior dimensions	Variable	Equal to Specified Values	0.0 in.
Concrete cover over column steel	2.0 in.	2.2 in	0.75 in.

arbitrary perturbation of the value of a variable results in either an increase or a decrease in the strength of the section. An extreme variation, $\bar{x} \pm 2s$, of a variable will then cause an extreme increase or decrease of the strength of the section. If the limit which causes an increase in the strength of the section were considered to occur simultaneously for each of the n variables that affect the strength, the resulting strength would be the largest possible, given the limits imposed on the value of the variables. This combination shall hereafter be referred to as the largest strength. A similar combination of the opposite extremes gives the smallest strength given the limits on the value of the variables. This combination shall hereafter be known as the smallest strength. There exists a possibility that the strength may be as large or larger than the largest strength or as small or smaller than the smallest strength but, given the assumption of normality, the probability of either of these occurrences is only 0.025^n. As the number of variables considered increases this probability quickly approaches zero. The highest and lowest strengths are not then of practical importance normally, but rather are important as limits of the range of the strength of the section. Conversely, the mean strength of the section, the strength of the section that has each of the variables equal to the mean value determined from the in-situ investigations, is of great practical importance as it represents the average strength of the section in the structure. The mean strength, however, does not convey in any manner the range of the strength in-situ. Thus despite the infinite possible combinations of the variables, the computation of just three strengths, the highest, mean and the lowest strengths, conveys most of the necessary information concerning the range of strength of a section of a structure.

The variations that have been studied can also be divided into two categories. The first is that type of variation that occurs when the member as initially constructed and is present throughout the service life of the member. This type of variation is time-independent. It is known as an initial variation. The only variable studied that was not of this type was the strength of the concrete. Because this variation is time-independent it is called the temporal variation. Since there are two types of variations, there are also two mean strengths for each member. These are the initial mean strength, which considers only the initial variations, and the temporal mean strength, which considers the concrete strength at the time of the test, rather than the 28-day strength, as well as the other initial variations. Similarly there is an initial smallest strength, temporal smallest strength, initial largest strength, and temporal largest strength.

The specified strength is defined as the strength of the section when each of the variables in Table 18 has the specified value. The ratio of the strength of a section to the specified strength is defined to be R_m. The range of values of R_m for a section was calculated by dividing the smallest, mean, and largest strengths by the specified strength of the section. The minimum value of R_m can be compared with the appropriate capacity reduction factor.

4.1 STRENGTH OF SLABS AND BEAMS

As already stated, rather than compute the strength of each slab and beam investigated, a range of probable strengths for representative sections was computed by using the data determined from the examinations of the structure. To investigate the effects of the variations on the strength of the slabs, five representative sections were analyzed. These sections included the positive and negative moment sections of a four-inch slab, and the positive and two negative moment sections, one at the edge beams and the other at the interior beams, of a five-inch slab. The strengths of each of these sections, including the specified strength, and both the initial and the temporal smallest, mean, and largest strengths, as well as the values of R_m are shown in Table 19 where the dimensions are as shown in Figure 23. The minimum value of R_m for the slabs was 0.61 for the initial strength of the negative moment section of the four-inch slab. This is much less than the capacity reduction factor of 0.90 specified by the A.C.I. Code [318-77] (2); however, the probability of this smallest strength occurring is only $(0.025)^5$ or 9.8×10^{-9} which is practically zero.

In general, for four out of the five sections studied, the mean strengths, both initial and temporal, have a value of R_m greater than 1.50. This fifty percent or greater increase in strength over the specified value is primarily due to the increase in reinforcement ratio which is caused by the decreased spacing of the reinforcement from the specified value. The magnitude of the effect of the increased amount of reinforcing steel can be seen by comparing the value of R_m, 1.17, for the initial mean strength of the negative moment section at the interior beam of the five-inch slab with the value, greater than 1.5, for the initial mean strength of the other four sections studied. The reason for the decreased value of R_m is

TABLE 19

Effects of Variations on Slab Strength

	A_s (in^2)	f_y (ksi)	f'_c (ksi)	d (in)	b (in)	h (in)	M_v/b in-kip	M_v $\frac{in\text{-}k}{ft}$	R_m
4 INCH SLAB									
SL-5 Negative Moment Section at Edge Beam									
Design	0.2	50	3.0	0.75	12	4.0	30.9	30.9	1.00
Actual Mean	0.2	60	3.7	1.0	8.7	4.1	34.6	47.7	1.54
Lowest	0.2	55	3.5	1.5	14.5	3.7	22.8	18.9	0.61
Highest	0.2	65	3.9	0.5	2.9	4.5	43.2	179.0	5.78
Time of Test Mean	0.2	60	8.0	1.0	8.7	4.1	36.0	49.6	1.60
Lowest	0.2	55	7.0	1.5	14.5	3.7	23.5	19.4	0.63
Highest	0.2	65	9.0	0.5	2.9	4.5	48.2	199.0	6.45
SL-5 Positive Moment Section at Midgpan									
Design	0.4	50	3.0	0.75	12	4.0	58.4	58.4	1.00
Actual Mean	0.4	60	3.7	0.70	8	4.1	70.1	105.0	1.80
Lowest	0.4	55	3.5	1.20	12	3.7	48.2	48.2	0.83
Highest	0.4	65	3.9	0.20	4	4.5	86.2	259.0	4.43
Time of Test Mean	0.4	60	8.0	0.70	8	4.1	76.3	114.0	1.96
Lowest	0.4	55	7.0	1.20	12	3.7	51.6	51.6	0.88
Highest	0.4	65	9.0	0.20	4	4.5	101.0	303.0	5.17

TABLE 19
(continued)

	A_s	f_y	f'_c	d	b	h	M_v/b	M_v	R_m
	in^2	ksi	ksi	in	in	in	in-k	$\frac{in-k}{ft}$	
5 INCH SLAB									
SL-2 Negative Moment Section at Edge Beam									
Design	0.2	50	3.0	0.75	12	5.0	40.9	40.9	1.00
Actual Mean	0.2	60	3.7	1.0	8.7	5.3	49.0	67.5	1.65
Lowest	0.2	55	3.5	1.5	14.5	4.7	33.8	28.0	0.68
Highest	0.2	65	3.9	0.5	2.9	5.9	61.4	254.0	6.21
Time of Test Mean	0.2	60	8.0	1.0	8.7	5.3	50.4	69.5	1.70
Lowest	0.2	55	7.0	1.5	14.5	4.7	34.5	28.6	0.70
Highest	0.2	65	9.0	0.5	2.9	5.9	66.4	275.0	6.72
SL-2 Negative Moment Section at Interior Beam									
Design	0.2	50	3.0	0.75	6	5.0	39.2	78.4	1.00
Actual Mean	0.2	60	3.7	1.0	6.25	5.3	47.9	92.0	1.17
Lowest	0.2	55	3.5	1.5	7.95	4.7	32.6	49.3	0.63
Highest	0.2	65	3.9	0.5	4.55	5.9	64.6	170.0	2.17
Time of Test Mean	0.2	60	8.0	1.0	6.25	5.3	49.9	95.8	1.22
Lowest	0.2	55	7.0	1.5	7.95	4.7	33.9	51.2	0.65
Highest	0.2	65	9.0	0.2	4.55	5.9	67.8	179.0	2.28

TABLE 19
(continued)

	A_s in^2	A'_s in^2	f_y ksi	f'_c ksi	d_1 in	d_2 in	b in	h in	M_v/b in-kip	M_v/f_t $\frac{in-k}{ft}$	R_m
SL-3 Positive Moment Section											
Design	0.51	--	50	3.0	--	0.75	12	5.0	97.7	97.7	1.00
Initial Mean	0.51	--	60	3.7	--	0.70	8	5.3	122	183	1.87
Smallest	0.51	--	55	3.5	--	1.2	12	4.7	87.1	87.1	0.89
Largest	0.51	--	65	3.9	--	0.2	4	5.9	147	442	4.52
Temporal Mean	0.51	--	60	8	--	0.7	8	5.3	132	198	2.03
Smallest	0.51	--	55	7	--	1.2	12	4.7	92.7	92.7	0.95
Largest	0.51	--	65	9	--	0.2	4	5.9	101.0	512.83	5.25

that the negative moment section at the interior beam is the only section which does not have an increased reinforcement ratio. Rather, the reinforcement ratio for this section is decreased as the mean spacing of the bars, 6.25 inches, is slightly larger than the specified value, 6.0 inches. Still, the mean strength of this section is seventeen percent greater than the specified value. This increase is due chiefly to the increased thickness of the slab and the increased strength of the reinforcement; the increased strength of the concrete has only a minor effect as will be shown.

The small effect of the change of concrete strength can be seen by examining the temporal strengths. The comparison of the values of R_m of the temporal mean and smallest strength with the values for the initial mean and smallest strengths shows only a slight increase in strength due to the doubling or more of the concrete strength. The value of R_m for the temporal smallest strength is 0.63, only a two percent increase in strength over the initial smallest strength. The strength gain of the concrete cannot reverse the negative effects of the other initial variations.

Although the mean compressive strength gain of the concrete did not significantly affect the flexural strength, this does not indicate that the strength of concrete is only of minor importance to the strength of a structure. The shear capacity of a member, which was not investigated in this study, is also very important, and it is more sensitive to the strength of concrete. Consequently, the increase in shear capacity at the time of the test from the initial value would be greater than the increase in the flexural strength.

As with the slabs, the strength of the beams and the effects of the variation of the parameters affecting the strength were investigated by analyziing the strength of several representative sections. In particular, two typical beams were analyzed at both their positive and negative moment sections. The specified strength and the initial and temporal smallest, mean, and largest strengths are shown in Table 20 for each section. The minimum value of R_m was only 0.98, an eight percent increase over the capacity reduction factor of 0.9 specified by the A.C.I. Code [318-77]. However, even this very small reduction from the specified strength has a very low probability of occurrence, only 9.8×10^{-9}. In general, the initial mean strengths are approximately 20 percent greater than the specified strength. This is due to the increased steel strength over the specified value and the increased overall thickness and consequent increase in d and d' (C_r and $H-C_b$ in Figure 22).

As was noted for the slabs, the value of R_m for the temporal strengths are only slightly larger than the values for the initial strengths, thus showing again that the concrete compressive strength is not a very important variable in the determination of the flexural strength of a section.

4.2 STRENGTH OF COLUMNS

As was done in the determination of the strength of the beams and slabs in-situ, rather than compute the strength of each column investigated, representative sections were chosen to be analyzed. The statistical data given in Table 18 was used in the determination of both the temporal and the initial smallest, mean, and largest

TABLE 20

Effects of Variations on Beam Strength

	b (in)	h (in)	f'_c (ksi)	A'_s (in^2)	A_s (in^2)	f_y (ksi)	d_1 (in)	d_2 (in)	M_v (in-k)	R_m
Positive Moment Section of Beam B-7										
Specified	12	18.375	3	2.54	2.37	50	1.5	1.5	1855	1.00
Initial Mean	12	19.125	3.7	2.54	2.37	55	1.5	1.15	2188	1.18
Initial Smallest	12	18.525	3.5	2.54	2.37	50	2.2	1.75	1813	0.98
Initial Largest	12	19.725	3.9	2.54	2.37	60	0.8	0.55	2623	1.41
Temporal Mean	12	19.125	8	2.54	2.37	55	1.5	1.15	2234	1.20
Temporal Smallest	12	18.525	7	2.54	2.37	50	2.2	1.75	1894	1.02
Temporal Largest	12	19.725	9	2.54	2.37	60	0.8	0.55	2642	1.42
Negative Moment Section of Beam B-7										
Specified	12	18.375	3	2.37	2.54	50	1.5	1.5	1985	1.00
Initial Mean	12	19.125	3.7	2.37	2.54	55	1.5	1.15	2341	1.18
Initial Smallest	12	18.525	3.5	2.37	2.54	50	2.2	1.75	1937	0.98
Initial Largest	12	19.725	3.9	2.37	2.54	60	0.8	0.55	2808	1.41
Temporal Mean	12	19.125	8	2.37	2.54	55	1.5	1.15	2390	1.20
Temporal Smallest	12	18.525	7	2.37	2.54	50	2.2	1.75	2019	1.02
Temporal Largest	12	19.725	9	2.37	2.54	60	0.8	0.55	2828	1.42

TABLE 20
(continued)

	b	h	f'_c	A'_s	A_s	f_y	d_1	d_2	M_v	R_m
Positive Moment Section C-4										
Specified	12	18.375	3.0	0.88	3.58	50	1.5	1.5	2656	1.00
Initial Mean	12	19.125	3.7	0.88	3.58	55	1.5	1.15	3174	1.20
Initial Smallest	12	18.525	3.5	0.88	3.58	50	2.2	1.75	2645	1.00
Initial Largest	12	19.725	3.9	0.88	3.58	60	0.8	0.55	3733	1.41
Temporal Mean	12	19.125	8.0	0.88	3.58	55	1.5	1.15	3331	1.25
Temporal Smallest	12	18.525	7.0	0.88	3.58	50	2.2	1.75	2781	1.05
Temporal Largest	12	19.725	9.0	0.88	3.58	60	0.8	0.55	3930	1.48
Negative Moment Section of Beam C-4										
Specified	12	18.375	3.0	2.58	1.88	50	1.5	1.5	1479	1.00
Initial Mean	12	19.125	3.7	2.58	1.88	55	1.5	1.15	1745	1.22
Initial Smallest	12	18.525	3.5	2.58	1.88	50	2.2	1.75	1456	1.02
Initial Largest	12	19.725	3.9	2.58	1.88	60	0.8	0.55	2011	1.40
Temporal Mean	12	19.125	8.0	2.58	1.88	55	1.5	1.15	1787	1.25
Temporal Smallest	12	18.525	7.0	2.58	1.88	50	2.2	1.75	1533	1.04
Temporal Largest	12	19.725	9.0	2.58	1.88	60	0.8	0.55	2102	1.42

strengths as well as the specified strength of representative section. The two sections chosen as representative of the columns were a 12 x 12-inch column with four #5 bars and a 14 x 14-inch column with four #10 bars. The former is the smallest column cross-section investigated and is also the smallest amount of reinforcement while the latter is the largest column cross-section investigated with the largest amount of reinforcement. The strength of each column that was of interest was the combined axial-flexural capacity which can be shown by a Load-Moment interaction diagram. For both of the columns that were chosen, five points on the interaction diagram for the specified strength section, the initial smallest, mean, and largest strength sections, and temporal smallest and mean strength sections were computed. These points are shown in Table 21 and the interaction diagrams for all of the sections are shown in Figures 25 and 26 for the 12 x 12-inch column and in Figure 27 for the 14 x 14-inch column.

For the 12 x 12-inch column the only section which has a smaller capacity than that of the specified section was the initial smallest section, and only at loads of less than approximately 225 kips. This reduction of capacity is only at large eccentricities. Not only is the probability of the smallest strength section very low, only 9.8×10^{-9}, but assumptions made concerning concrete cover over the reinforcement are unrealistic. The inital smallest section has an assumed increase concrete cover over both the tension and compression steel, whereas in reality if the prefabricated reinforcement cage were misplaced in the column forms it would have increased cover over one side and decreased cover over the other. This would result in at

TABLE 21

Effects of Variations on the Column Interaction Diagrams

	A'_s & A_s	h	b	d_1 & d_2	f'_c	f_y	P_O	M_O	P_D	M_D	P_C	M_C	P_T	M_T
Design Coll. Story II	0.62	12	12	2.0	3	50	426	304	167	794	291	579	93	658
Initial Mean	0.62	12	12	2.2	3.7	55	517	338	195	925	348	706	106	738
Initial Smallest	0.62	12	12	3.7	3.5	50	487	329	162	747	283	694	53	489
Initial Largest	0.62	12	12	0.7	3.9	60	548	404	229	1109	419	655	153	1046
Temporal Mean	0.62	12	12	2.2	8	55	1039	384	323	1557	554	1500	183	1151
Temporal Smallest	0.62	12	12	3.7	7	50	911	419	267	1248	446	1200	114	811
Design Col 75 Story	2.54	14	14	2.0	3.0	50	741	1341	234	2141	491	1327	137	1965
Initial Mean	2.54	14	14	2.2	3.7	55	880	1448	274	2407	581	1547	141	2070
Initial Smallest	2.54	14	14	3.7	3.5	50	822	1116	234	1819	492	1375	19	1184
Initial Largest	2.54	14	14	0.7	3.9	60	938	1928	315	3056	677	1667	241	3000
Temporal Mean	2.54	14	14	2.2	8	55	1578	1544	453	3435	870	2983	249	2745
Temporal Smallest	2.54	14	14	3.7	7	50	1390	1306	386	2646	728	2458	108	1723

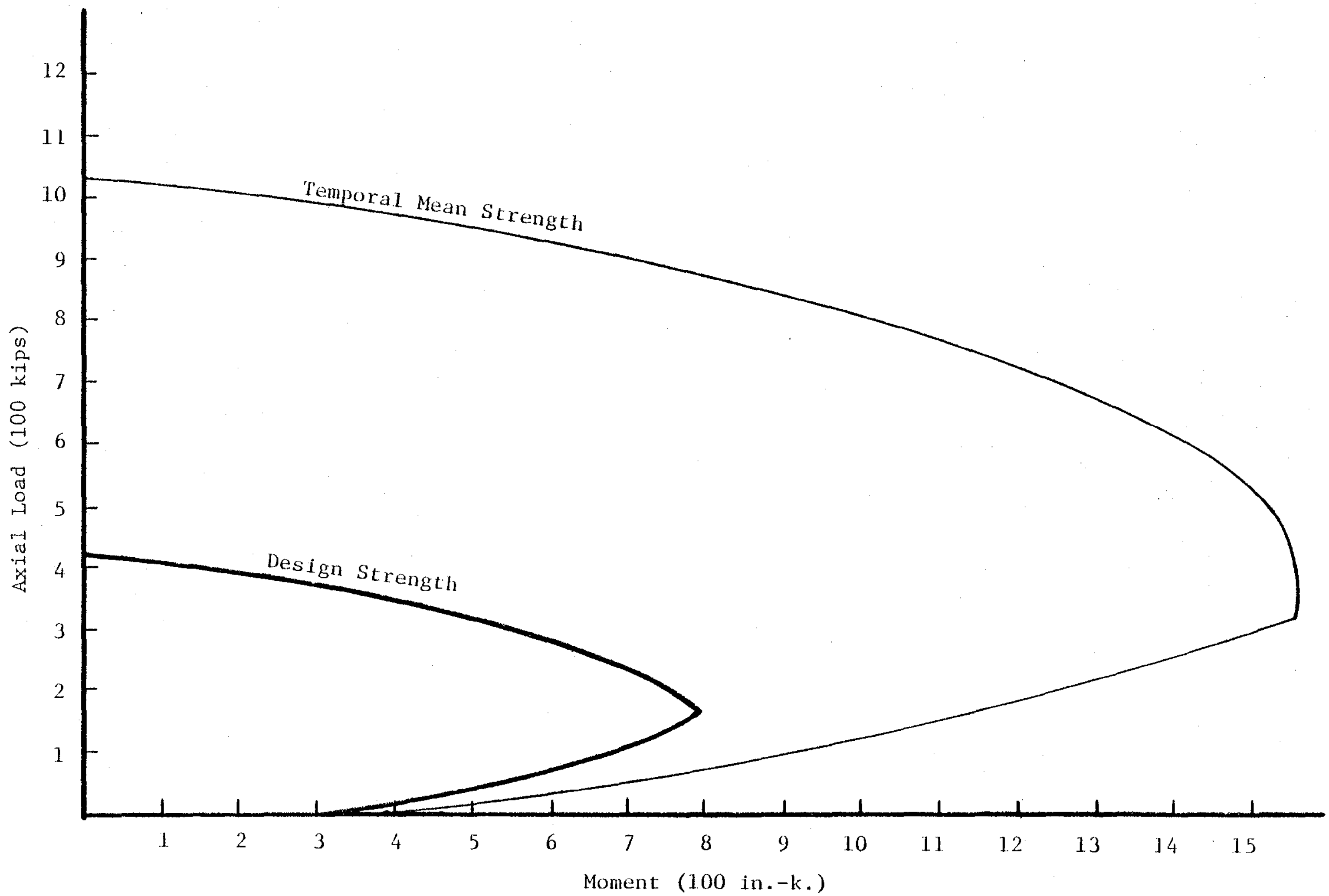


FIGURE 24. INTERACTION DIAGRAMS FOR THE 12 x 12-INCH COLUMN

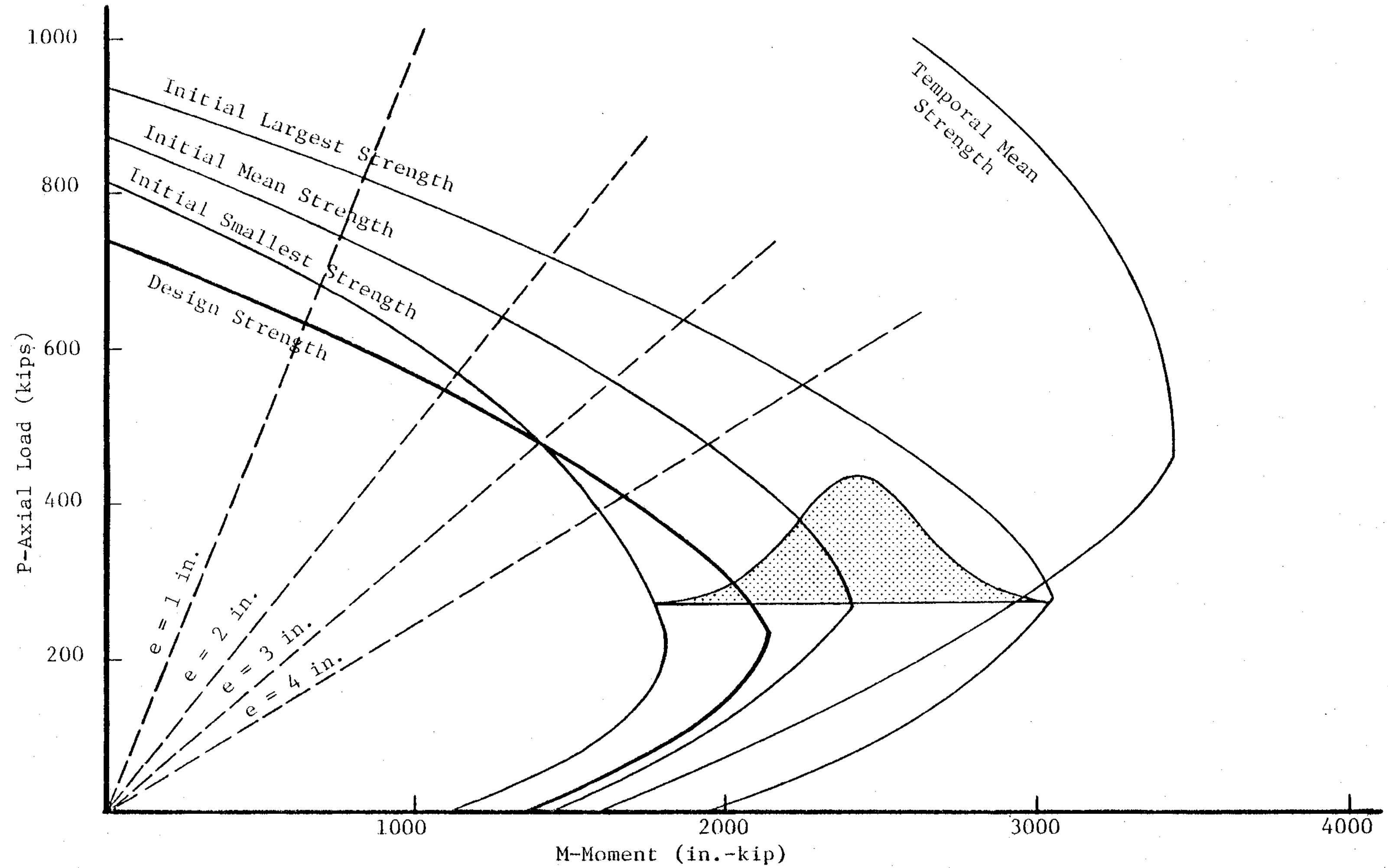


FIGURE 25. INTERACTION DIAGRAMS FOR 12 x 12-INCH COLUMN WITH PROBABILITY DISTRIBUTION AND MEASURED ECCENTRICITIES

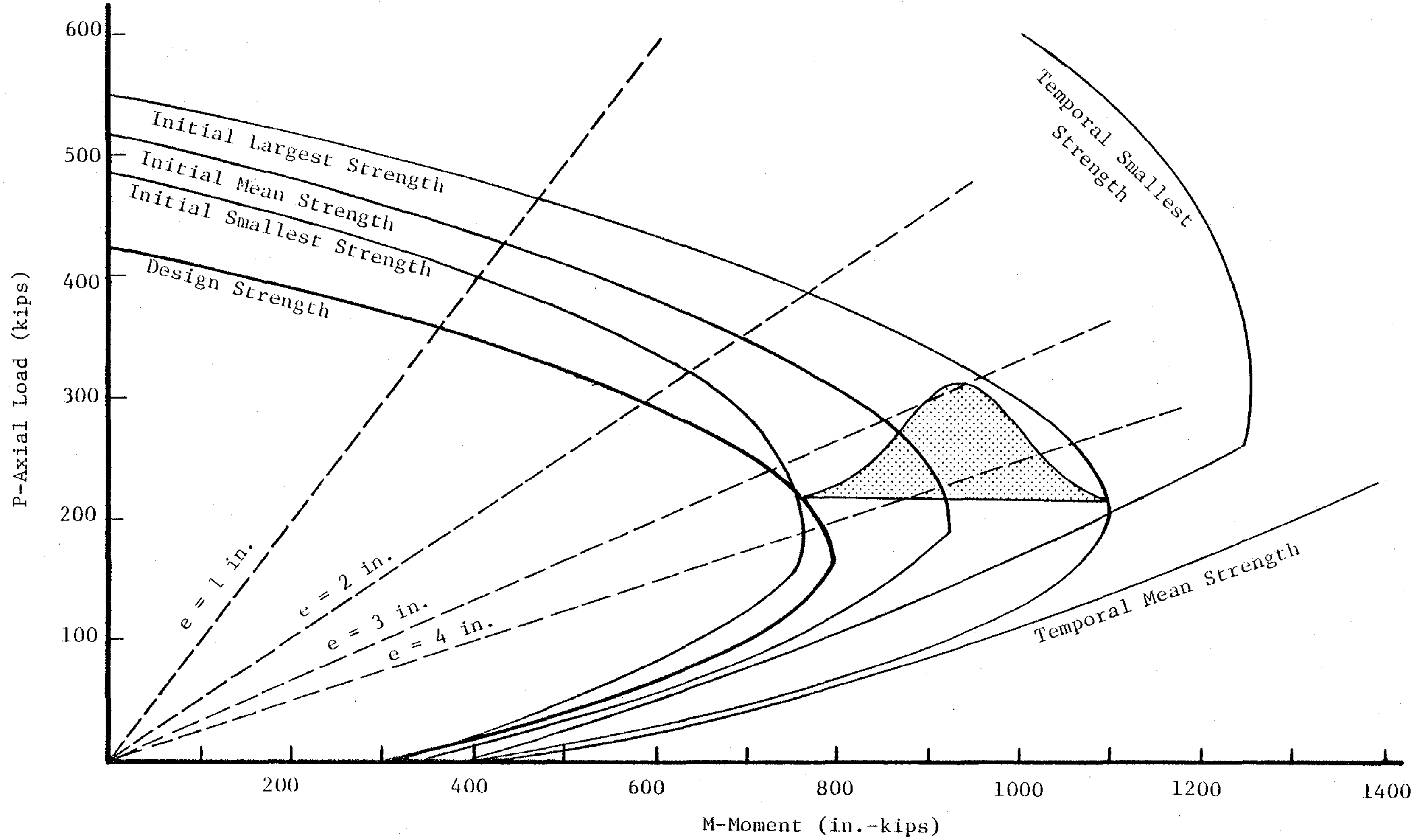


FIGURE 26. INTERACTION DIAGRAM FOR 14 x 14-INCH COLUMN

least a small increase in the flexural strength in the weakest direction. Similarly the initial largest strength section is unrealistic in the assumption that the concrete cover over both the tension and compression steel will be decreased simultaneously. Consequently, the flexural capacity is slightly over estimated for this section.

As was noted for the slabs and beams, the large increase in concrete strength does not result in a great increase in the flexural strength of a member. It can be seen that this is true for the columns too, by the small increase in the moment intercept of the temporal smallest and mean strength section interaction diagrams. At the same time, the large influence of the concrete strength on the axial capacity of the column is evident from the same diagram.

The minimum value of R_m may be closely approximated by the ratio of the balanced moment of specified strength section to the balanced moment of the initial smallest strength section, which occur close to the same load. From the values given in Table 21, it can be seen that the ratio is 747/794 or 0.94. This much greater than the capacity reduction factor of 0.7 specified by the A.C.I. Code.

The interaction diagrams for the 14 x 14-inch column (Figure 26) are very similar to the ones for the 12 x 12-inch column. Only for loads less than about 650 kips does the initial smallest strength section have a reduced flexural capacity from that of the specified strength section. As was noted in the discussion of the 12 x 12-inch column, the low probability of occurrence and unrealistic assumptions concerning concrete cover should be noted with regard to the smallest and largest strength sections. The reduced capacity occurs at slightly lower levels of eccentricity than for the smaller column. The minimum value of R_m may be approximated, as before, by the ratio of the balanced moments of the initial smallest strength and the specified strength sections. This value is 2140/2410 or 0.89 which is still much greater than the capacity reduction factor albeit it is smaller than the value of the smaller column.

A difference between the two sets of interaction diagrams may be noted with respect to the temporal smallest strength section. For extremely small loads, less than 200 kips, this section has a flexural strength less than that of the specified strength section. The observations with regard to axial capacity are the same as before, the increase in axial strength is nearly proportional to the increase in compressive strength of the concrete.

5. SUMMARY

The major results of this project are stated by the following points. The order should not be misconstrued as the order of importance, but rather follows closely the body of this text.

1) The strength of the steel in the structure is approximately 10 to 20 percent greater than the specified value.

2) The strength of concrete in-situ can be reasonably well estimated from the results of non-destructive tests. The mean strength of the concrete in the building tested was 8 k.s.i. This represents a strength gain of 213% over the estimated 28-day strength, which is within the range of strength gain determined by other investigators.

3) The average variation of centerline-to-centerline dimensions between columns and floor-to-floor heights from the specified value is only .2 percent. The maximum difference of any dimension from the specified value was somewhat less than 2 inches.

4) The thickness of the slab in the structure is generally greater than the specified thickness. The mean thickness of the specified 5-inch slabs is 5.3 inches and the specified 4-inch slab is 4.1 inches.

5) The spacing of reinforcement may be quite erratic from floor to floor. In general the spacing discovered was less than specified. The only exception to this was the negative moment reinforcement over the interior beams.

6) The average concrete cover over the reinforcement, although specified to be 3/4 inch both for top and bottom steel, was 1.0 inch for the top steel and 0.70 inch for the bottom steel. These values agree very well with data given by other investigators.

7) In no beam investigated did the width differ from the specified value by as much as 1/8 inch whereas the average depth was 0.8 inch greater than specified. The average cover over the top beam steel was 1.5 inch, equal to the specified value, while the cover for the bottom steel was 1.15 inch, or 0.35 inch less than the specified value.

8) The statistical data concerning structural variations were used to determine the average strength of the slabs in the structure. In general the average strength was more than 50 percent greater than the strength of the specified slab and the lowest average was still in excess of 15% stronger than specified.

9) The statistical data was used to determine the smallest strengths of the slabs (only 10^{-9} probability of lower strength). In general the smallest strength of the slabs was approximately 65 percent of the specified value. The capacity reduction factor used for designing slabs is 0.90 so that in this case there may exist a slab which is not strong enough to support the allowable capacity, although the probability of this is very small.

10) The effects of temporal variations were also investigated. The large increase in concrete compressive strength had only small effect on the flexural strength of the slabs.

11) The average strength of the beams in-situ was 19 percent greater than the specified value.

12) The smallest strength was approximately equal to the specified strength. This is equivalent to a reduction factor of 1.0 compared to the specified value of 0.90.

13) The effects of temporal variations were very small. The results of the large increase in concrete strength were only a few percent increase in flexural strength.

14) The average flexural strength of the columns in the structure was greater than the flexural strength of the specified section at all axial load levels.

15) At low axial load levels the smallest strength section has a reduced flexural capacity from that of the specified section. The largest reduction is approximately 11 percent, whereas the A.C.I. Code specifies a reduction of 30 percent for all columns.

16) The effect of the large increase in concrete strength on the axial capacity of the columns is a large increase in that capacity.

6. CONCLUSIONS

The project achieved most of the objectives that were set forth at the onset. Statistical data concerning the variations of material properties and structural dimension was gathered and analyzed. This data, as an addition to the small body of information concerning structural variations in existence, may be the most valuable contribution of this work.

The effect of the structural variations of the strength on the members was examined. The comparisons of probable strength reductions from the design strength with the reductions specified by the A.C.I. Code seem to indicate that the values specified by the Code are conservative.

7. ACKNOWLEDGEMENTS

This work could not have been accomplished without the aid of many individuals, only two of which I will mention here. A complete list can be found in Appendix A of Reference 1. I am grateful to all who helped. Here I would especially like to acknowledge the invaluable assistance given me by Dr. D. S. Hatcher and Dr. T. V. Galambos. Their encouragement and help enabled me overcome the many difficulties inevitable in a project such as this.

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9. VITA

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