SC

PRELIMINARY INVESTIGATION INTO THE SHORING SYSTEMS

by

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ABSTRACT

In multistory reinforced concrete building construction, the freshly placed floor is supported by shores which are supported themselves on several previously cast floors. The construction loads in these supporting floors may appreciably exceed the design service loads. Such loads depend on the sequence of erection and cannot be easily determined.

In this report, some methods for determining these erection loads are presented with a few simplifying assumptions. Laboratory experiments were undertaken to determine the variation of the modulus of elasticity and crushing strength of concrete cylinders with age. The variation of flexural strength and stiffness of model reinforced concrete beams with age was also investigated. The shoring loads in a typical flat slab type building were measured and the results are discussed.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF SYMBOLS

a and b = the lengths of the edges of the slab in the directions x and y respectively (see Fig. A).

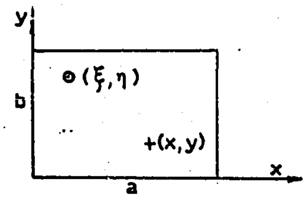

Fig. A

h = the thickness of the slab.

α = the density of the slab.

q' = the dead weight per unit area of the slab.

$E^{(\omega)}$ = the modulus of elasticity at the age of ω days after placing of concrete.

υ = Poisson's ratio.

$N_s^{(\omega)}$ = the modulus of the slab $N_s^{(\omega)} = \dfrac{E^{(\omega)}\ h^3}{12(1-\upsilon^2)}$ at the age of ω days after placing of concrete.

$W^{(\omega)}(x,y)$ = the additional deformation of the slab at right angles to the plane of the slab at the age of ω days after placing of concrete.

$W_q^{(\omega)}(x,y)$ = the additional deformation of the slab due to the load q at the age of ω days after placing of concrete.

$W_n(x,y)$ = the n-th normalised characterisic function of the slab.

$x_n a$ = the value of the n-th root of the equation of frequency of the slab.

$$\left(\text{The angular frequency } \omega_a = x_n^2 \sqrt{N_s/\alpha h}\right)$$

$\lambda_n^{(\omega)}$ = the n-th characteristic value $\left(\lambda_n^{(\omega)} = x_n^4\ N^{(\omega)}\right)$ at the age of ω days after placing of concrete.

A_n = $\displaystyle\int_0^a\int_0^b W_n\ dxdy$

$K^{(\omega)}(x,y,\xi,\eta)$ = the icfluence function of the slab (the nucleus of an integral equation) at an age of ω days after placing of concrete. This function expresses the deflection at the point (x,y) due to unit load at the point (ξ,η).

$p^{(\omega)}(x,y)$ = the additional load on formwork at the age ω days after placing of concrete.

k = the coefficient of compression of the formwork (the force per unit area of the formwork which is required in order to produce a deformation of unit length in a vertical direction).

m = No. of levels of shores.

N = time between placing of a fresh slab and the removal of props from above the lowest slab in the system.

E_s = modulus of elasticity of reinforcement.

E_c = modulus of elasticity of concrete.

M_u = ultimate design resisting moment.

d = effective depth of the member.

f'_c = crushing strength of concrete.

f_y = yield stress of steel.

A_s = area of steel bar.

δ = deflection.

I = moment of inertia.

L = span.

P_b = reinforcement ratio producing balanced conditions at ultimate strength.

P_u = ultimate load.

ε_s = steel strain.

ε_c = concrete strain.

$q = \frac{p\, f_y}{f'_c}$

V_{cr} = cracking shear force.

v_{cr} = cracking shear stress.

$p = \frac{A_s}{bd}$

Chapter 1

INTRODUCTION

Traditionally engineering education and design practices have concentrated upon the problems of the detailed design of the finished structure to support the specified loads due to occupancy, self weight and earthquake or wind load. However the design of temporary members (formwork, scaffold, and props etc.) to resist the construction loads has usually been considered less important and left to the contractor to deal with on site. This philosophy is illogical as a survey of failures would reveal that as many failures occur during the short constructional period as during the complete service life of the complete structures. Considering the construction of reinforced concrete buildings the problem is more complex than for a steel framed building. The usual practice for a concrete frame or flat slab building is to use formwork which is supported on the lower parts of the building. Two problems arise, firstly the formwork needs to be strong enough to support the wet concrete and stiff enough to give the required form. Secondly the formwork is supported on other parts of newly completed reinforced concrete structure which will not have obtained their full design strength. The problem is further aggravated by the fact that the construction loads may even be greater than the occupancy load of the building.

Quality, safety, and economy are the three basic objectives in formwork. The design manual "Formwork of Concrete"(13) states that formwork costs may range from 35 to 60 percent of the cost of the concrete structure. Since formwork is an appreciable proportion of the cost of the concrete building, its re-use is desirable as soon as possible. If the construction period can be reduced due to the use of the smaller building cycle a lot of advantage can be obtained. In some cases, extra profits can be earned by being able to use the structure earlier. For the contractors, how to get more space to let other building operations proceed freely is a problem in site. Stripping the formwork earlier is the method to meet the special requirements above. Against this, however, there is a risk that damage to the floors may result if the forms are removed too soon.

Consider the construction sequence of a typical multi-story flat slab type reinforced concrete building. The formwork for the lowest floor is supported on grade and the floor slab cast. The formwork for the second floor is placed upon the first floor and the second floor cast. These loads are transmitted directly to grade as the first floor is still on formwork. However for the third floor, the first floor is reshored and the first floor formwork is used for the third floor. Similarly the fourth floor formwork is a reuse of the second floor formwork. Naturally a point comes when the two form supported floors are supported on two (or more) reshored floors, all of which are partially supported upon the lowest floor of the sequence. Thus the constructional load of the freshly completed floor is supported by the lower floors in the system. Naturally the load is distributed between these floors as some function of their stiffnesses, which are a function of age. Unfortunately the strength of the floors is also a function of age and rate of gain in strength is slower than the gain of elasticity.

The problem then is to derive a reshoring system and construction sequence such that the loads on any slab are less than some desired function of the " design" load of that slab taking into account the age of the slab.

A complete design of the optimum shoring system for a typical reinforced concrete building would involve the following considerations:

1. Knowledge of the rate of gain of strength and stiffness with age of the concrete members.
2. Accurate analysis of the loads on the members of the partially completed building. It must be noted that the behaviour of the structure during construction is completely different from the completed structure because the top floor (one being cast) does not restrain the columns: thus the moments in the newest slab are increased. Furthermore the distribution of moments in the slabs due to the concentrated loads from the shores will be different from the assumed design distribution. The flexibility of the shores must also be considered in calculating the distribution of load between the floors.
3. A knowledge of the economic advantages of a decrease in construction time, with its higher intrinsic cost (use of high early strength concrete, more levels of shoring, closer shores etc.), against the profitability of being able to use the structure earlier.
4. Knowing the load redistribution during the reshoring operation when the reshores are tightened up. This factor can never be estimated reliably, so any design must allow some margin of safety as over tightening of the reshores increases the load on the lowest floor.

This project involved measuring the load distribution on the formwork and reshores of a typical flat slab building (the office tower of The Engineering Building, University of Ottawa), and trying to relate the load on the floors to the load capacity of the floors. In addition laboratory experiments were undertaken to determine the variation of the modulus of elasticity and crushing strength of concrete cylinders with age and the flexural strength and stiffness of model reinforced concrete beams with age.

Chapter 2

HISTORICAL REVIEW

Nielsen[1] in 1952 presented a detailed analysis of the interaction between formwork and slab floor under loads applied to the system by placing fresh slabs and by removing props from beneath the lowest slab in the system. The method employed in the construction of multistory building is illustrated in Fig. 2-1. After laying the foundations, the formwork is erected for the walls and floor slabs of the first story. Then the walls are cast, and a few days later when the concrete of the walls has hardened the first floor slab is cast (Stage No.1). The numbers of days given in the following are reckoned from this day. The erection of the formwork for the next story is started a few days later, and under normal condition, the second floor slab can be cast 6 to 7 days after the first floor slab (Stage No.2). The third floor slab is cast a week after that (Stage No.3). Then the first formwork in ground floor is removed (e.g. after 15 days) for further use (stage No. 4). The fourth floor slab is cast after about 21 days (Stage No. 5), and so on.

Nielsen's method was derived from the theory of elasticity enables the load distribution in the formwork and the moment in a floor slab during construction to be determined. This method assumed:-

1. That the shrinkage and creep of the concrete in the slabs may be disregarded for analysis, and that the slabs behave elastically.
2. That the props supporting the slabs and formwork may be regarded as a continuous uniform elastic support.

3. That the slabs are supported from a completely rigid foundation.

4. That the modulus of elasticity of the concrete E_c, which is increased with time, is taken into account.

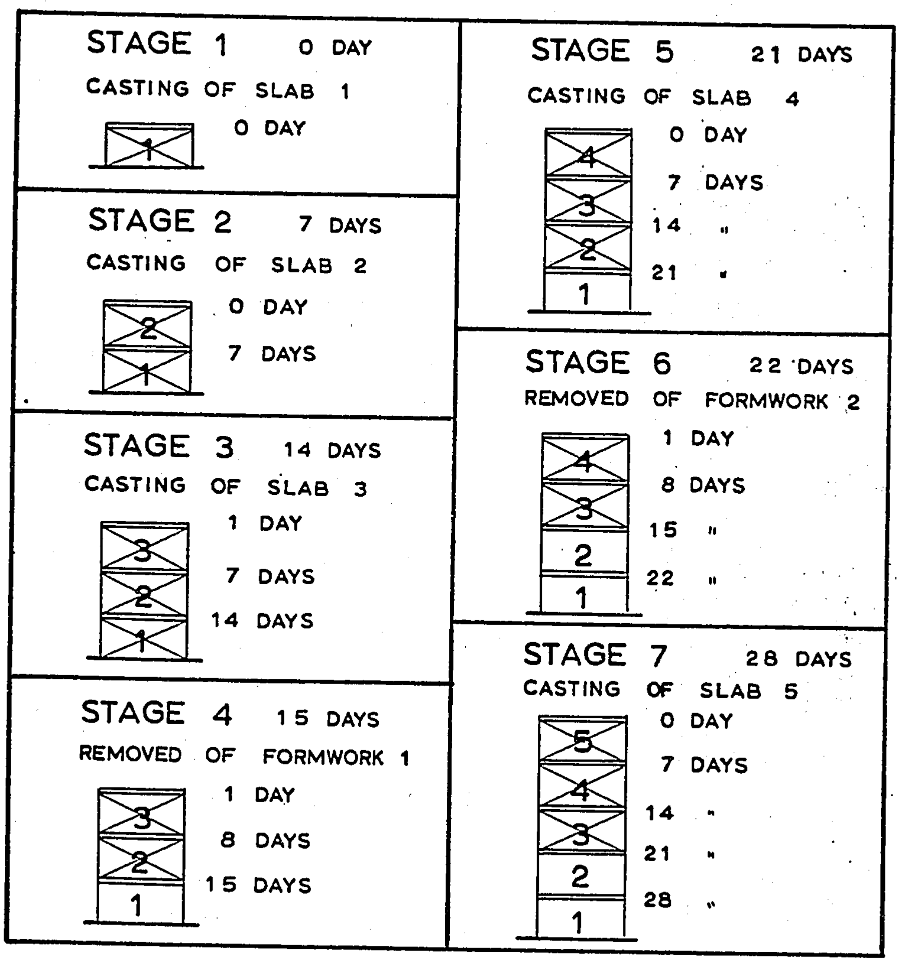

FIG. 2-1 RATE OF CONSTRUCTION FOR 7-DAY CYCLE, THREE LEVELS OF SHORES (m = 3), AND N = 1 DAY. (REF. NO. 1)

The analysis is confined to a rectangular slab subjected to take that arbitrary boundary conditions. The characteristic functions and the characteristic values of this slab are assumed to be known.

The derivation of this method of calculation is based on the assumptions described above and the rate of construction is shown in Fig. 2-1. The first seven stages corresponding to this rate of construction are dealt with and the derivation for the additional load acting on the formwork and the additional moments applied to the floor slabs in each stage are shown in the following:

Stage No.1

The first floor slab is cast. Since shrinkage and creep are to be disregarded, the concrete of the slab will harden without being subjected to any moments at all. The load acting on the formwork is uniformly distributed, and can be written

$$p^{(0)}(x,y) = q' \quad ....(2\text{-}1)$$

Stage No.2

The second floor slab is cast a week later. The dead load of this slab will be carried by the first floor slab and the formwork of the second floor. This formwork is assumed to rest on a perfectly rigid support.

In the first place, the load distribution $p^{(7)}(x,y)$ in the lower formwork which is caused by the additional load due to the upper floor slab has to be calculated. If the characteristic functions of a slab under any arbitrary boundary conditions are normalised in such a manner that $\int_0^a \int_0^b W_n^2 \, dx \, dy = 1$, then the influence function can be written as follows

$$K(x,y,\xi,\eta) = \sum \frac{1}{\lambda_n} W_n(x,y) W_n(\xi,\eta) \qquad \text{...........(2-2)} \quad n = 1,2,3....$$

Then the deformation due to a uniformly distributed q' per unit area is

$$W_{q'}(x,y) = q' \int_0^a \int_0^b K(x,y,\xi,\eta) \, d\xi d\eta = q' \sum \frac{1}{\lambda_n} W_n(x,y) \int_0^a \int_0^b W_n(\xi,\eta) \, d\xi d\eta \quad \text{...(2-3)}$$

By introducing the notation

$$A_n = \int_0^a \int_0^b W_n(\xi,\eta) \, d\xi d\eta \quad \text{......(2-4)}$$

Simplifying

$$W_{q'}(x,y) = q' \sum \frac{A_n}{\lambda_n} W_n(x,y) \quad \text{....(2-5)} \qquad n = 1,3,5......$$

The characteristic functions of an even order are antisymmetric. In that case, A_n becomes equal to zero for these functions. Consequently, every second characteristic function in the expression for $W_{q'}(x,y)$ will vanish.

The additional deformation $W^{(7)}(\xi,\eta)$ of the lower floor slab gives rise to the reaction $kW^{(7)}(\xi,\eta)$ in the elastic support. Then

$$W^{(7)}(x,y) = W_{q'}^{(7)}(x,y) - k \int_0^a \int_0^b K^{(7)}(x,y,\xi,\eta) W^{(7)}(\xi,\eta) \, d\xi d\eta \quad \text{.....(2-6)}$$

This is an inhomogeneous linear integral equation having a symmetrical nucleus.

Imagine now $W^{(7)}(x,y)$ and $W^{(7)}(\xi,\eta)$ be expanded into a series which are built up by means of the characteristic functions of the slab

$$W^{(7)}(x,y) = \sum c_n^{(7)} W_n(x,y)$$

and

$$W^{(7)}(\xi,\eta) = \sum c_n^{(7)} W_n(\xi,\eta)$$

respectively.

After insertion of these expressions, the sign of summation can be deleted. The coefficients of even order vanish, and we get

$$c_n^{(7)} = \frac{\frac{q'}{k} A_n}{1 + \frac{\lambda_n^{(7)}}{k}} \qquad n = 1,3,5.....$$

The additional moment on the slab can be determined from the deformation

$$W^{(7)}(x,y) = \frac{q'}{k} \sum \frac{A_n}{1 + \frac{\lambda_n^{(7)}}{k}} W_n(x,y) \qquad n = 1,3,5.....$$

Then the additional load acting on the formwork is

$$p^{(0)}(x,y) = q'$$

$$p^{(7)}(x,y) = k\, W^{(7)}(x,y) = q' \sum \frac{A_n}{1 + \frac{\lambda_n^{(7)}}{k}} W_n(x,y) \qquad n = 1,3,5....$$

Stage No.3

The third floor slab is cast 14 days after the first floor slab was cast. The load due to the newly placed concrete is uniformly distributed over the second floor slab (7-day). The second floor slab is deformed, and a part of the load is transmitted through the formwork to the first floor slab (14-day). Consider the effect of the additional load only. Then the deformation of the floor slab No.2 is

$$W^{(7)}(x,y) = W_{q'}^{(7)}(x,y) - k\int_0^a \int_0^b K^{(7)}(x,y,\xi,\eta)\left[W^{(7)}(\xi,\eta) - W^{(14)}(\xi,\eta)\right] d\xi d\eta$$

Where $k\left[W^{(7)}(\xi,\eta) - W^{(14)}(\xi,\eta)\right]$ denotes the upward load $p^{(7)}(\xi,\eta)$ per unit area of the formwork at the point $x=\xi$, $y=\eta$.

The deformation of the floor slab No.1 is written

$$W^{(14)}(x,y) = \int_0^a \int_0^b K^{(14)}(x,y,\xi,\eta)\left[p^{(7)}(\xi,\eta) - k\, W^{(14)}(\xi,\eta)\right] d\xi d\eta$$

By analogy with the above the assumed functions per deflection are

$$W^{(7)}(x,y) = \sum c_n^{(7)}\; W_n(x,y)$$

$$W^{(14)}(x,y) = \sum c_n^{(14)}\; W_n(x,y)$$

These expressions are inserted in the equations for $W^{(7)}$ and $W^{(14)}$ and

$$c_n^{(7)}\left[1 + \frac{\lambda_n^{(7)}}{k}\right] - c_n^{(14)} = \frac{q'}{k}\, A_n$$

and

$$c_n^{(7)} = \left[2 + \frac{\lambda_n^{(14)}}{k}\right] c_n^{(14)}$$

Taken together, these equations yield

$$c_n^{(7)} = \frac{\left[2 + \frac{\lambda_n^{(14)}}{k}\right] \frac{q'}{k}\, A_n}{\left[1 + \frac{\lambda_n^{(7)}}{k}\right]\left[2 + \frac{\lambda_n^{(14)}}{k}\right] - 1}$$

$$c_n^{(14)} = \frac{\frac{q'}{k} A_n}{\left[1 + \frac{\lambda_n^{(7)}}{k}\right]\left[2 + \frac{\lambda_n^{(14)}}{k}\right] - 1} \quad .....(2\text{-}15b)$$

Hence the additional deformation is also given, and the additional moments can be determined. The additional load on the formwork is

$$p^{(0)}(x,y) = q' \quad .....(2\text{-}16a)$$

$$p^{(7)}(x,y) = k\left[W^{(7)}(x,y) - W^{(14)}(x,y)\right]$$

$$= q' \sum \frac{\left[1 + \frac{\lambda_n^{(14)}}{k}\right] A_n}{\left[1 + \frac{\lambda_n^{(7)}}{k}\right]\left[2 + \frac{\lambda_n^{(14)}}{k}\right] - 1} W_n(x,y) \quad ....(2\text{-}16b)$$

$$p^{(14)}(x,y) = k\, W^{(14)}(x,y)$$

$$= q' \sum \frac{A_n}{\left[1 + \frac{\lambda_n^{(7)}}{k}\right]\left[2 + \frac{\lambda_n^{(14)}}{k}\right] - 1} W_n(x,y) \quad ....(2\text{-}16c)$$

$n = 1,3,5......$

Stage No.4

One day after the third floor slab was placed the formwork below first floor slab is removed. Since the upward reaction caused by this formwork has disappeared, the system is submitted to a downward additional load of the same character as the total upward reaction due to the first form in Stage No. 3. The increase in the modulus of elasticity of the concrete during the preceding day is neglected.

In accordance with the previous calculations, the additional load is

$$p^{(14)}(x,y) = q' \sum B_n \, W_n(x,y)$$

where

$$B_n = A_n \left(1 + \frac{1}{1 + \frac{\lambda_n^{(7)}}{k}} + \frac{1}{\left[1 + \frac{\lambda_n^{(7)}}{k}\right]\left[2 + \frac{\lambda_n^{(14)}}{k}\right] - 1} \right)$$

Since this additional load is directed downwards, the formwork can be considered to be subjected to tensile stresses.

For the floor slab No.2, the deformation is

$$W^{(7)}(x,y) = k \int_0^a \int_0^b K^{(7)}(x,y,\xi,\eta) \left[W^{(14)}(\xi,\eta) - W^{(7)}(\xi,\eta) \right] d\xi d\eta$$

and the deformation of the floor slab No.1 is

$$W^{(14)}(x,y) = \int_0^a \int_0^b K^{(14)}(x,y,\xi,\eta) \Big[p^{(14)}(\xi,\eta) - k \left\{ W^{(14)}(\xi,\eta) - W^{(7)}(\xi,\eta) \right\} \Big] d\xi d\eta$$

If use is made of the assumed functions

$$W^{(7)}(x,y) = \sum c_n^{(7)} \, W_n(x,y)$$

$$W^{(14)}(x,y) = \sum c_n^{(14)} \, W_n(x,y)$$

we get

$$c_n^{(14)} = \left[1 + \frac{\lambda_n^{(7)}}{k} \right] c_n^{(7)}$$

$$c_n^{(14)}\left[1+\frac{\lambda_n^{(14)}}{k}\right] = \frac{q'}{k}B_n + c_n^{(7)} \qquad \text{....(2-20b)}$$

Taken together, these equations yield

$$c_n^{(7)} = \frac{\frac{q'}{k}\,B_n}{\left[1+\frac{\lambda_n^{(7)}}{k}\right]\left[1+\frac{\lambda_n^{(14)}}{k}\right] - 1} \qquad \text{.....(2-21a)}$$

$$c_n^{(14)} = \frac{\frac{q'}{k}B_n\left[1+\frac{\lambda_n^{(7)}}{k}\right]}{\left[1+\frac{\lambda_n^{(7)}}{k}\right]\left[1+\frac{\lambda_n^{(14)}}{k}\right] - 1} \qquad \text{.....(2-21b)}$$

The additional load on the formwork is

$$p^{(0)}(x,y) = 0 \qquad \text{......(2-22a)}$$

$$p^{(7)}(x,y) = k\left[W^{(7)}(x,y) - W^{(14)}(x,y)\right]$$

$$= q'\sum \frac{-\frac{\lambda_n^{(7)}}{k}\,B_n}{\left[1+\frac{\lambda_n^{(7)}}{k}\right]\left[1+\frac{\lambda_n^{(14)}}{k}\right] - 1}\,W_n(x,y) \qquad \text{......(2-22b)}$$

$n = 1,3,5.....$

Stage No.5

At 21 days the fourth floor slab is cast. By using the above method of analysis

$$W^{(7)}(x,y)=W_q^{(7)}(x,y) - k \int_0^a \int_0^b K^{(7)}(x,y,\xi,\eta) \left[W^{(7)}(\xi,\eta) - W^{(14)}(\xi,\eta) \right] d\xi d\eta \qquad \text{...........(2-23a)}$$

$$W^{(14)}(x,y)= k \int_0^a \int_0^b K^{(14)}(x,y,\xi,\eta) \left[W^{(7)}(\xi,\eta) - 2W^{(14)}(\xi,\eta)+W^{(21)}(\xi,\eta) \right] d\xi d\eta \qquad \text{...........(2-23b)}$$

$$W^{(21)}(x,y) = k \int_0^a \int_0^b K^{(21)}(x,y,\xi,\eta) \left[W^{(14)}(\xi,\eta) - W^{(21)}(\xi,\eta) \right] d\xi d\eta \qquad \text{...........(2-23c)}$$

Hence the assumed functions are:-

$$W^{(7)}(x,y) = \sum c_n^{(7)} W_n(x,y) \qquad \text{.........(2-24a)}$$

$$W^{(14)}(x,y) = \sum c_n^{(14)} W_n(x,y) \qquad \text{.........(2-24b)}$$

$$W^{(21)}(x,y) = \sum c_n^{(21)} W_n(x,y) \qquad \text{.........(2-24c)}$$

By substituting these functions, the following system of equations is obtained

$$c_n^{(7)} \left[1 + \frac{\lambda_n^{(7)}}{k} \right] - c_n^{(14)} = \frac{q'}{k} A_n \qquad \text{.......(2-25a)}$$

$$c_n^{(14)} \left[2 + \frac{\lambda_n^{(14)}}{k} \right] = c_n^{(7)} + c_n^{(21)} \qquad \text{.........(2-25b)}$$

$$c_n^{(21)} \left[1 + \frac{\lambda^{(21)}}{k} \right] = c_n^{(14)} \qquad \text{...........(2-25c)}$$

Finally, this system of equations yields

$$c_n^{(21)} = \frac{\frac{q'}{k} A_n}{\left[1 + \frac{\lambda_n^{(7)}}{k}\right]\left[\left(1 + \frac{\lambda_n^{(21)}}{k}\right)\left(2 + \frac{\lambda_n^{(14)}}{k}\right) - 1\right] - \left[1 + \frac{\lambda_n^{(21)}}{k}\right]} \quad \text{....(2-26a)}$$

$$c_n^{(14)} = \left[1 + \frac{\lambda_n^{(21)}}{k}\right] c_n^{(21)} \quad \text{..........(2-26b)}$$

$$c_n^{(7)} = \left[\left(1 + \frac{\lambda_n^{(21)}}{k}\right)\left(2 + \frac{\lambda_n^{(14)}}{k}\right) - 1\right] c_n^{(21)} \quad \text{........(2-26c)}$$

From the additional deformation given, and additional moments can be determined.

The additional load on the formwork is

$$p^{(0)}(x,y) = q' \quad \text{.......(2-27a)}$$

$$p^{(7)}(x,y) = k\left[W^{(7)}(x,y) - W^{(14)}(x,y)\right]$$

$$= q' \sum \frac{\left[\left(1 + \frac{\lambda_n^{(21)}}{k}\right)\left(1 + \frac{\lambda_n^{(14)}}{k}\right) - 1\right] A_n}{\left[1 + \frac{\lambda_n^{(7)}}{k}\right]\left[\left(1 + \frac{\lambda_n^{(21)}}{k}\right)\left(2 + \frac{\lambda_n^{(14)}}{k}\right) - 1\right] - \left[1 + \frac{\lambda_n^{(21)}}{k}\right]} W_n(x,y)$$

.........(2-27b)

$$p^{(14)}(x,y) = k\left[W^{(14)}(x,y) - W^{(21)}(x,y)\right]$$

$$= q' \sum \frac{\frac{\lambda_n^{(21)}}{k} A_n}{\left[1 + \frac{\lambda_n^{(7)}}{k}\right]\left[\left(1 + \frac{\lambda_n^{(21)}}{k}\right)\left(2 + \frac{\lambda_n^{(14)}}{k}\right) - 1\right] - \left[1 + \frac{\lambda_n^{(21)}}{k}\right]} W_n(x,y)$$

$n = 1,3,5......$

......(2-27c)

Stage No.6

One day after the fourth floor slab was cast, the formwork below second floor slab is removed. Similarly to Stage No. 4, we can say that the system is subjected a downward additional load equal to the upward reaction caused by form No.2 in the Stage No. 5. The increase in the modulus of elasticity during the preceding day is neglected.

In accordance with the previous calculations, the additional load is

$$p^{(14)}(x,y) = q' \sum B_n^* W_n(x,y) \qquad \ldots\ldots\ldots(2\text{-}28a)$$

where

$$B_n^* = A_n + \frac{\left[1 + \frac{\lambda_n^{(14)}}{k}\right] A_n}{\left[1 + \frac{\lambda_n^{(7)}}{k}\right]\left[2 + \frac{\lambda_n^{(14)}}{k}\right] - 1} + \frac{-\frac{\lambda_n^{(7)}}{k} B_n}{\left[1 + \frac{\lambda_n^{(7)}}{k}\right]\left[1 + \frac{\lambda_n^{(14)}}{k}\right] - 1}$$

$$+ \frac{\frac{\lambda_n^{(21)}}{k} A_n}{\left[1 + \frac{\lambda_n^{(7)}}{k}\right]\left[\left(1 + \frac{\lambda_n^{(21)}}{k}\right)\left(2 + \frac{\lambda_n^{(14)}}{k}\right) - 1\right] - \left[1 + \frac{\lambda_n^{(21)}}{k}\right]} \qquad \ldots(2\text{-}28b)$$

The following calculations are similar to those made in the Stage No.4, and the results can be written directly.

By using the assumed functions

$$W^{(7)}(x,y) = \sum c_n^{(7)} W_n(x,y) \qquad \ldots\ldots\ldots\ldots(2\text{-}29a)$$

$$W^{(14)}(x,y) = \sum c_n^{(14)} W_n(x,y) \qquad \ldots\ldots\ldots(2\text{-}29b)$$

we find $$c_n^{(7)} = \frac{\frac{q'}{k} B_n^*}{\left(1 + \frac{\lambda_n^{(7)}}{k}\right)\left(1 + \frac{\lambda_n^{(14)}}{k}\right) - 1} \quad \text{.......(2-30a)}$$

$$c_n^{(14)} = \frac{\frac{q'}{k} B_n^* \left[1 + \frac{\lambda_n^{(7)}}{k}\right]}{\left[1 + \frac{\lambda_n^{(7)}}{k}\right]\left[1 + \frac{\lambda_n^{(14)}}{k}\right] - 1} \quad \text{........(2-30b)}$$

From the additional deformation given, and additional moments can be determined. The additional load on the formwork is

$$p^{(0)}(x,y) = 0 \qquad \text{........(2-31a)}$$

$$p^{(7)}(x,y) = q' \sum \frac{- \frac{\lambda_n^{(7)}}{k} B_n^*}{\left[1 + \frac{\lambda_n^{(7)}}{k}\right]\left[1 + \frac{\lambda_n^{(14)}}{k}\right] - 1} W_n(x,y) \quad \text{....(2-31b)}$$

$n = 1,3,5 \ldots\ldots$

Stage No.7

Twenty-eight days after the beginning of construction, the fifth floor slab is cast. This stage is completely analogous to Stage NO. 5.

Consquently, the expression for $p^{(\omega)}(x,y)$ and $W^{(\omega)}(x,y)$ can generally be written in the form

$$p^{(\omega)}(x,y) = q' \sum C_n^{(\omega)} W_n(x,y) \quad \text{........(2-32)}$$

and $$W^{(\omega)}(x,y) = \frac{q'}{k} \sum D_n^{(\omega)} W_n(x,y) \quad \text{........(2-33)}$$

$n = 1,3,5 \ldots\ldots$

Where the coefficients of expansion $C_n^{(\omega)}$ and $D_n^{(\omega)}$ contain $\frac{\lambda_n^{(\omega)}}{k}$ and A_n only.

This method is consistent and accurate but the large quantity of mathematics involved renders it undesirable in practice.

Grundy and Kabaila[2] developed a simplified method in 1963 which enabled the loads imposed on formwork members and on individual slabs in a system to be determined quite easily. In a typical multistory construction cycle there are two alternating operations which control the loads being applied to the slabs. These are : -

(1) Placing a fresh slab, usually rising at the rate of one floor per week, and

(2) Removing the lovest level of shores, when the youngest floor has the age of 5 days (N=5).

For example, in the single-bay multistory building frame, zero time will be taken at the pouring of the footings. The first floor, second floor and third floor are poured at the time of 7 days, 14 days, and 21 days respectively (operation 1). As there are only three levels of shores to be used, the lowest level of shores (on the ground floor) would be removed to the third floor at the time of 26 days (operation 2). When the props are removed the three floors undergo the same deflection, and therefore the total weight of the three floors will be distributed between them in proportion to their relative stiffnesses. In a set of identical floors the relative stiffness of each will be approximately unity, and therefore each floor at this stage will carry its own weight.

The flexural stiffness of an uncracked section was assumed to be directly proportion to the modulus of elasticity E_c in this analysis.

Typical developments of E_c and concrete crushing strength f'_c in terms of their 28-day values are shown in Fig. 2-2. Grundy and Kabaila[2] have shown that the load ratios obtained based on constant E_c and variable E_c are not greatly different, so the error introduced by the assumption that the relative stiffness of the floors are about equal is not great.

Until the ground shores are removed all the loads are transmitted through the shores to the rigid foundation, as indicated by the condition at 21 days in Fig. 2-4. The numbers in Fig. 2-4 are the load ratios (a factor by which the self weight plus formwork must be multiplied) carried by the slabs or shores at different stages. At 26 days the shore force (3.0) is distributed equally between slabs at levels 1, 2, and 3. At 28 days the weight of the slab at level 4 (1.0) is distributed equally between slabs at levels 1, 2, and 3. At 33 days the shore force at level 1 (0.33) is distributed equally between slabs at levels 2, 3, and 4, and so on.

The analysis above is based on the following assumptions:

1. Three levels of shores to be used.
2. The stiffness of slabs is equal.
3. The props supporting the slabs and formwork may be regarded as a continuous uniform elastic support.
4. The initial slabs are supported from a completely rigid foundation.
5. The shrinkage and creep of the concrete in the slabs is disregarded for analysis and that the slabs behave elastically.
6. Props are infinitely rigid.

Repeating the analysis using two levels of shores, the construction sequences and the load ratios carried by the floors and props are shown in Fig. 2-3.

From the analysis presented above it will be found that the maximum loads are always carried by the last slab cast before the shores at ground level are removed. For example in Fig. 2-4, load ratio 2.36 is carried by the third floor, in Fig. 2-3 load ratio 2.25 is carried by second floor.

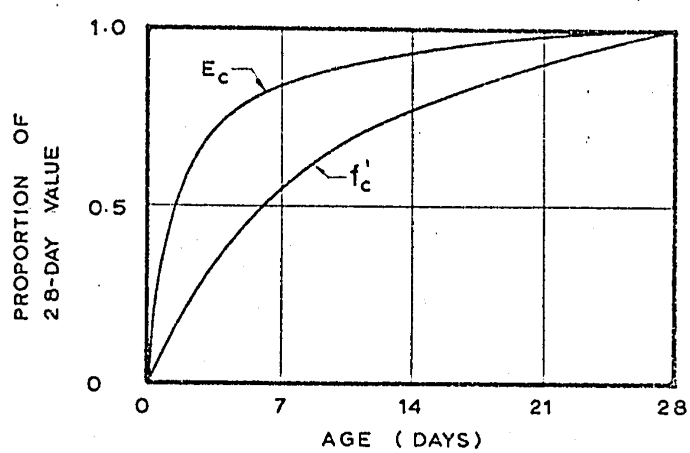

FIG. 2-2 DEVELOPMENT OF E_c & f'_c WITH AGE. (REF. NO. 2)

Table 2-1 shows the analysis results for different number of levels shored. It is evident that when increasing the number of levels shored there is no reduction in the maximum converged load ratios on the slabs. The maximum load ratio actually increases with an increasing number of levels shored. The increasing load ratio may be balanced by the increasing concrete strength due to the increase of age on the slab which carries the maximum load.

Blakey and Beresford[4] have suggested a stepwise construction method in double bays multi-story building frame as shown in Fig. 2-5.

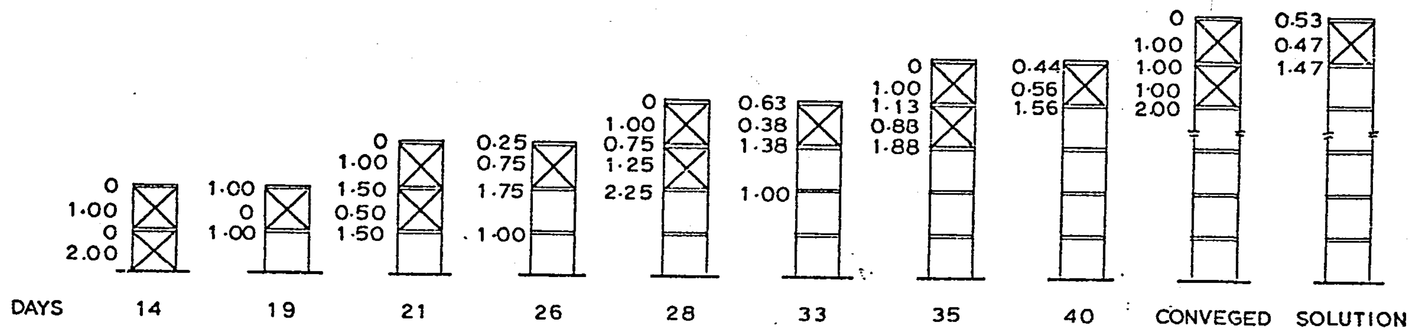

FIG. 2-3 LOAD RATIOS APPLY TO SLAB & SHORES FOR 7-DAY CYCLE, TWO LEVELS OF SHORES (m = 2), N = 5 DAYS & CONSTANT E_c

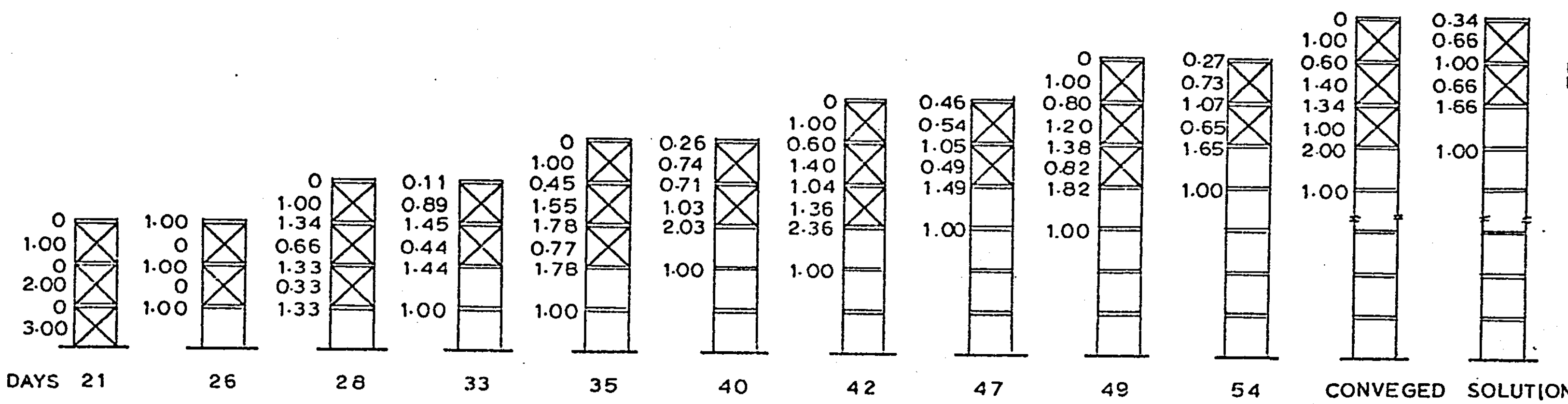

FIG. 2-4 LOAD RATIOS APPLY TO SLAB & SHORES FOR 7-DAY CYCLE, THREE LEVELS OF SHORES (m = 3), N = 5 DAYS & CONSTANT E_c (REF. NO. 2)

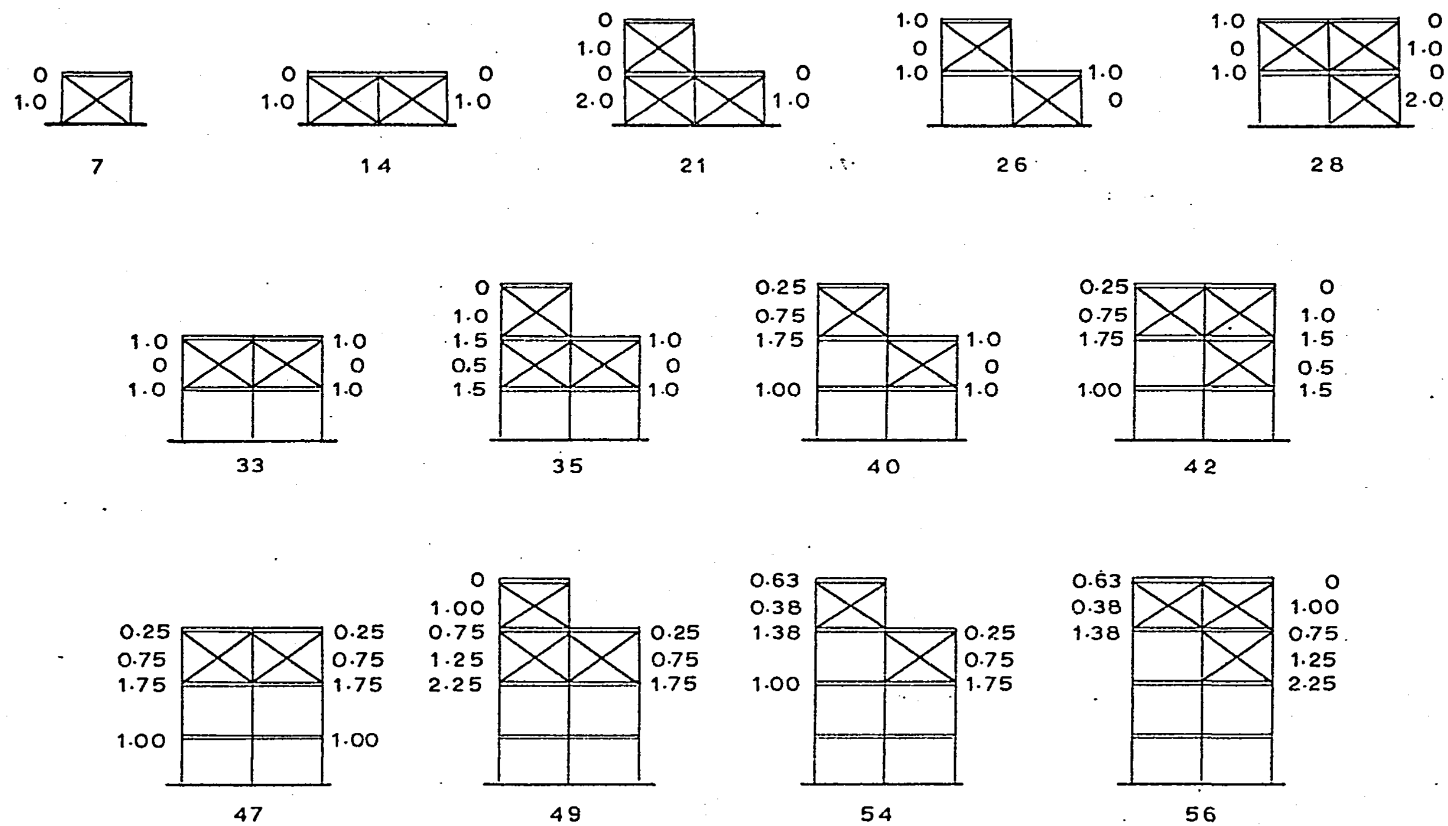

FIG. 2-5 LOAD RATIOS APPLY TO SLAB & SHORES; STEPWISE CONSTRUCTION FOR 7-DAY CYCLE, THREE LEVELS OF SHORES ($m = 3$), $N = 5$ DAYS, & CONSTANT E_c (REF. NO. 4)

No. of Levels Shored		2	3	4	5
Max. Load Ratio		2.25	2.35	2.45	2.50
Slab Supporting Max. Load Ratio	No.	2	3	4	5
	Age (Days)	14	21	28	35
Max. Converged Load Ratio		2.0	2.0	2.0	2.0
First Slab Supporting Max. Converged Load Ratio	No.	4	5	6	7
	Age (Days)	14	21	28	35

Table 2-1 Analysis Results for different No. of Levels Shored. Assumed E_c = Constant, N = 5 days.

The progress in this case is stepwise instead of uniformly vertical. the maximum load ratio carried by a floor is 2.25 , this time on floor aged 28 days (49 days minus 21 days). In this sequence the prop load ratios never rise above 2.0 .

During the time of construction, there is no problem for the props to carry heavy loading because a greater "density" of props can be used. But if a high load ratio is carried by a slab and the self weight of the slab is a high proportion of the total load that the slab is going to carry in service, it will happen that the construction loads are critical in all aspects of the slab design. For example[(2)], normal office construction shows that for a 10-inch thick flat slab designed for a loading of 200 lbs/sq.ft., including 120 lbs/sq.ft. dead load of the slab, the maximum load sustained under construction is 306 lbs/sq.ft. at 21 days, effectively an overload of 60 per cent.(see Table 2-2)

Construction Load lbs. per sq. ft.			Service Load lbs. per sq. ft.	
Item	Typical Floor	Maximum at Level 3	Item	Design Load
10-in. Slab	120	120	10-in. Slab	120
Formwork	10	10	Finishes	15
Subtotal	130	130	Partitions	15
Load Ratio	2.06	2.35	Live Load	50
Total (at 21 days)	268	306	Total (after 28 days)	200

Table 2-2 Construction and Service Loads for Flat Plate Building Shored at Three Levels. (Ref. No.2)

In order to control the construction Loads, some effects of loading on floors have been considered:

(a) Construction cycle.

(b) Creep

(c) Prop spacing.

(d) Reshore tightening.

(e) Prop rigidity.

(f) Use of light weight concrete.

Construction Cycle

Let N represents the time between placing of a fresh slab and the removal of props from above the lowest slab in the system. Values of N used in construction cycles is usually from 1 day to 6 days. The choice of N is based upon the time requirement to remove and reinstall formwork at a higher level. Both the stiffness of a slab and its strength increase with age; however the stiffness increases faster than the strength. Thus initially the load on the slab increases faster than the load capacity of the slab if the construction cycle time is reduced. Thus the advantage of using more levels of shores is to allow the slabs to gain as much strength as possible. Generally the props are left at the lowest level as long as possible after placing a fresh slab.

Creep Deflection

With very early loadings creep deformation may be expected to influence the load ratios. Nielsen(1) has suggested that this factor may be ignored because upper slabs, loaded at a fairly young age and which therefore may be expected to show high creep, are under fairly small stresses, whereas the more highly stressed lower slabs will show lower creep because they are older. It is expected that the high creep and low stress will give a deformation roughly equal to that from a high stress and low creep. Blakey and Beresford(4) did not completely agree with this point, because at the time of removal of the first set of props from the foundation, the assumed load distribution between the floors are equal, according to the analysis given, therefore it should follow that the upper one should creep more because it is younger. Measurements on actual buildings are needed to clarify this point.

Prop Spacing

The spacing of the props must produce sufficient strength to carry all dead load and live construction loads with a factor of safety at least equal to that used in the design of the permanent structure. As the analysis in Fig. 2-3, Fig. 2-4, and Fig. 2-5 the maximum load ratio for props is 3.0, so that the spacing must be designed to carry a load three times the self weight of the slab plus formwork, and should be located in the same position on each floor so that they will be continuous in their support from floor to floor. Care should be used in locating the reshores, so that the loads do not cause excessive punching shear or reversed bending stress in the slab.

Reshore Tightening

During the reshoring operation in site, the reshores are tightened up, the degree of tightening affects the loads in props and thus in the slabs. It is probable that generally the props are not completely tightened, as it seems hard to control the degree of tightening in site. Some of them may be over tightened which will increase the load on the lowest floor. The difference between the analysis results and site condition is hard to estimate reliably. The use of a greater density of props would help to reduce the error. Actual site investigations should be made to find out typical over tighting effects.

Prop Rigidty

The analysis of Grundy and Kabaila(2) assumed that the props are infinitely rigid. Actually in practice the props are not rigid, and this will reduce the loads on the slabs beneath and will increase the load by a corresponding amount on the slabs above. Thus the load ratio distribution may be different and adjustment to the calculated values which based on this assumption is needed.

Use of Light Weight Concrete

For a multistory building construction, the construction load is always above the total design load when the dead load/live load ratio for a floor is high. An overload of 60 per cent above the design load is shown in the previous example (see Table 2-2). This overload of the slab under construction can be reduced by using light weight concrete. A similar slab constructed in light weight concrete would carry a reduced overload of 40 per cent, for the dead load has been reduced. Table 2-3 shows the comparison of the weight of light weight concrete and the regular concrete.

Slab Thickness in.	Slab Thickness ft.	Wt. per sq.ft. 150 pcf Concrete	Wt. per sq. ft. 110 pcf Concrete
3	0.25	37.5	27.5
4	0.33	49.5	36.3
5	0.42	63.0	46.2
6	0.50	75.0	55.0
7	0.58	87.0	63.8
8	0.67	100.5	73.7
9	0.75	112.5	82.5
10	0.83	124.5	91.3
11	0.92	138.0	101.2
12	1.00	150.0	110.0

Table 2-3 Weight of Light Weight Concrete and Regular Concrete. (Ref. No. 12)

Beresford[7] presented in 1964 that the different characteristies of concrete produced the different rate of gaining modulus of elasticity E_c. Generally E_c is not developed exactly the same as the typical case shown in Fig. 2-2. The water-cement ratio, mix proportion and the type of cement will affect the rate of development of E_c. Fig. 2-6

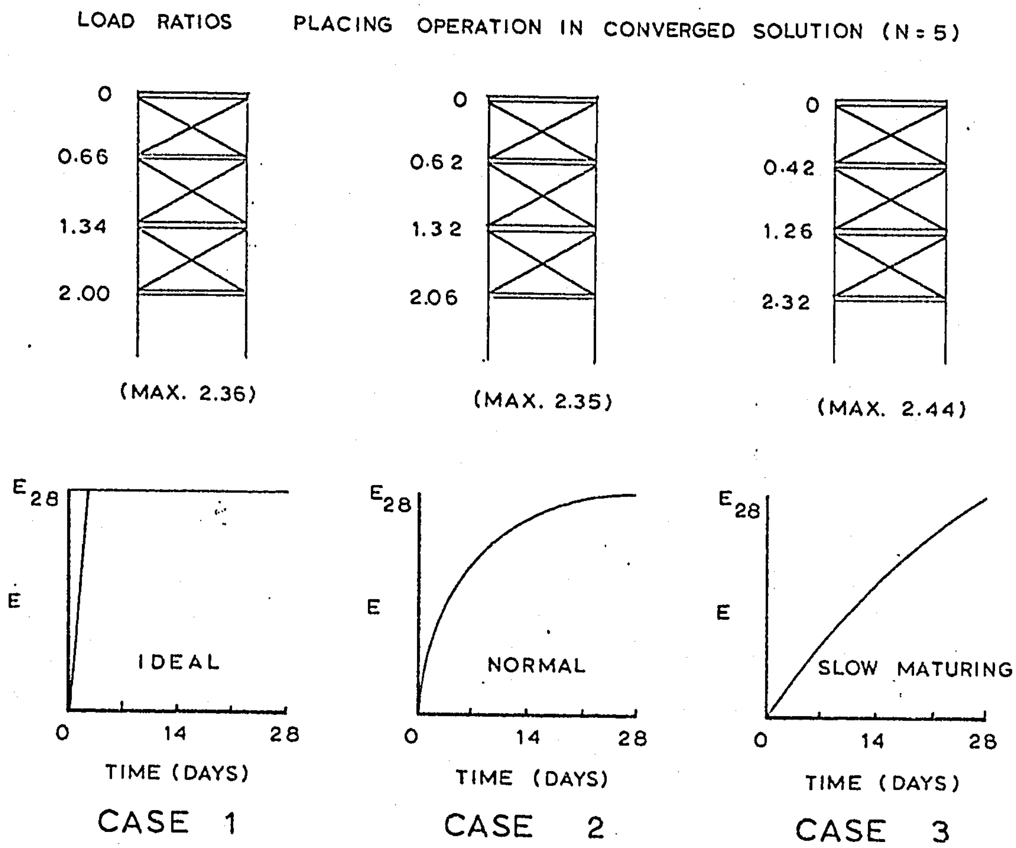

FIG. 2-6 COMPARISON OF CONVERGED SOLUTION (N = 5 DAYS) WITH VARIATION OF MODULUS OF ELASTICITY FOR CONCRETE WITH TIME. (REF. NO. 7)

shows the maximum load ratio and the converged solution with variation of modulus of elasticity for different types of concrete. The first case represents a theoretically perfect concrete which attains full stiffness shortly after placing, for the second case, a normal concrete as would be commonly used, and for third case, a concrete selected as particularly slow in gaining stiffness. The result indicates that there is little difference in the maximum load ratios in the first two cases, and thus the benefits to be gained from high early strength concrete will be due mainly to its improved ability to resist the stresses imposed at the early age. The third case indicates that a significant increase in the load ratio on the lowest floor may occur if the concrete is very slow in maturing. The converged solution is affected more than the maximum value obtained during the analysis, although this value also increases. However the concrete was considered to have extreme behaviour, and would be unlikely to apply except in abnormal conditions. Analysis for the first case would probably be of sufficient accuracy in most cases.

An integral part of this problem, the relation between flexural strength and f'_c was studied by Blakey and Beresford [4]. Crushing strength at 28 days is usually used as the design concrete strength. A typical curve, Fig. 2-2 shows the development of crushing strength of concrete in terms of their 28 days strength with time. That means the concrete strength at the age less than 28 days must be less than the design strength. The flexural strength of reinforced concrete members shall be calculated by the formula in 4.5.4B.9.(1) of the National Building Code of Canada.[16]

$$M_u = \phi \left[bd^2 f'_c \, q \, (1 - 0.59q) \right] \quad \text{................(2-1)}$$

where M_u = Ultimate design resisting moment.

ϕ = Strength variability adjustment factor for flexure use 0.9.

b = Width of the member.

d = Effective depth of the member.

q = $p\, f_y/f'_c$

f_y = Yield stress of steel.

A_s = Area of steel bar.

f'_c = Crushing strength of Concrete.

p = Reinforcement ratio = A_s/bd, for 4.5.4B.9.(2) of National Building Code of Canada.(16)

$$p \leq 0.75\left[\frac{(0.85)(0.85)\, f'_c}{f_y}\left(\frac{87{,}000}{87{,}000+f_y}\right)\right] \quad \ldots\ldots(2\text{-}2)$$

rewrite Eq. (2-1) and Eq. (2-2), assumed f_y = 40,000 psi.

$$\frac{p\, f_y}{f'_c} \leq 0.372 \quad \ldots\ldots\ldots\ldots\ldots\ldots(2\text{-}3)$$

and

$$\frac{M_u}{bd^2} = 0.9\, p\, f_y\left(1 - 0.59\,\frac{p\, f_y}{f'_c}\right) \quad \ldots\ldots\ldots\ldots (2\text{-}4)$$

Fig. 2-7 shows the variation of flexural strength of reinforced concrete members for different levels of tensile reinforcement, with concrete crushing strength. At any level the change in strength of the member for a large change in concrete strength is no more than about ten per cent, so even if the concrete has not reached its design strength at the early age this will not imply any serious encroachment on the factor

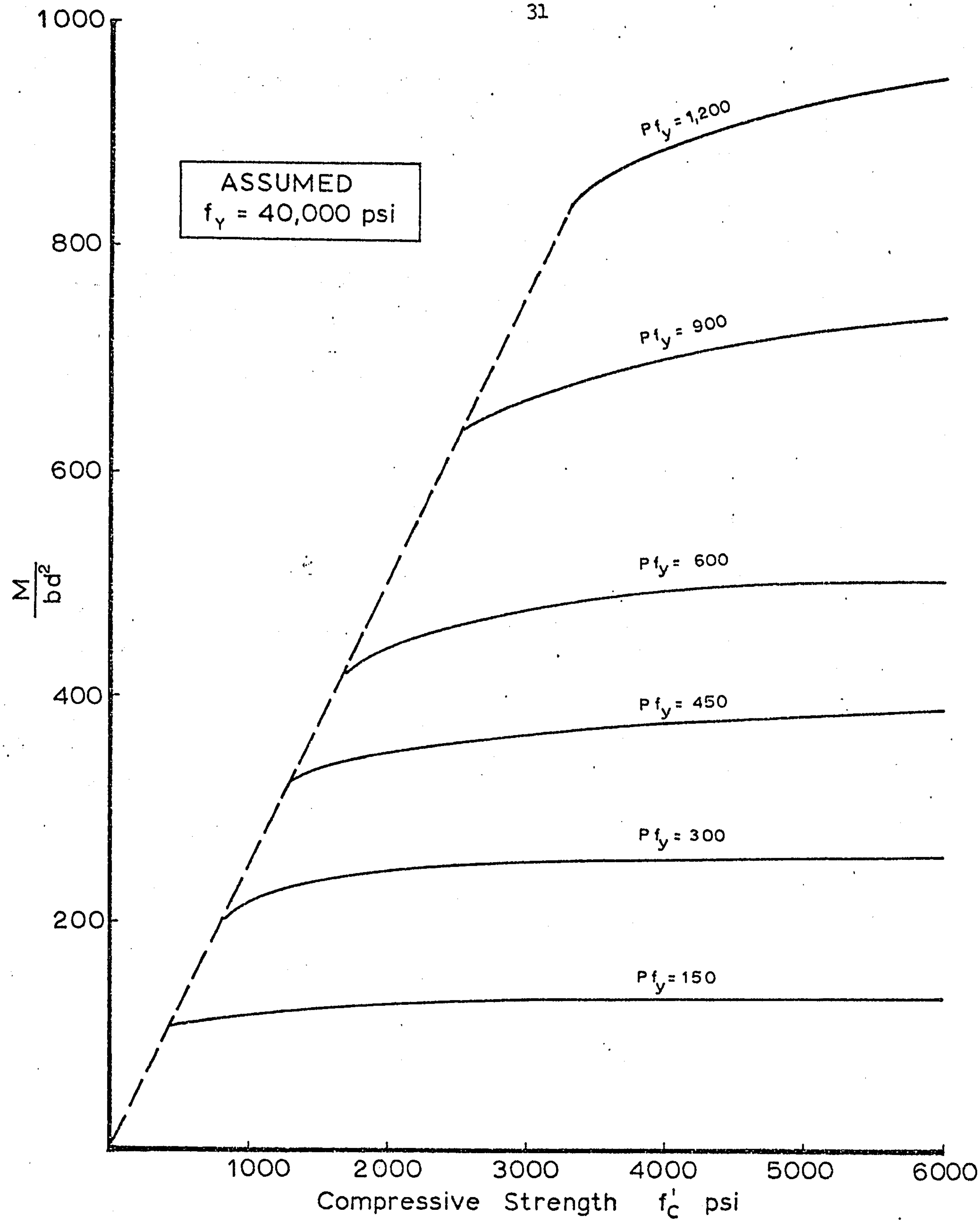

FIG. 2-7 VARIATION OF BEAM OR SLAB FLEXURAL STRENGTH WITH COMPRESSIVE STRENGTH f'_c

$$\frac{M}{bd^2} = 0.9\,p f_y\left(1 - 0.59\frac{p f_y}{f_y}\right) \quad \text{FOR} \quad \frac{p f_y}{f_y} < 0.372$$

of safety against flexural failure.

Taylor[11] presented a method to reduce the load ratio in slabs by employing a special technique into the formwork stripping program. This technique is slackening and tightening the shores by rotating the threaded collar on the middle part of the props. The operations are shown in Fig. 2-8 and describes in Table 2-4.

Time (Days)	Operations
21	Pour level 3.
27	Remove lowest shoring and place for level 4 slab.
28	Pour level 4.
34	Remove lowest shoring and place for level 5 slab.
	Slackening the shores under level 4 (by ratating the threaded collar on the props), then under level 3. Tightening the props which had been slacken.
35	Pour level 5.
Repeat procedures at 34 and 35 days for upper levels.	

Table 2-4 Operations of Slackening and Tightening in The Reshoring Program. (Ref. No.11)

Using the assumptions adopted by Grundy and Kabaila[2] and the operations above, the maximum load ratio is 1.44 at a concrete age of 21 days as shown in Fig. 2-8 giving a construction load of 187 lbs. per sq. ft. which has been reduced about 38% for the slab of Table 2-2. When the shores slackened each slab took their own weight, and no load from the upper slab would transfer to the lower slabs throught the shores. After the shores were tightened up and a new slab was poured to the top,

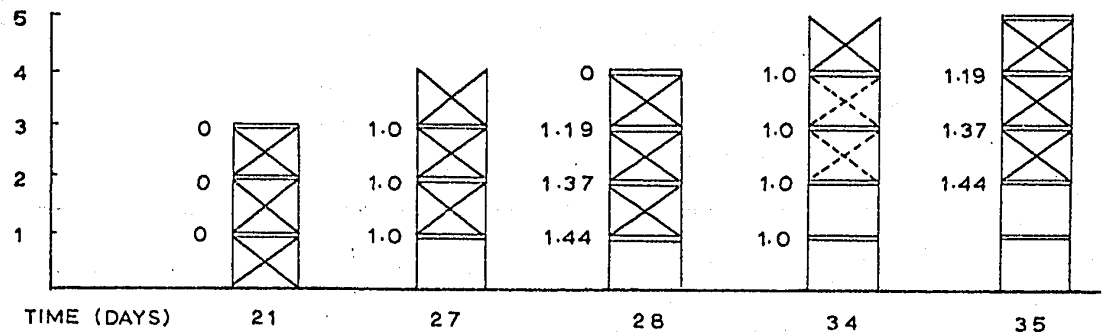

FIG. 2-8 SLAB LOAD RATIOS FOR TAYLOR'S SHORING METHOD ASSUMING A 7-DAY CYCLE, THREE LEVELS OF SHORES (m = 3), N = 6 DAYS AND VARIABLE E_c. (REF. NO. 11)

the load ratios distributed on the supporting floors are practically the same as at 28 days (1.19, 1.37, 1.44). The load ratios calculated above are not as severe as for the method presented by Grundy and Kabaila[(2)], but for standard mix concrete may still exceed allowable design stresses for concrete at that age. In order to avoid the concrete overstress, Taylor[(11)] suggested that:

(i) Using a concrete that will attain sufficient strength to carry the shoring loads without overstress. (such as using the high early strength cement in the concrete mix)

(ii) Using a greater number of levels of shoring, then the maximum load is applied to a older floor with higher strength.

(iii) Increasing the time cycle, so the concrete has more time to gain higher strength to support the construction load.

Chapter 3

SITE INVESTIGATION

The object of this investigation was to determine the typical loads in shores during construction. The measurement were carried out at block A of The Engineering Building of University of Ottawa from 27th Feb. 1970 to 13th Mar. 1970. This is a flat slab type building of the type commonly used for offices and appartments. The temperature during this period was approximately 5°F below. Because of the cold weather the construction rate was approximately 14 days per floor. The thickness of the flat slab is 8" and has a maximum panel length of twenty feet. The details of the design of block A is shown in Appendix E. Fig. 3-2 shows the shape and dimensions of the shoring unit used to support the slab in construction. The shoring plan on the 7th floor is shown in Fig. 3-1.

The first part of this measurement was to determine the loads in the shoring unit when a fresh slab was cast. The shaded shoring unit in Fig. 3-1 placed between the 7th floor and the 8th floor, was instrumented. Six posts of this shoring unit were named A, B, C, D, E, and F. Since the post might be bent under loading, in order to minimize the error in measuring, two Electrical Resistance Foil Type Strain Gages wired in series were affixed on the lower part of each named post, as shown in the detail of Fig. 3-2. By recording the initial reading from the indicator before casting and comparing it with the reading after the upper floor was completed, the loading at each named post was measured and are listed in Table 3-1. The theoretical loading carried by the post was calculated from the unit weight acting over the dotted area shown in

Fig. 3-1. Since the slab of the 8th floor is 8" thick with an approximated dead weight of 100 lbs. per sq. ft., The expected loading for each named post can be determined and shown in Table 3-1.

Post	Expected Load	Measured Load
A	1750 lbs.	940 lbs.
B	2400 lbs.	663 lbs.
C	2190 lbs.	4000 lbs.
D	2190 lbs.	2750 lbs.
E	2400 lbs.	5200 lbs.
F	2350 lbs.	1715 lbs.

Table 3-1 Expected and Measured Loadings at One Shoring Unit between The 7th Floor and The 8th Floor of The Engineering Building of University of Ottawa.

The second part of this investigation was to determine the loads in the shorings between a number of floors below the freshly placed floor. Fig. 3-3 shows the arrangement of shores and their numbers. Shores No. 1 and No. 2, No.3 and No. 4 were numbers of the shoring unit on the 8th floor and the 7th floor respectively. Whereas NO. 5 and No.6 were the only two props set up by the author on the 6th floor. Again two electrical resistance strain gages wired in series were put on the lower part of each shore. The measured results are shown in Table 3-2. The expected load ratio carried by the posts between the floors can be obtained by the method which considered the change of EI with ages as presented by Grundy and Kabaila[2] is shown in Table 3-2. Both measured and calculated load ratios on floors and post are shown in Fig. 3-4.

Posts On	Post	Load	Average	Measured Load Ratio	Calculated Load Ratio
8th Floor	1 2	2780 lbs 2170 lbs	2475 lbs	1.0	1.0
7th Floor	3 4	710 lbs 729 lbs	720 lbs	0.29	0.69
6th Floor	5 6	146 lbs 109 lbs	127.5 lbs	0.05	0.345

Table 3-2 Measured Results for Three Levels of Shores

Comparison of the Computed values and the measured values in Table 3-1 and Table 3-2 show no agreement at all. The reasons for causing these differences can be explained as follows:

1. Since the shores do not provide a continuous elastic support the load distribution will differ from that calculated.
2. The reliability of the strain gages may have been reduced due to working under very cold conditions. This could be due to the properties of the adhesive changing, etc.
3. The analysis of Grundy and Kabaila[2] assumed that the props are rigid. Naturally in practice the props are not rigid and this will cause a transfer of load to the upper floors.

Chapter 4

TESTING PROCEDURE AND RESULTS

To supplement the field investigation it was decided to investigate in the laboratory the rate of gain of cylinder strength and modulus of elasticity and the rate of gain of flexural strength and stiffness of model reinforced concrete beams with age.

Cylinder Tests

The object of this investigation was to determine experimentally, the variation in the compressive strength f'_c and the modulus of elasticity E_c of concrete at different ages. Two groups of 6"x12" cylinders were made with different strengths in this test. They were designed to carry compressive stresses of 3600 psi. and 6000 psi. respectively. The cement used in this test was "high early strength Portland Cement". The sand had an average fineness modulus 2.5, and the maximum size of coarse aggregate was 3/8". The mixing proportion per cu.ft. by weight is listed in Table 4-1.

	Dry Weight (lbs.)	
	Groups 3600 psi.	Group 6000 psi.
Cement	24.6	35.4
Sand	42.5	48.9
Stone(3/8")	65.0	44.0
Water	12.6	14.2

Table 4-1 Mixing Proportion per cu. ft. of Concrete.

The cylinders were cast in Sono molds and compacted by tamping. Thirty 6"x12" cylinders were cast at the same time to form a group.

Twenty-four hours after the casting, the specimens were removed from the modes and cured under the water tank at a temperature of about 70°F.

The tests were carried out at the concrete laboratory of the University of Ottawa. Each group of cylinders was divided into six batches (each batch containing five cylinders) and tested at ages of 1 day, 4 days, 7 days, 14 days, 21 days, and 28 days respectively. At the time of testing, five cylinders were taken out from the water tank and were allowed to dry for an hour. The end faces of the cylinders were capped with standard-sulpher compound.

The modulus of elasticity is the ratio of stress to strain but unlike most metals, concrete has no true "straight line" portion of its stress-strain curve. Since there is no true straight-line portion, there can be no true proportional limit and the modulus will be different at any selected stress point. Three methods of expressing and calculating the modulus of elasticity have been proposed. These are known as "chord modulus", "secant modulus", and "tangent modulus". Different values of modulus of elasticity may be determind from one test if different methods of calculation has been used. Each modulus of elasticity value may be represented by the slope of the appropriate line shown in Fig. 4-1.

In this test secant modulus was used. In order to determine the stress-strain curve, a dial gage frame was clamped on the cylinder as shown in Fig. 4-2 and Fig. 4-3. Tests were carried out in a Forney Compression Testing Machine having a capacity of 300 kips. shown in Fig. 4-4. The movement of the gage frame is measured by dial gages and the strain is calculated by dividing the total measured movement by the gage lengths. As the strains at various loadings are obtained the stress-

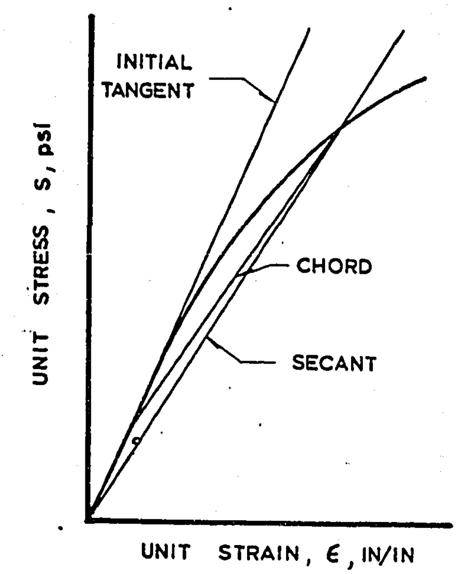

FIG. 4-1 TYPICAL STRESS-STRAIN CURVE FOR CONCRETE. (REF. NO.12)

strain curve can be established. Fig. 4-5 shows a typical stress-strain curve for concrete cylinders.

Table 4-2 contains results of the compressive strength f'_c and modulus of elasticity E_c of the 3600 psi. specimens and the 6000 psi. specimens at the different ages. The calculation of E_c in this test was the mean of the slopes which calculated from every measurement point (up to about 40% of ultimate stress) in the stress-strain curve. Calculations are shown in Appendix A. From ACI Building Code 318-51[15],1102(a) we can calculate the modulus of elasticity E_c, for concrete by using the formula:

$$E_c = 33\, w^{1.5} \sqrt{f'_c} \quad ....(4\text{-}1)$$

where w is the unit weight of the concrete. The average unit weight of the cylinders was 150.5 lbs. per cu. ft. which has been substituted into Eq. 4-1 and the results are shown the Table 4-2. Calculations are shown in Appendix A. Fig. 4-6 and Fig. 4-7 show the measured E_c and the E_c which calculated by Equation 4-1. From Fig. 4-7 no significant differences is found between measured E_c and computed E_c.

Group	Age (Days)	f'_c psi.	f'_c Proportion of 28-day Value %	E_c psi.	E_c Proportion of 28-day Value %	E_c Calculated by $w^{1.5}\ 33\sqrt{f'_c}$
3600 psi.	1	2336.28	42.5	3,842,000	77.9	2,970,000
	4	3610.62	69.5	4,460,000	90.0	3,690,000
	7	4329.20	78.9	4,537,000	91.5	4,040,000
	14	4750.44	86.5	4,745,000	95.5	4,230,000
	21	5168.14	94.4	4,882,000	98.5	4,400,000
	28	5497.34	100.0	4,951,000	100.0	4,550,000
6000 psi.	2	4481.41	59.1	4,475,000	83.2	4,110,000
	4	5253.10	69.4	4,877,000	90.7	4,450,000
	7	6407.08	84.5	5,001,000	93.0	4,910,000
	14	6605.93	87.0	5,288,000	98.2	5,000,000
	21	7327.43	96.6	5,332,000	99.1	5,250,000
	28	7592.92	100.0	5,372,000	100.0	5,350,000

Table 4-2 Compressive Strength f'_c and Modulus of Elasticity E_c of Group 3600 psi. and Group 6000 psi. Cylinders in Different Ages

Fig. 4-8 and Fig. 4-9 represent graphically the results of compressive strength f'_c and modulus of elasticity E_c in terms of 28-day value. From Fig. 4-8 and Fig. 4-9 it can be seen that the modulus of elasticity of concrete E_c increases with age as well as the crushing strength f'_c, although at a different rate and E_c develops more rapidly and is there after more constant than f'_c.

Fig. 4-10 and Fig. 4-11 show that the development of f'_c and E_c for the 6000 psi. specimens is faster than that for the 3600 psi. specimens.

Medel Beam Tests

The object of this investigation was to find out how age affects the stiffness (EI) and ultimate strength of simple supported beams.

Two specimen forms were made of steel, each form being able to cast 10 beams. This was advantageous because it was thus possible to cast twenty beams at the same time. These steel forms are shown in Fig. 4-13. The vertical sides of the steel forms were bolted solidly to the base. To prevent adhesion between the inside surface of the forms and the specimens, the inside surface of the forms were coated with mould oil.

The cement used in this test was ordinary Portland Cement, the sand had an average fineness modulus 2.5 and no coarse aggregate was used in this cement sand mortar. The mix proportions per cubic foot by weight were:

Dry Weight

Cement	48.4	lbs.
Sand	77.4	lbs.
Water	22.0	lbs.

The reinforcement used in this test was 1/8" diameter Cold Rolled Mild Steel SAE 1020 which has the stress-strain curve shown in Fig. 4-14, the strength characteristic from the tension tests are listed in Table 4-3.

No.	f_y ksi.	$f_{ult.}$ ksi.	Modulus of Elasticity E x 10^6 psi.
1	86.0	90.0	29.0
2	85.8	89.5	29.1
3	86.5	88.7	29.0
4	85.4	86.5	29.5
5	86.0	89.0	29.1
Average	86.0	88.74	29.1

Table 4-3 Yield Stress, Ultimate Stress and Modulus of Elasticity of 1/8" diameter Cold Rolled Mild Steel SAE 1020

For anchorage, two aluminum plates were placed at the ends of the beam's mould, the steel bar with threaded ends were fixed to these two plates with nuts as shown in Fig. 4-12. After the reinforcements and end plates had been set-up, twenty beams and twenty 2" cubes were cast with cement sand mortar. Twenty-four hours later the beams and cubes were removed from the molds and cured in water.

In order to know the rate of gain of flexural strength and stiffness of the beams, the twenty beams and associated twenty cubes were divided into four groups (each group had five beams and five cubes) and were tested at the ages of 7 days, 14 days, 21 days and 28 days respectively. At the time of testing five beams and five cubes were taken out from the water tank and were allowed to dry for an hour before testing. A thin layer of Plaster of Paris brushed over the surface of the beams so that hair cracks could be seen easily during the test.

The beams were simply supported over a span of 2' - 9" and loaded by two symmetrical point loads which acting separately at one-third of the span from both end supports. To prevent crushing the concrete, bearing plates were put on the beams at the supports and loading points. Load was applied by the loading head of the machine through the loading beam. Fig. 4-15 and Fig. 4-16 show the set-up while the beams were testing. A sketch of the test set-up is given in Fig. 4-17. The time from the beginning of the test to failure of a specimen was approximately thirty minutes. The required measurements of this test were the applied loads, the corresponding deflections at the center of span, ultimate loads of the reinforced beams and the location and extent of cracks.

The five cubes which were cast at the same time with the beams, were tested to determine the ultimate compressive strength by placing in the testing machine and tested to failure.

Fig. 4-18 to Fig. 4-21 show the location and extend of the cracks of the failed beams. Fig. 4-22 to Fig. 4- 25 show the relations between loading P and the central deflection of beams at different ages.

Table 4-4 represents the results of compressive strength f'_c, stiffness EI, and ultimate load P_u of the beams. All the results above are average of five readings from the tests at the same age. The calculations of stiffness of the beams are shown in Appendix B. The ratios of $\frac{P_u}{EI}$ of the beams are almost unity as shown in Table 4-4.

The test results of P_u, f'_c and stiffness EI are plotted in terms of their 28-day values as shown in Fig. 4-26. It can be seen that the ultimate load P_u of the beams increases with ages as well as the stiffness EI of the beams, and they almost have the same rate of development.

In order to study the reliability of the results, the theoretical values of ultimate load P_u and stiffness EI of the beams at different ages are calculated and shown in Table 4-5. The calculations of the theoretical ultimate load P_u are based on the text written by Winter[14] (see Appendix C). P_u was calculated assuming that the beam failed either by flexure or shear and both results are presented in Table 4-5. The calculations indicated that most beams should have failed in shear and this is confirmed by observation of the crack pattern. (see Fig. 4-18 to Fig. 4-21)

The moment of inertia of the cracked and uncracked section of the beams were evaluated and are shown in Appendix D. The values of stiffness EI of the beams, calculated assuming either that the section was uncracked or a cracked section, are shown in Table 4-5. Comparison of the test results shows no great difference between the EI calculated on the basis of a cracked section and the measured EI.

Days	f'_c		P_u		EI		$\frac{P_u}{EI}$
	psi.	% of 28-day Value	lbs.	% of 28-day Value	#-in^2	% of 28-day Value	
7	4,688.8	69.8	259.31	79.2	1,550,000	81.0	1.671
14	5,620.0	83.7	285.75	87.2	1,680,000	87.8	1.700
21	6,163.1	91.5	302.10	92.2	1,810,000	94.5	1.670
28	6,721.8	100.0	327.30	100.0	1,915,000	100.0	1.710

Table 4-4 Results of Compressive Strength f'_c, Ultimate Load P_u and Stiffness EI of The Beams at Different Ages

Days	E_c psi.	I in^4		EI #-in^2			P_u #		
	$5{,}750\sqrt{f'_c}$	Uncracked	Cracked	Calculated		Measured	Calculated		Measured
				Uncracked	Cracked		Flexure	Shear	
7	3,940,000	0.4961	0.3934	1,955,000	1,550,000	1,550,000	251	261	259.31
14	4,310,000	0.4887	0.3845	2,105,000	1,660,000	1,680,000	314	285	285.75
21	4,500,000	0.4879	0.3805	2,200,000	1,710,000	1,810,000	326	299	302.10
28	4,700,000	0.4828	0.3770	2,265,000	1,780,000	1,915,000	332	313	327.30

Table 4-5 Results of Calculated and Measured Values of Modulus of Elasticity of Concrete E_c, Moment of Inertia of Model Beam Section, Stiffness of Beam, and Ultimate Load P_u at Different Ages.

Chapter 5

CONCLUSIONS AND RECOMMENDATIONS

The elementary analysis presented in this report provides a basis for determining the loads which might be applied to concrete floors during construction, by props from floors above. This construction load can exceed the design load in the multistory flat slab building construction, if the dead load of a floor is about half the total design loads.

The principal conclusions from this experimental study are:

(1) The cylinder test results show that the modulus of elasticity E_c varies as the square root of the compression strength f'_c, as suggested by the ACI Building Code (1963).

(2) The failure of model beams were well predicted by the standard prediction formulae which can be used for the analysis of construction load carrying capacities.

(3) Ultimate load P_u of the beams increases with age as well as the stiffness EI of the beams, and has almost the same rate of development.

(4) The ratios of $\frac{P_u}{EI}$ at different age of beams are equal.

(5) Increasing the number of proped floors or the time between placing of fresh slabs to allow the concrete a longer time to attain strength may not be considered economical. The development of strength may not be great enough to compensate for the disadvantages.

(6) Investigating the methods of control the of construction loads two methods can be recommended; (a) reduce the dead load/live load

ratio. and (b) employ the technique of slackening and tightening the shores during the time of construction.

For future research, estimating the calculated load ratios must be on the basis of further experimental measurements in the field. The most important points are: the examination of prop forces and slab deflections during the building of typical structures.

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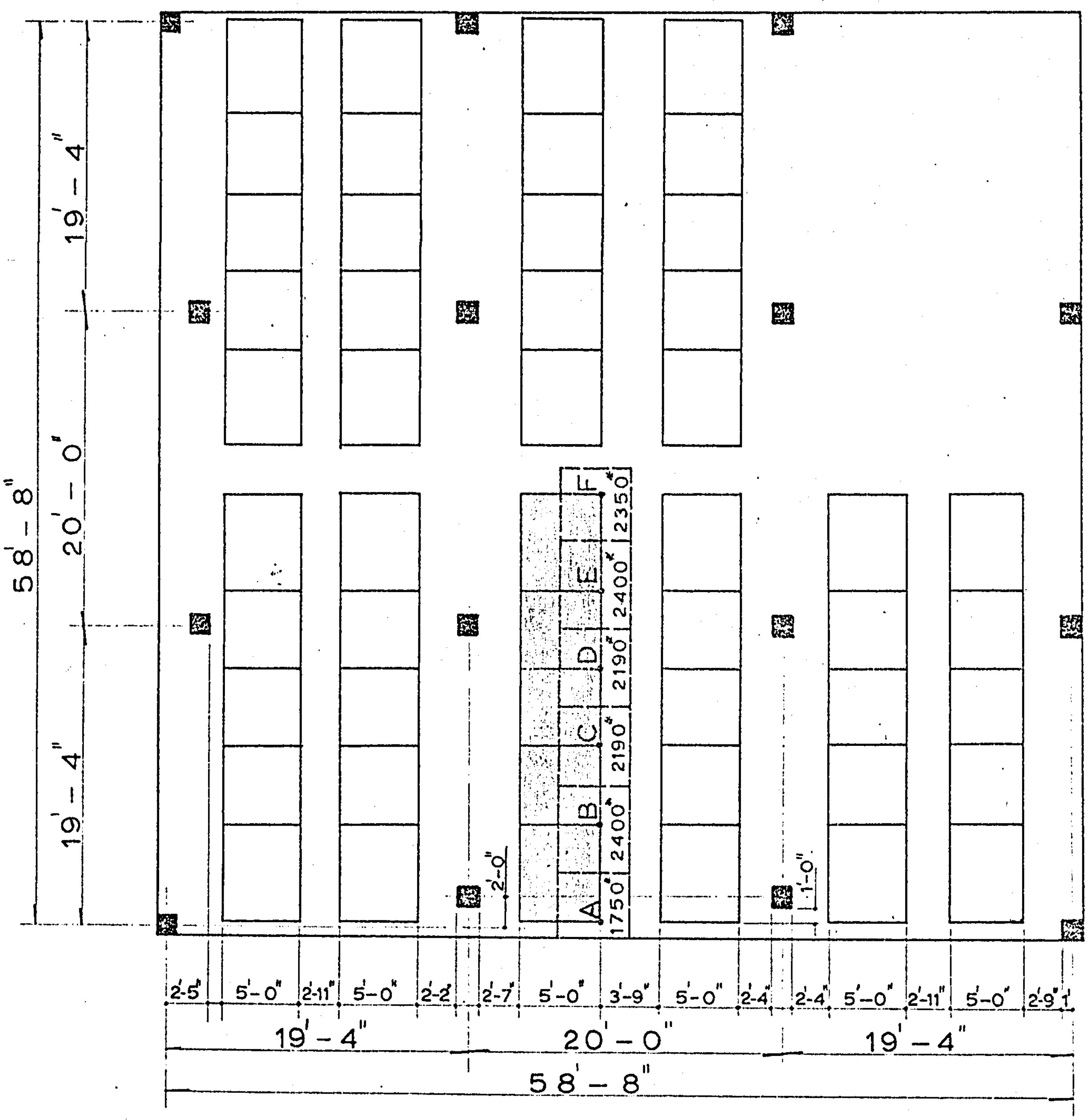
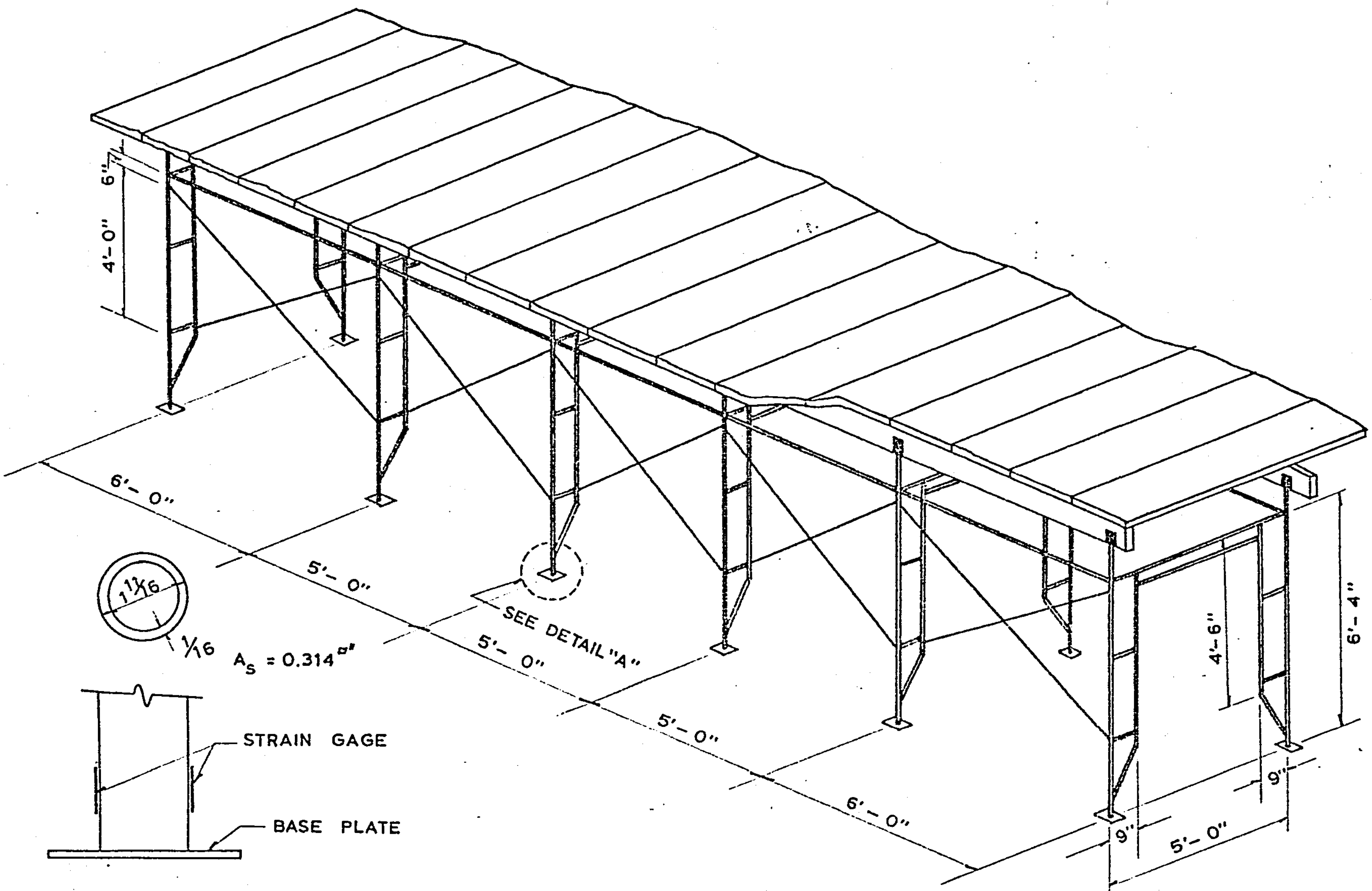


FIG. 3-1 PART OF 7TH FLOOR PLAN OF BLOCK A IN THE ENGINEERING BUILDING OF UNIVERSITY OF OTTAWA AND THE SHORING UNIT ARRANGEMENT



DETAIL "A"

FIG. 3-2 SHORING UNIT DETAIL

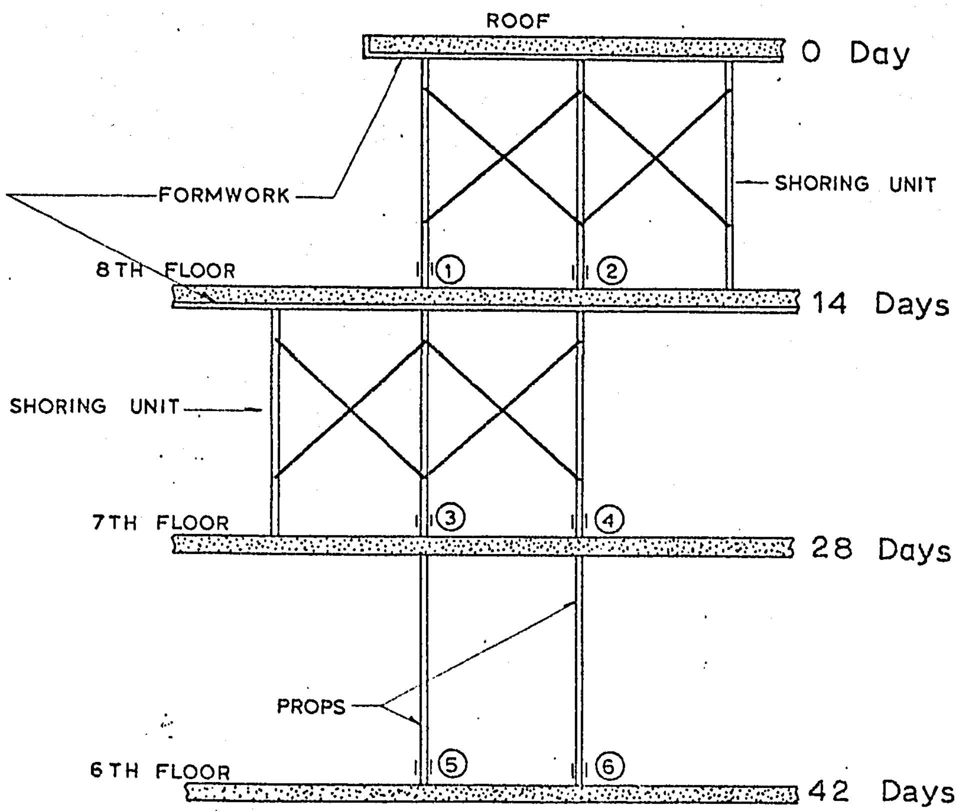


FIG. 3-3 ARRANGEMENT OF SHORES

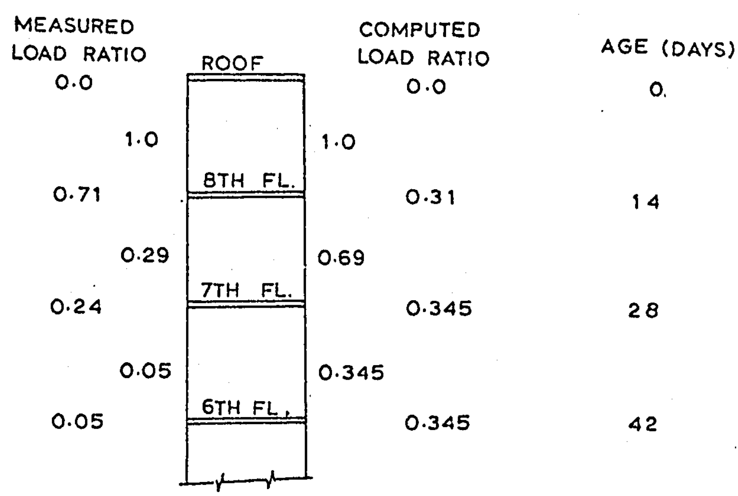


FIG. 3-4 LOAD RATIOS IN SHORES AND FLOORS AFTER THE ROOF WAS POURED.

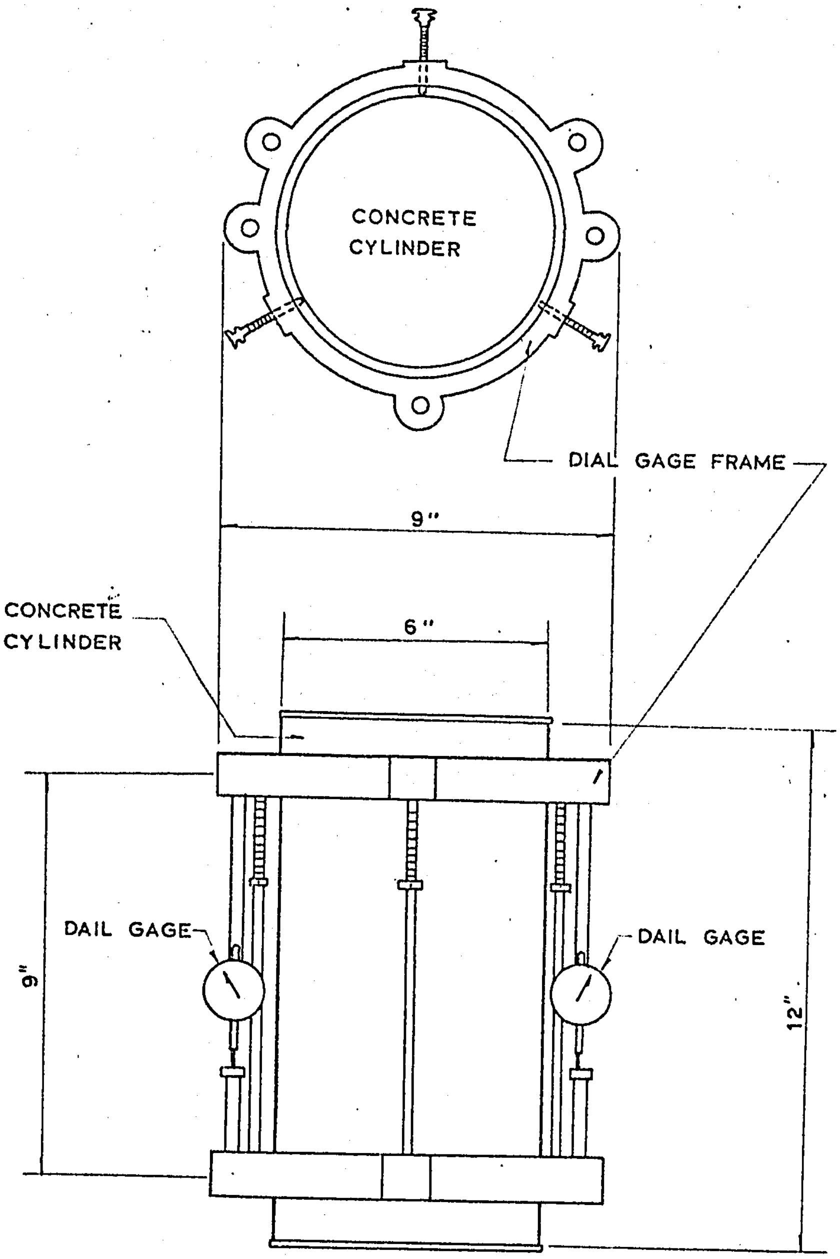


FIG. 4-2 DIAL GAGE FRAME

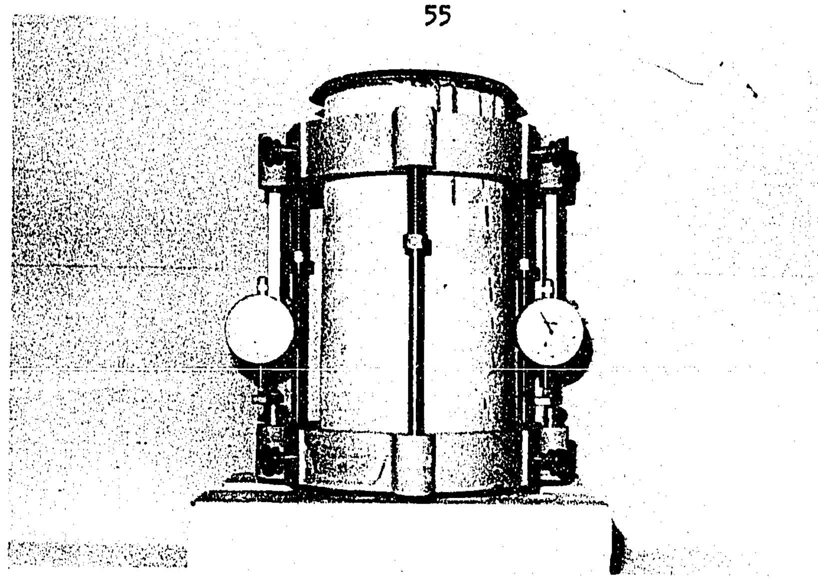


FIG. 4-3 GENERAL VIEW OF CYLINDER TEST

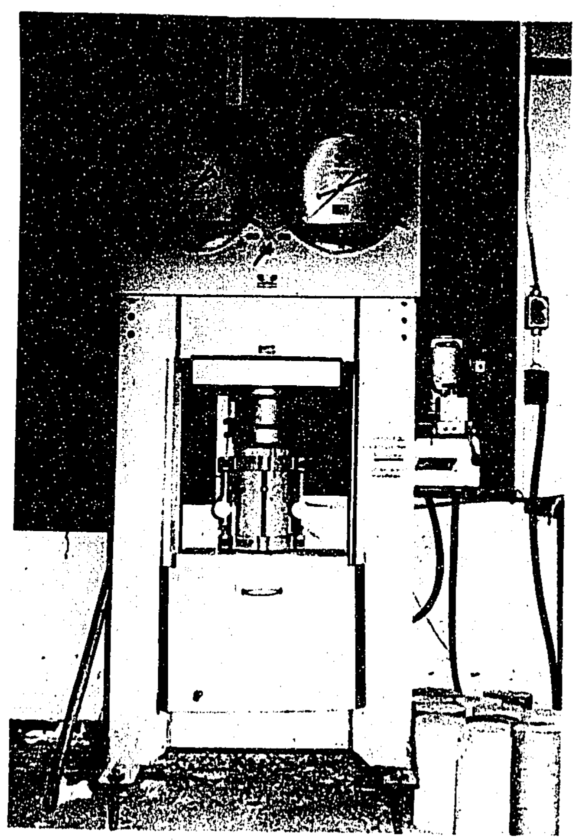


FIG. 4-4 TEST ARRANGEMENT FOR CYLINDER COMPRESSIVE TEST

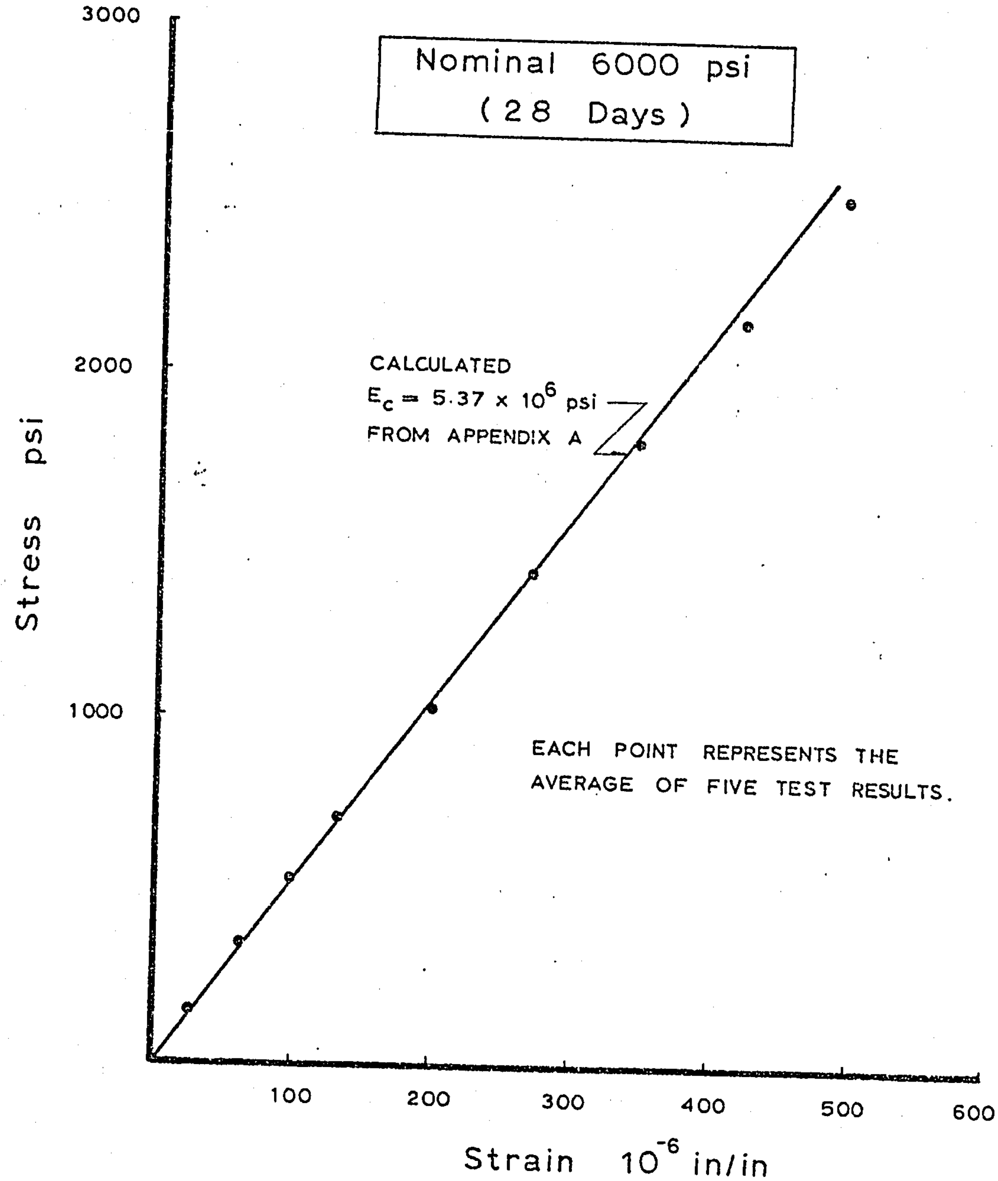


FIG. 4-5 COMPRESSIVE CHARACTERISTICS OF CONCRETE CYLINDER

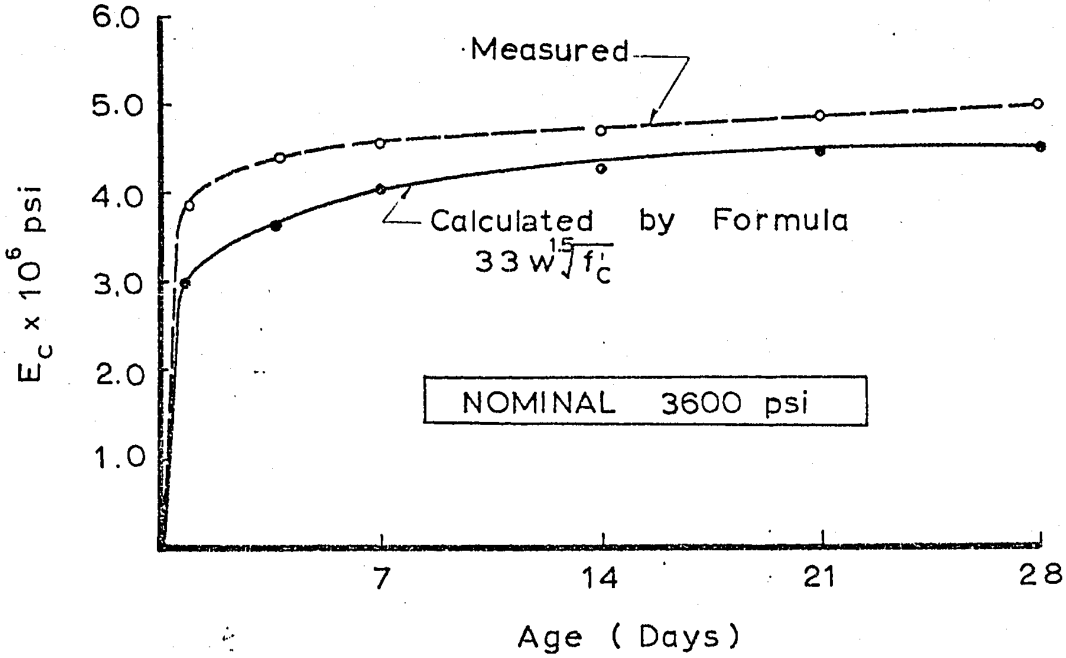


FIG. 4-6 MEASURED E_c & COMPUTED E_c AT DIFFERENT AGES

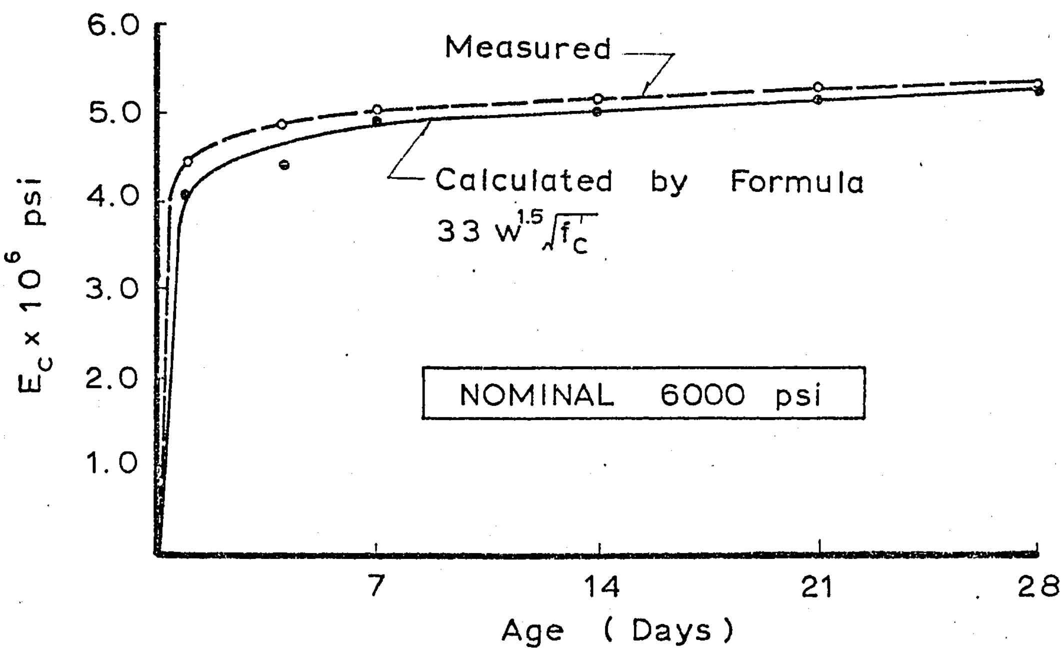


FIG. 4-7 MEASURED E_c & COMPUTED E_c AT DIFFERENT AGES

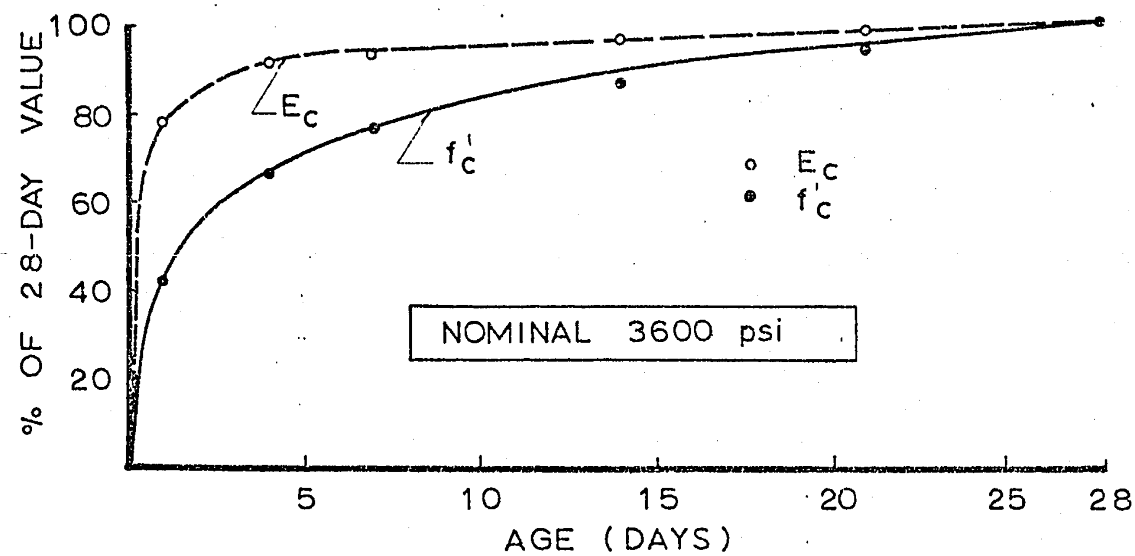


FIG. 4-8 TESTING RESULTS OF MODULUS OF ELASTICITY & COMPRESSIVE STRENGTH IN TERMS OF 28-DAY VALUE.

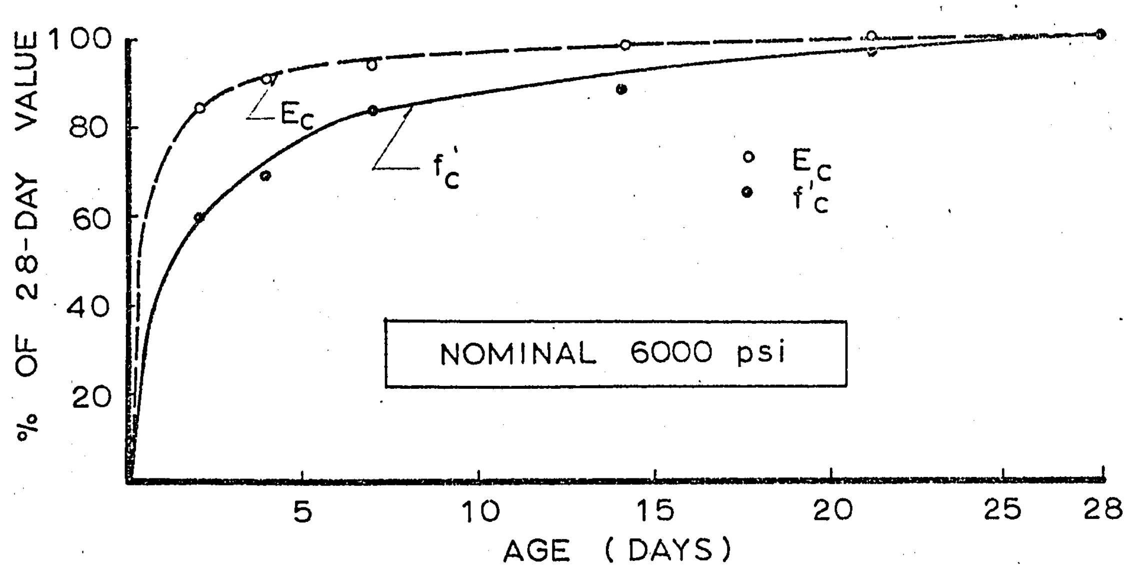


FIG. 4-9 TESTING RESULTS OF MODULUS OF ELASTICITY & COMPRESSIVE STRENGTH IN TERMS OF 28-DAY VALUE

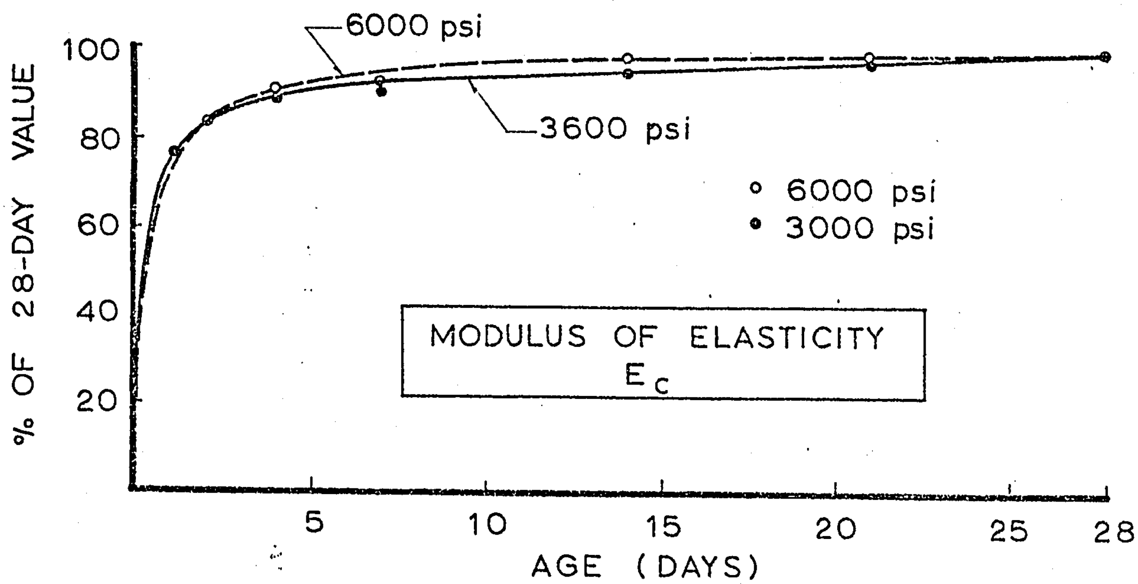


FIG. 4-10 MODULUS OF ELASTICITY RESULTS OF THE 3600 psi & 6000 psi SPECIMENS IN TERMS OF THEIR 28-DAY VALUE.

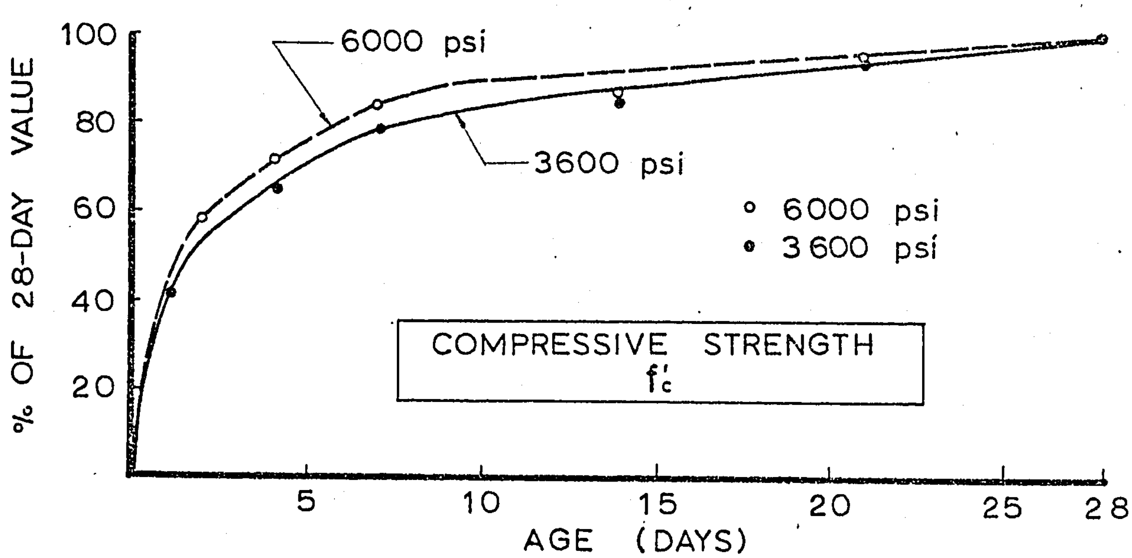


FIG. 4-11 COMPRESSIVE STRENGTH OF THE 3600 psi & 6000 psi SPECIMENS IN TERMS OF THEIR 28-DAY VALUE.

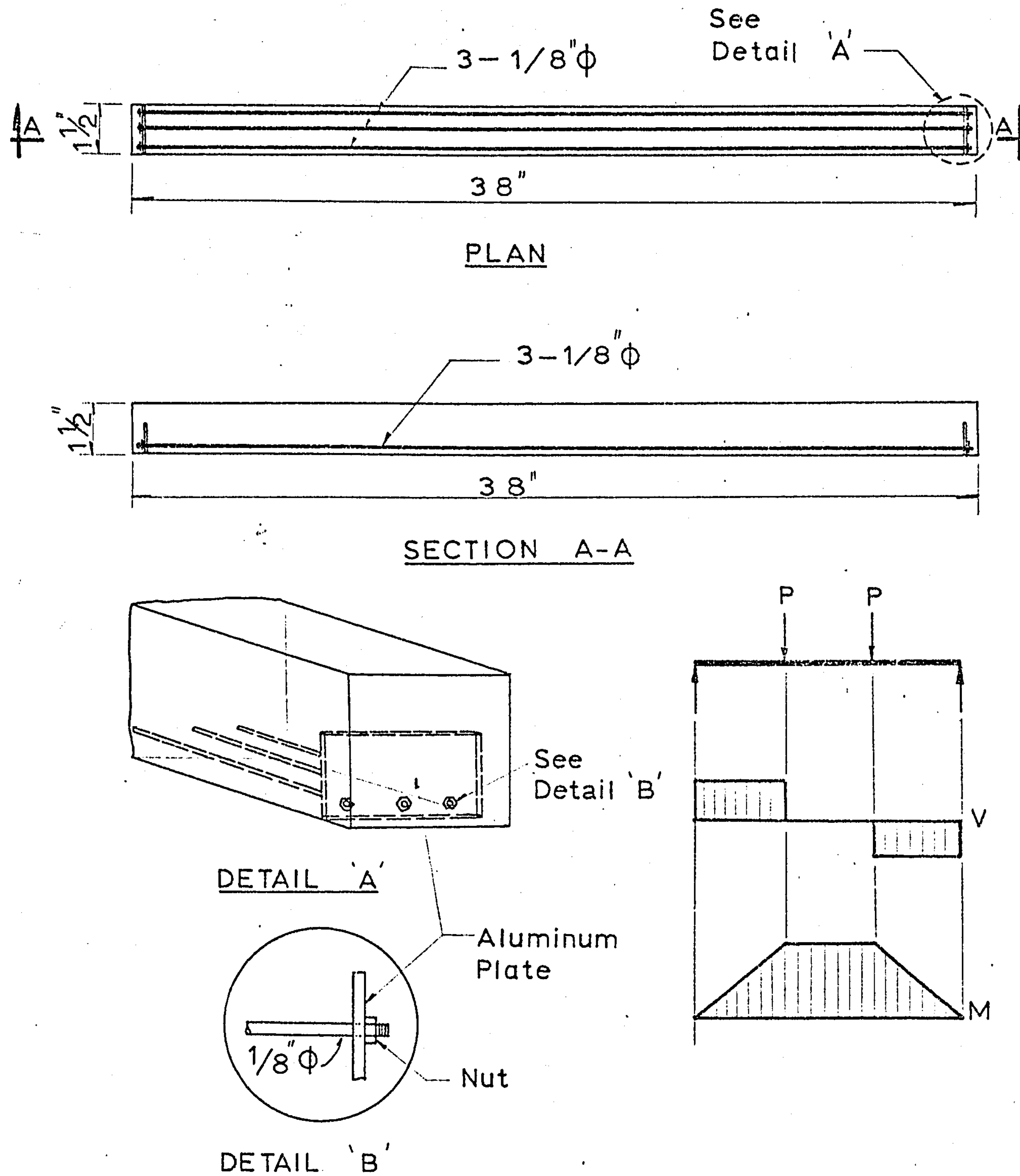


FIG. 4-12 SIMPLE BEAM DETAIL

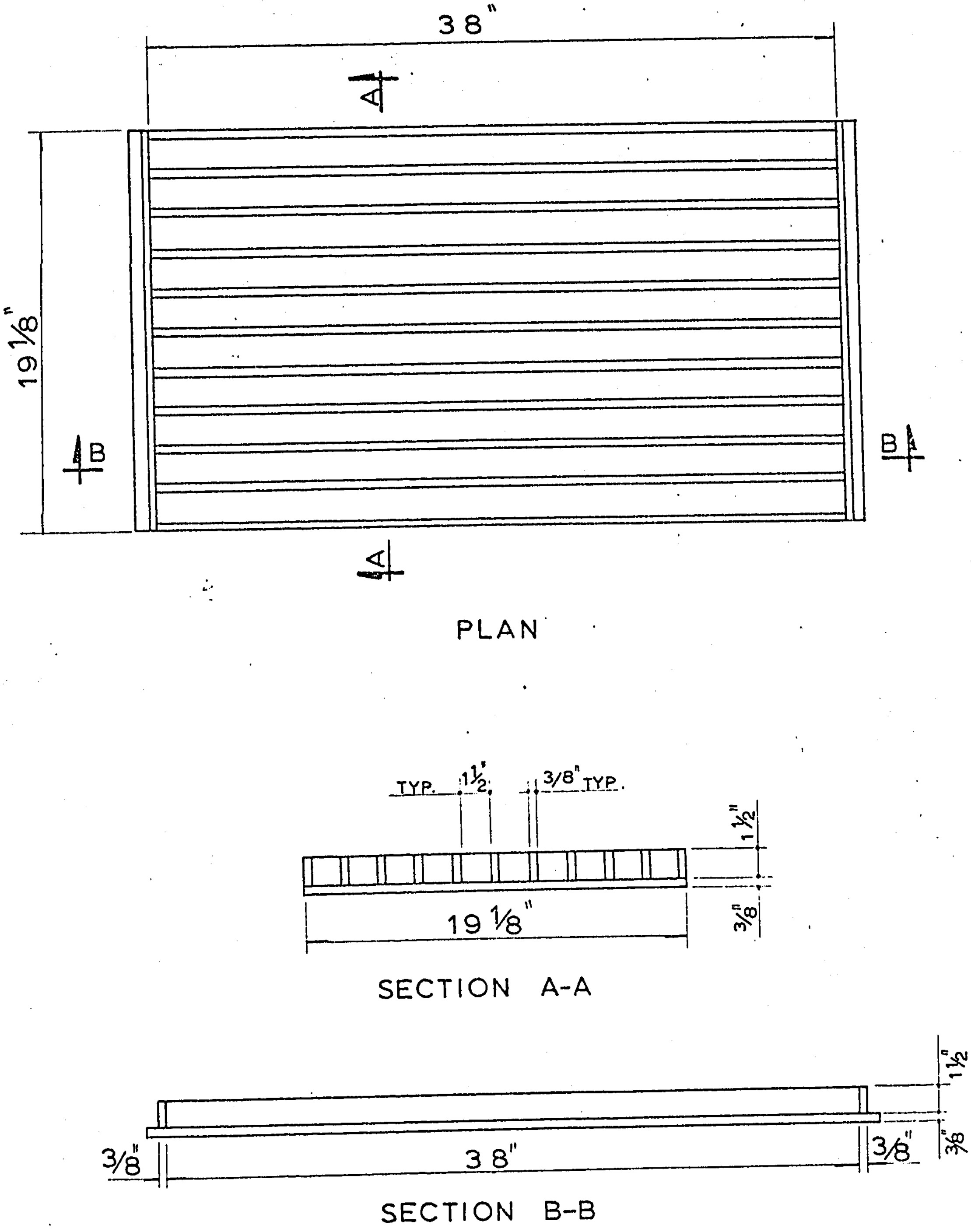


FIG. 4-13 STEEL BEAM FORM DETAIL

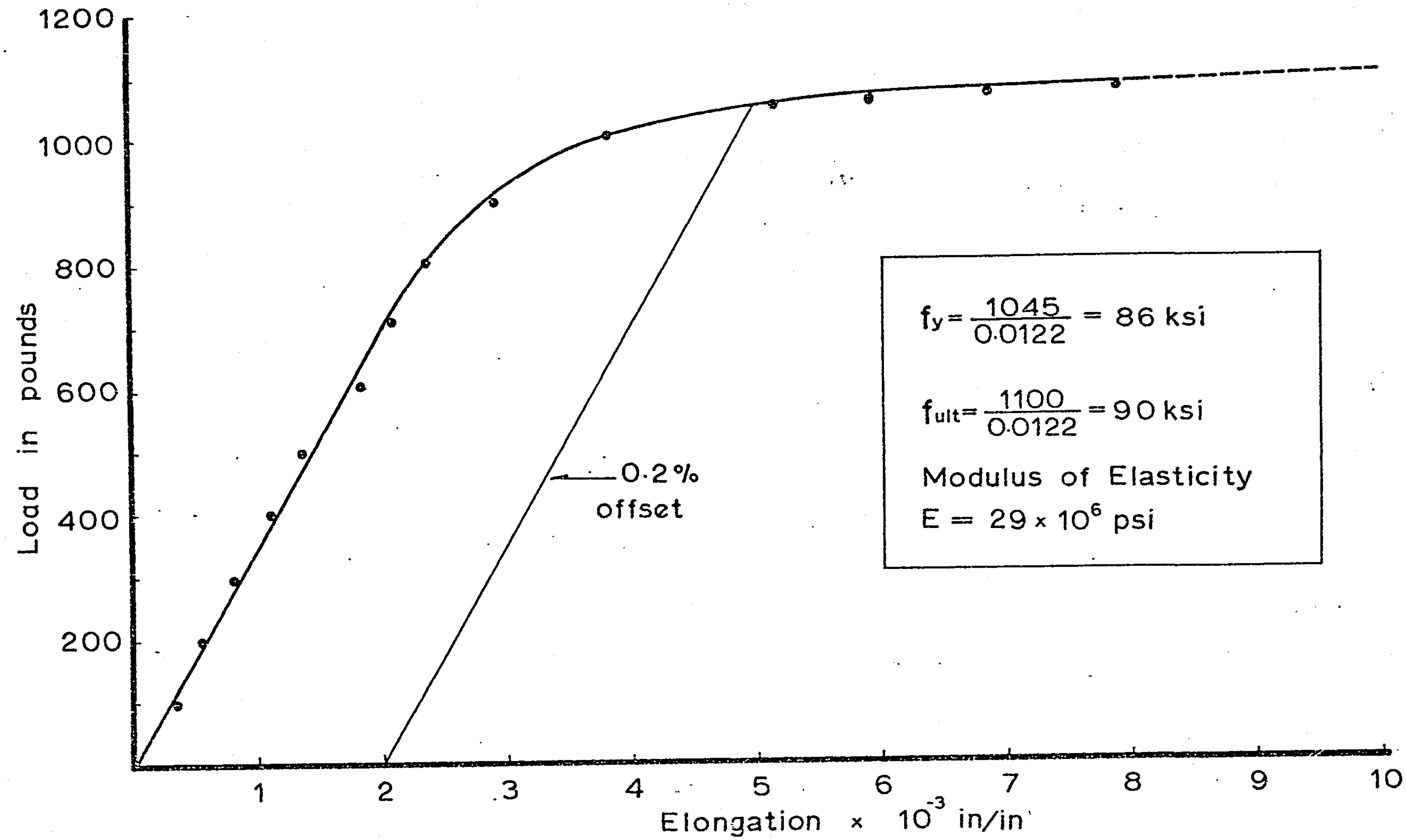


FIG. 4-14 TYIPICAL STRESS-STRAIN CURVE OF 1/8" DIAMETER COLD ROLLED MILD STEEL SAE1020

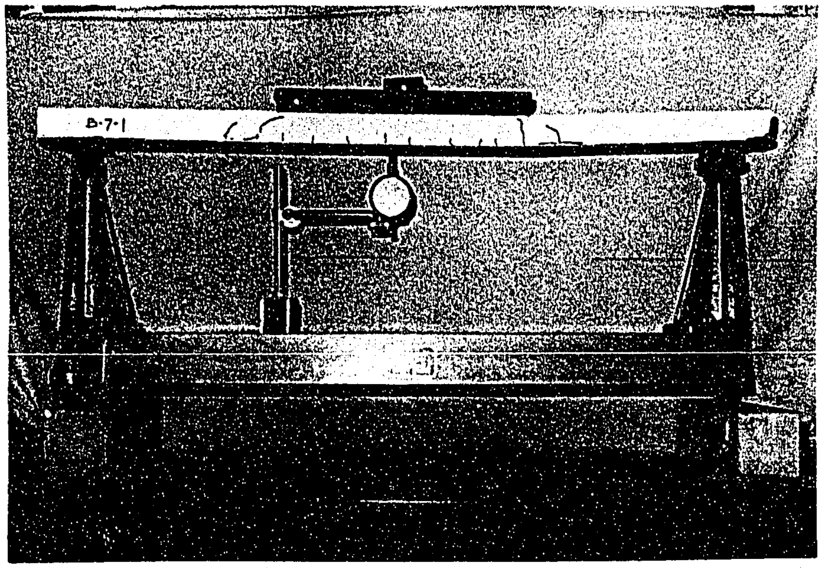


FIG. 4-15 GENERAL VIEW OF SIMPLY SUPPORTED BEAM

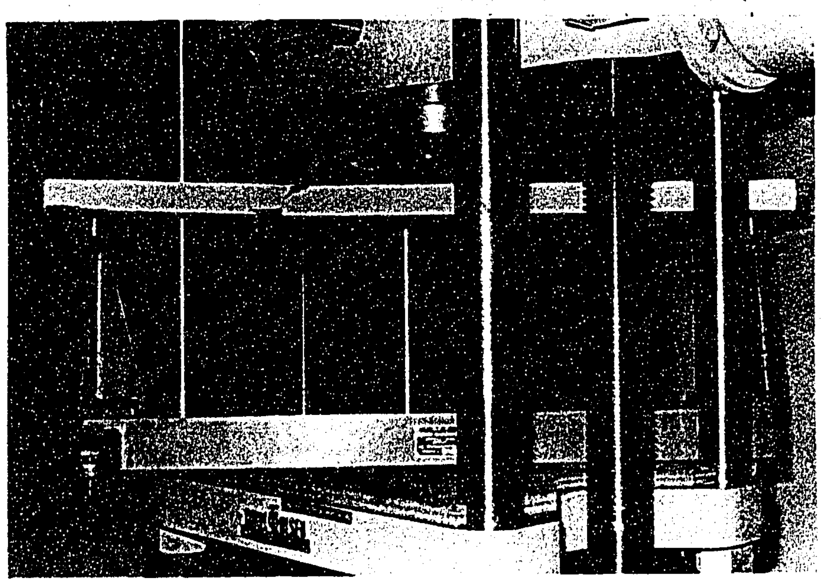


FIG. 4-16 TEST ARRANGEMENT FOR SIMPLY SUPPORTED BEAM

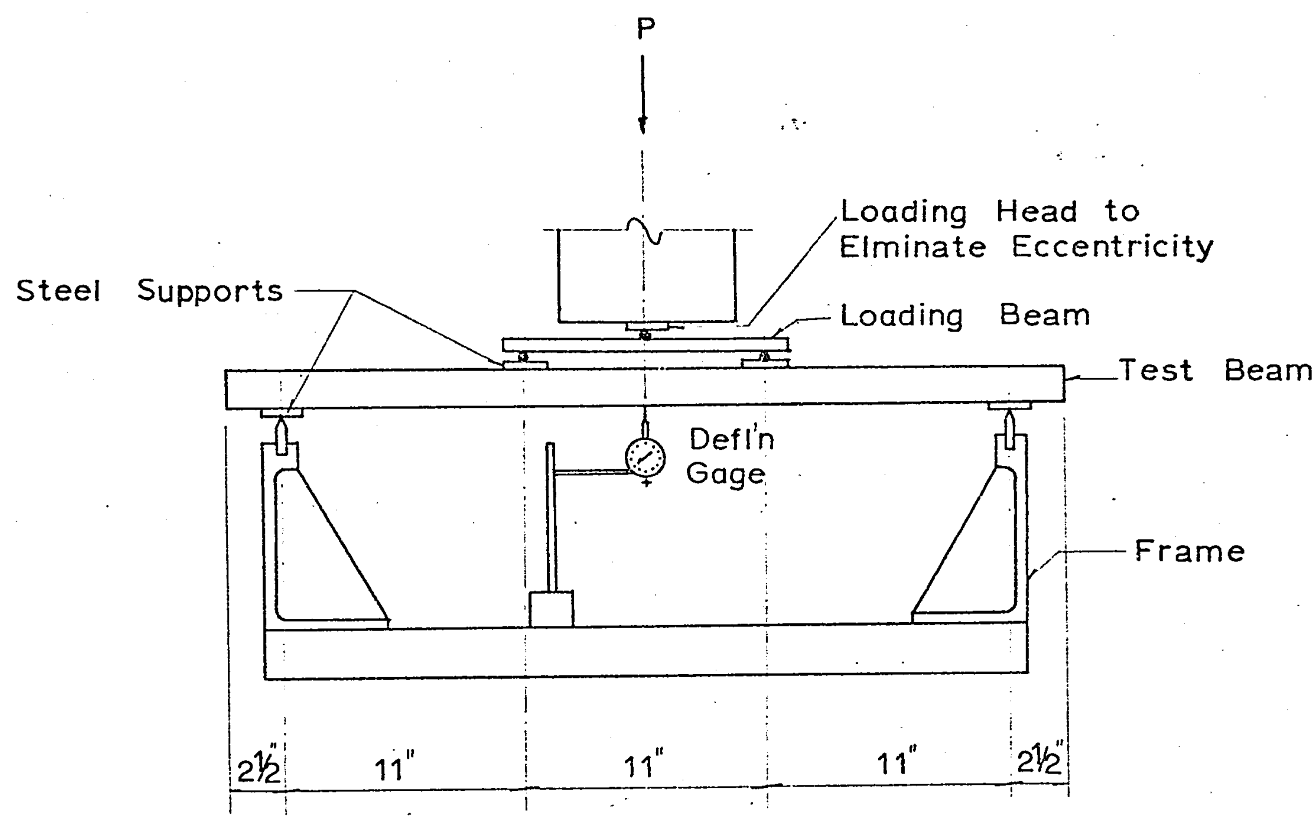


FIG. 4-17 MODEL TEST SETUP FOR SIMPLE BEAMS

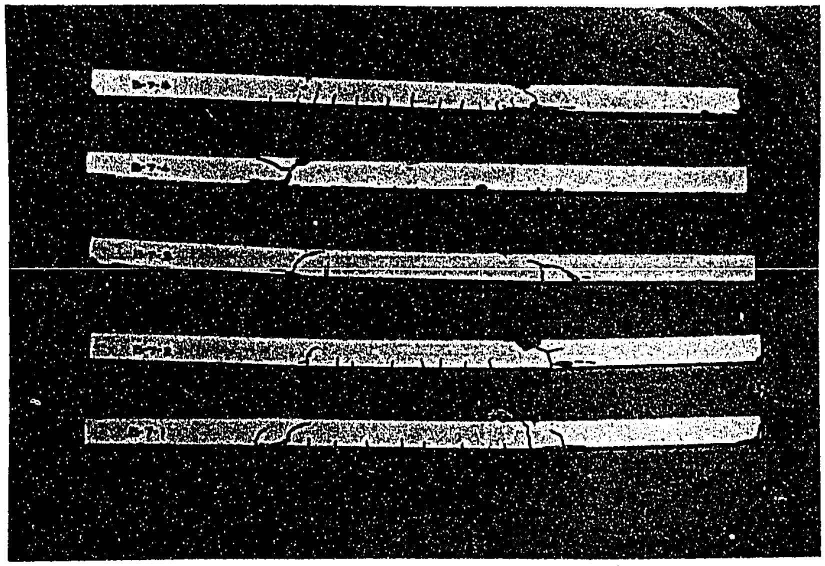


FIG. 4-18 FAILURE MODE OF SIMPLY SUPPORTED BEAMS (AT 7 DAYS)

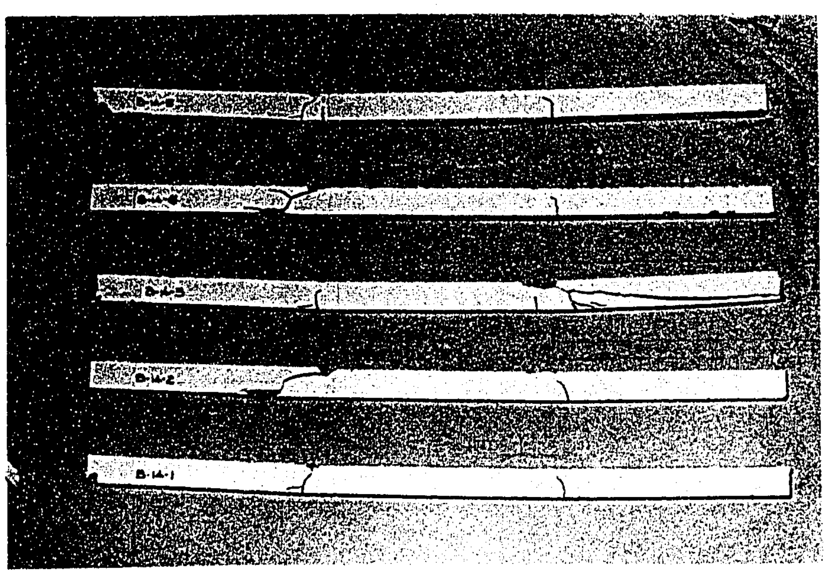


FIG. 4-19 FAILURE MODE OF SIMPLY SUPPORTED BEAMS (AT 14 DAYS)

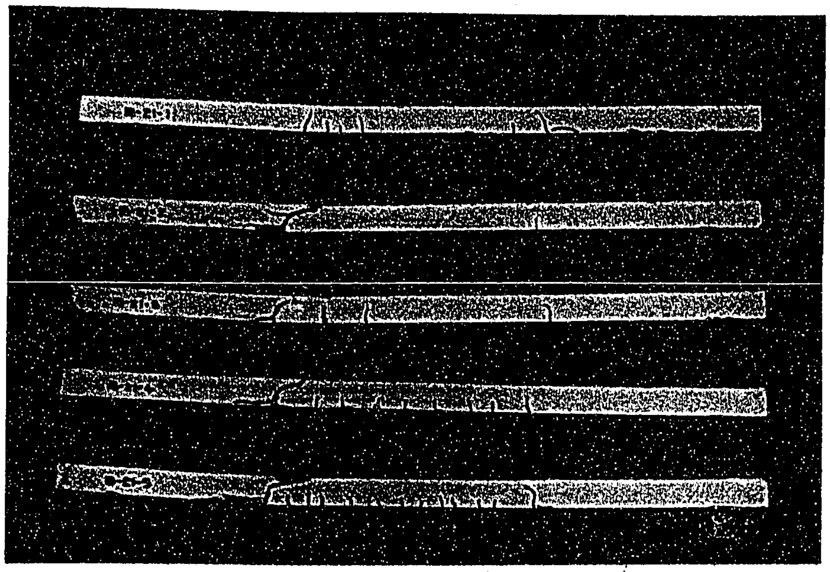


FIG. 4-20 FAILURE MODE OF SIMPLY SUPPORTED BEAMS (AT 21 DAYS)

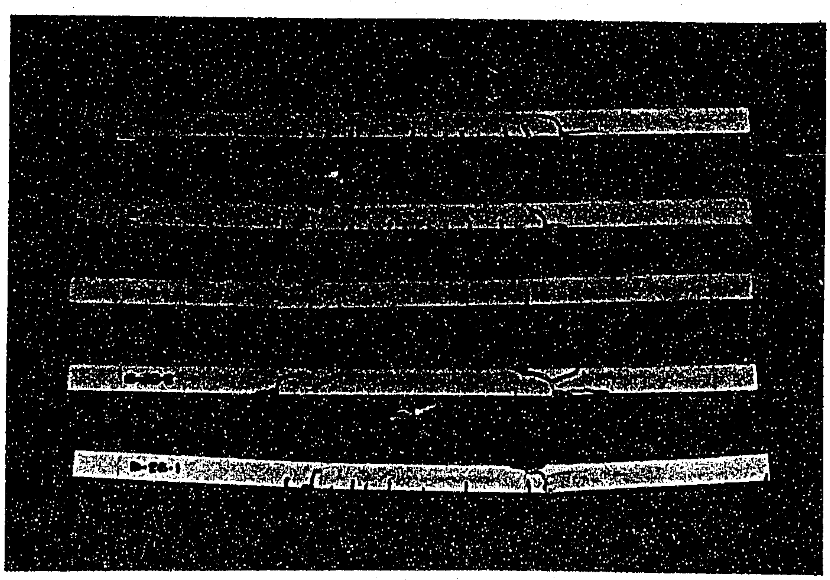


FIG. 4-21 FAILURE MODE OF SIMPLY SUPPORTED BEAMS (AT 28 DAYS)

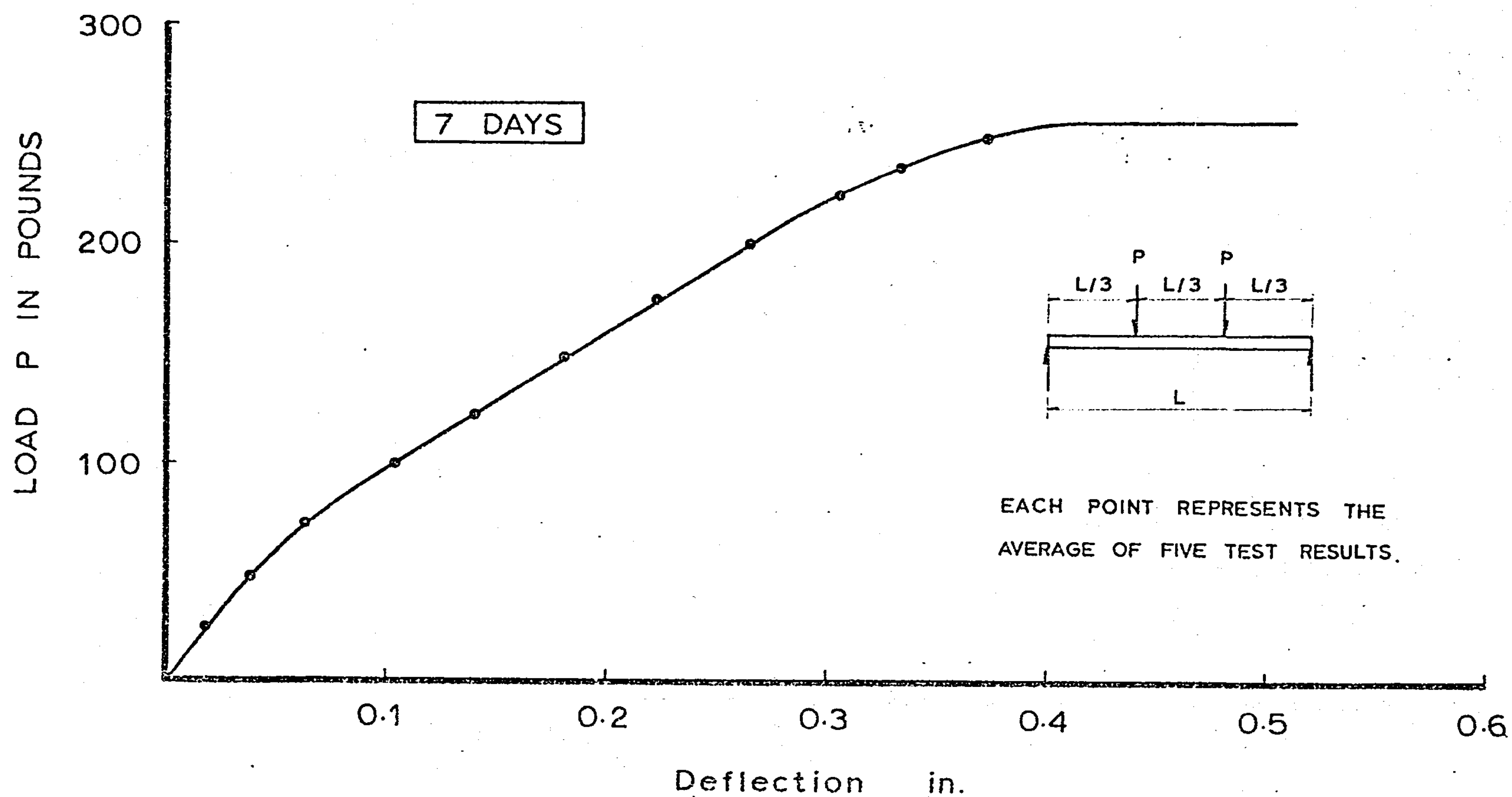


FIG. 4-22 SIMPLE BEAM CENTRAL POINT DEFLECTION
(7 DAYS)

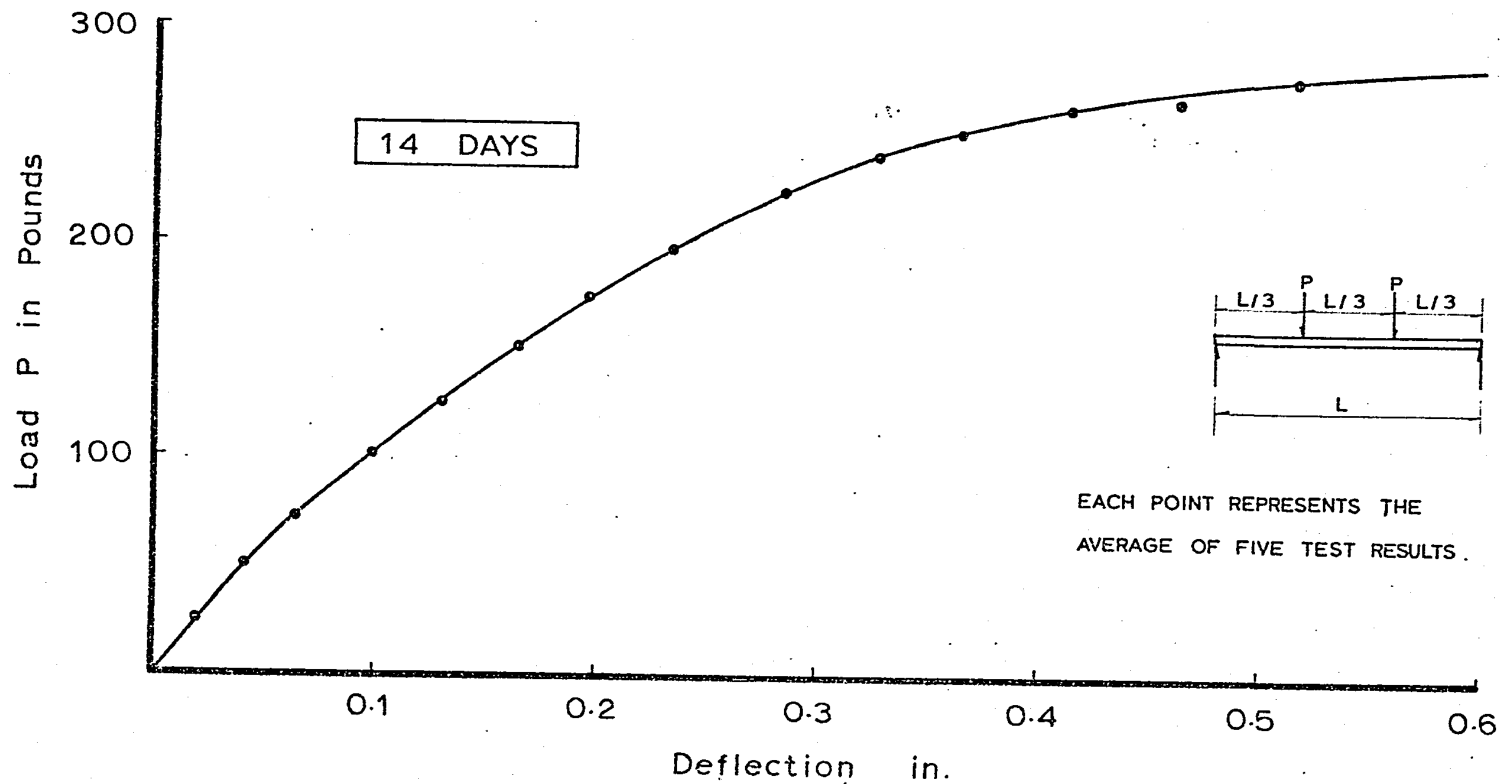


FIG. 4-23 SIMPLE BEAM CENTRAL POINT DEFLECTION
(14 DAYS)

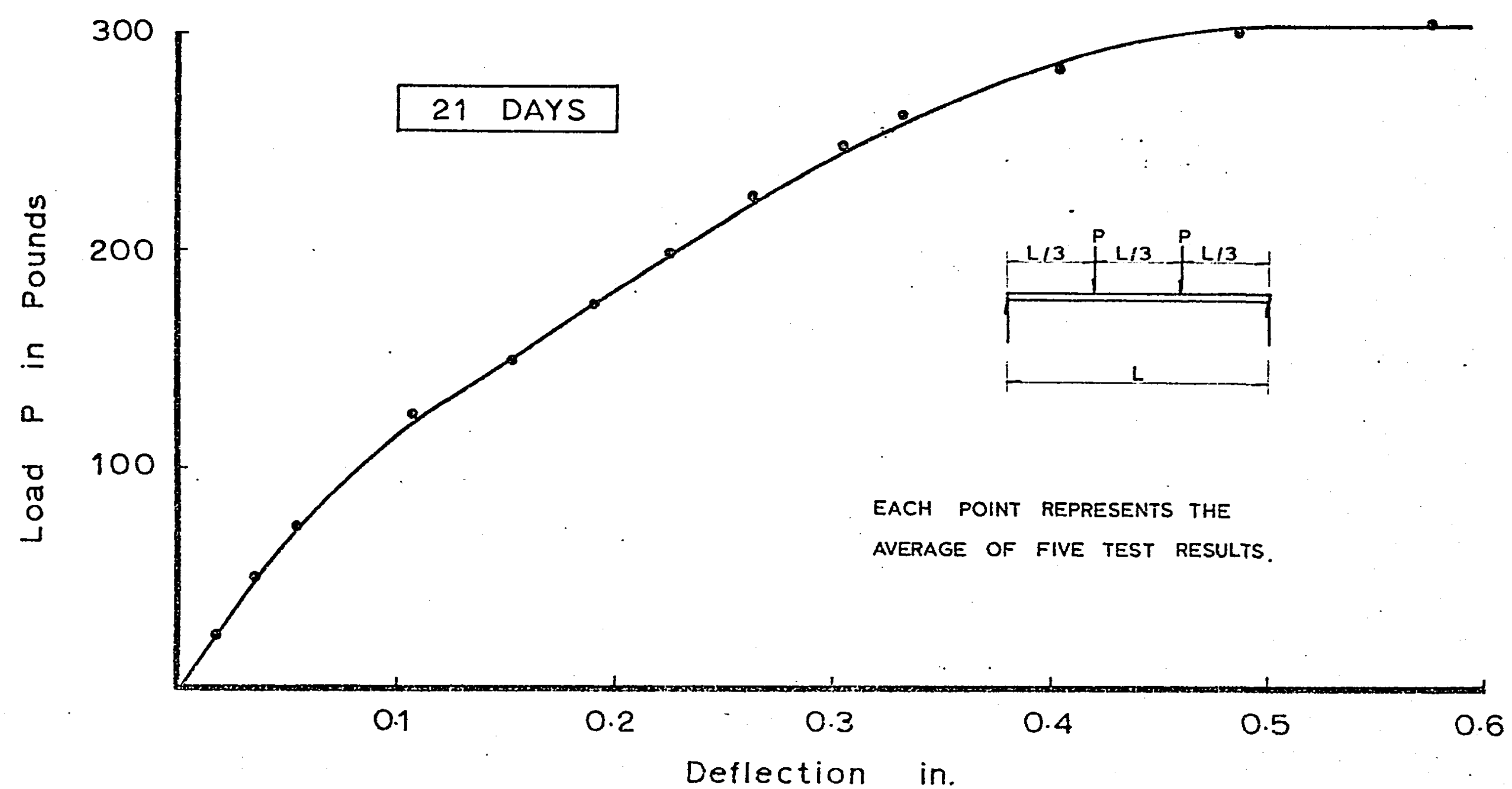


FIG. 4-24 SIMPLE BEAM CENTRAL POINT DEFLECTION
(21 DAYS)

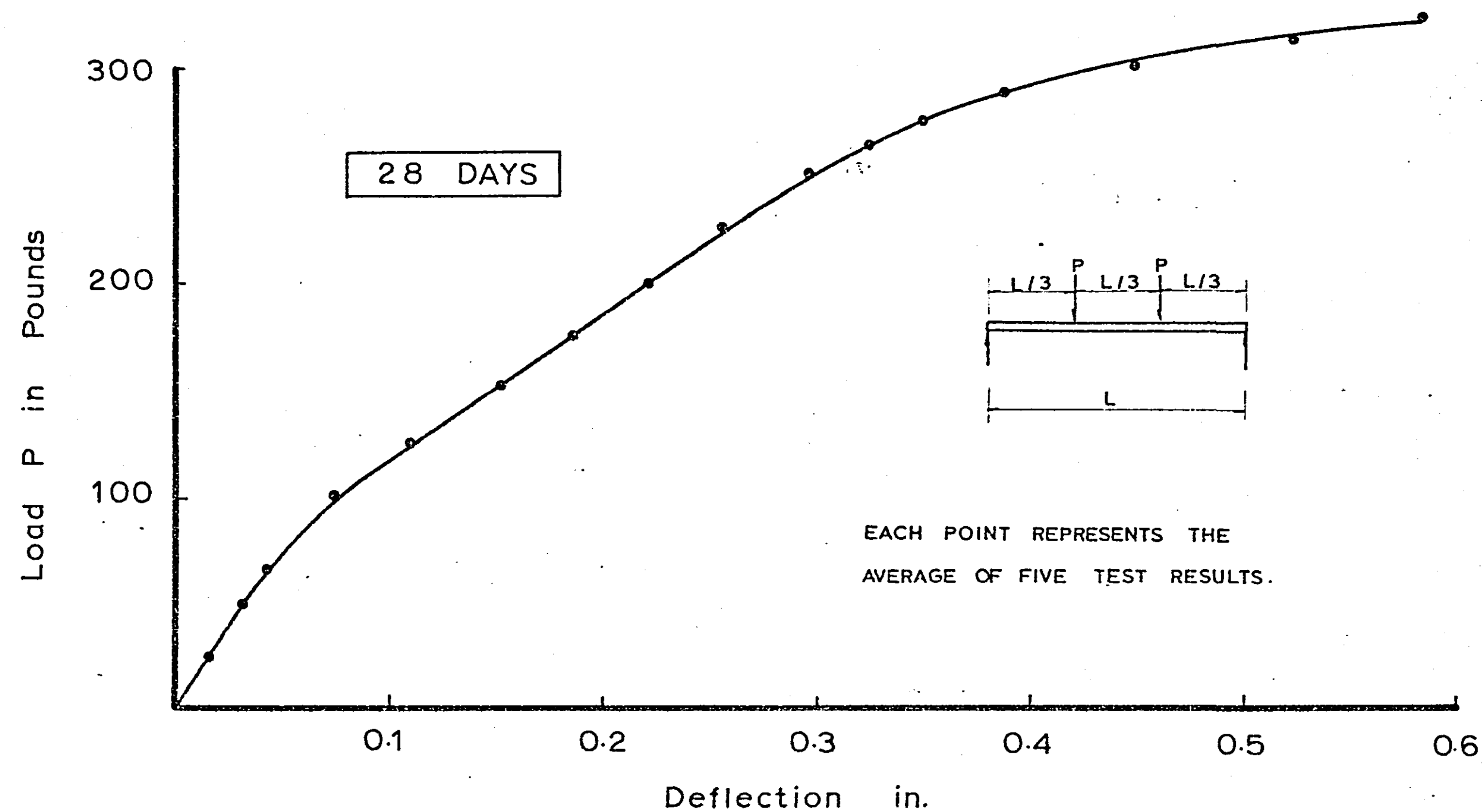


FIG. 4-25 SIMPLE BEAM CENTRAL POINT DEFLECTION
(28 DAYS)

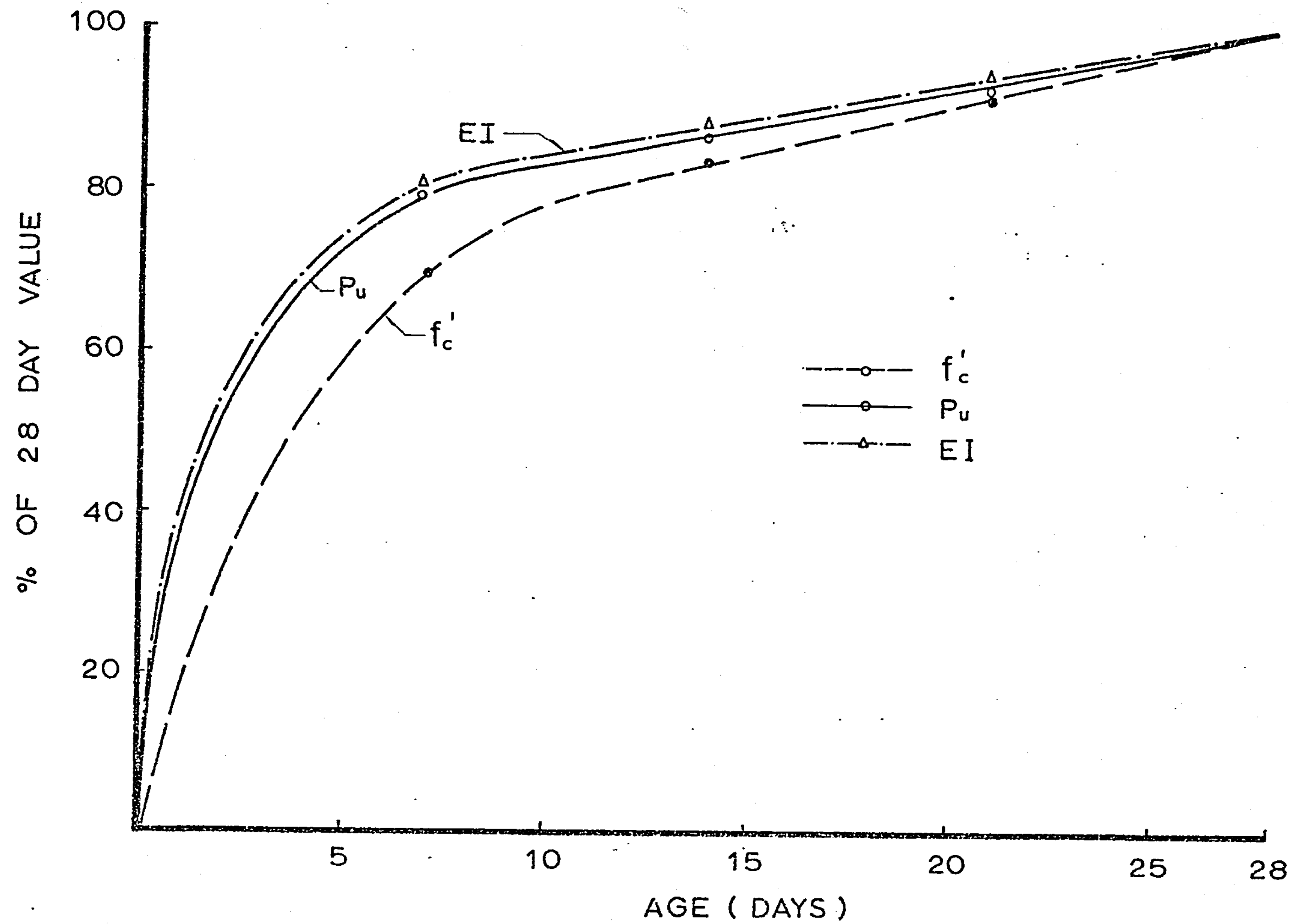


FIG. 4-26 TESTING RESULTS OF P_u , f_c' & STIFFNESS EI OF BEAMS IN TERMS OF THEIR 28-DAY VALUE

APPENDIX A

CONCRETE CYLINDER COMPRESSION TEST RESULTS

GROUP 3600 1DAYS

P	STRESS	L	R	MEAN	STRAIN
1000.00	35.40	.80	.60	.70	7.778
2000.00	70.80	1.00	2.00	1.50	16.667
3000.00	106.19	1.50	3.50	2.50	27.778
4000.00	141.59	2.00	5.00	3.50	38.889
5000.00	176.99	2.50	6.50	4.50	50.000
1000.00	35.40	.50	1.00	.75	8.333
2000.00	70.80	1.10	1.70	1.40	15.556
3000.00	106.19	1.60	2.60	2.10	23.333
4000.00	141.59	2.30	3.70	3.00	33.333
5000.00	176.99	3.20	4.50	3.85	42.778
6000.00	212.39	4.40	5.60	5.00	55.556
7000.00	247.79	5.50	6.50	6.00	66.667
8000.00	283.19	5.60	7.50	6.55	72.778
1000.00	35.40	1.00	.40	.70	7.778
2000.00	70.80	1.80	1.40	1.60	17.778
3000.00	106.19	3.00	2.00	2.50	27.778
4000.00	141.59	4.40	2.40	3.40	37.778
5000.00	176.99	5.60	3.00	4.30	47.778
6000.00	212.39	7.00	4.00	5.50	61.111
7000.00	247.79	8.00	4.80	6.40	71.111
8000.00	283.19	9.00	5.80	7.40	82.222
1000.00	35.40	.98	.50	.74	8.222
2000.00	70.80	2.00	1.10	1.55	17.222
3000.00	106.19	3.00	2.00	2.50	27.778
4000.00	141.59	4.00	3.00	3.50	38.889
5000.00	176.99	5.00	4.00	4.50	50.000
6000.00	212.39	6.00	5.00	5.50	61.111
7000.00	247.79	7.00	6.00	6.50	72.222
8000.00	283.19	8.00	7.00	7.50	83.333
2000.00	70.80	2.50	1.00	1.75	19.444
4000.00	141.59	5.00	2.10	3.55	39.444
5000.00	176.99	5.80	2.80	4.30	47.778
6000.00	212.39	6.50	3.50	5.00	55.556
7000.00	247.79	7.80	4.00	5.90	65.556
8000.00	283.19	9.20	4.50	6.85	76.111

FC = 2336.2831
MODULUS OF ELASTICITY = E = 3.8619

CONCRETE CYLINDER COMPRESSION TEST RESULTS

GROUP 3600 4DAYS

P	STRESS	L	R	MEAN	STRAIN
1000.00	35.40	.43	.90	.67	7.389
3000.00	106.19	1.10	2.90	2.00	22.222
4000.00	141.59	1.30	4.00	2.65	29.444
5000.00	176.99	1.60	5.20	3.40	37.778
7000.00	247.79	2.40	7.10	4.75	52.778
8000.00	283.19	3.10	8.12	5.61	62.333
10000.00	353.98	2.90	10.70	6.80	75.556
1000.00	35.40	.40	1.10	.75	8.333
2000.00	70.80	.50	2.60	1.55	17.222
4000.00	141.59	1.00	5.10	3.05	33.889
6000.00	212.39	1.80	7.50	4.65	51.667
8000.00	283.19	2.50	10.00	6.25	69.444
10000.00	353.98	3.20	13.90	8.55	95.000
1000.00	35.40	1.00	.20	.60	6.667
2000.00	70.80	1.60	.85	1.23	13.611
3000.00	106.19	2.40	1.60	2.00	22.222
4000.00	141.59	3.00	2.00	2.50	27.778
5000.00	176.99	3.90	2.50	3.20	35.556
6000.00	212.39	5.00	3.10	4.05	45.000
7000.00	247.79	6.00	3.90	4.95	55.000
8000.00	283.19	6.90	4.20	5.55	61.667
9000.00	318.58	7.90	5.00	6.45	71.667
10000.00	353.98	8.90	5.80	7.35	81.667
1000.00	35.40	.00	1.50	.75	8.333
2000.00	70.80	.40	2.60	1.50	16.667
3000.00	106.19	.50	4.00	2.25	25.000
4000.00	141.59	.80	5.20	3.00	33.333
5000.00	176.99	.80	6.80	3.80	42.222
6000.00	212.39	.80	8.20	4.50	50.000
7000.00	247.79	.90	9.80	5.35	59.444
8000.00	283.19	.90	11.50	6.20	68.889
1000.00	35.40	.60	.80	.70	7.778
2000.00	70.80	1.25	1.65	1.45	16.111
3000.00	106.19	1.90	2.50	2.20	24.444
4000.00	141.59	2.60	3.10	2.85	31.667
5000.00	176.99	3.75	3.75	3.75	41.667
6000.00	212.39	4.50	4.40	4.45	49.444
7000.00	247.79	5.30	5.10	5.20	57.778
8000.00	283.19	6.20	5.80	6.00	66.667

FC = 3610.6195
MODULUS OF ELASTICITY = E = 4.4603

CONCRETE CYLINDER COMPRESSION TEST RESULTS

GROUP 3600 7DAYS

P	STRESS	L	R	MEAN	STRAIN
5000.00	176.99	3.00	2.00	2.50	27.778
10000.00	353.98	5.80	6.10	5.95	66.111
20000.00	707.96	13.00	15.30	14.15	157.222
25000.00	884.96	16.80	20.00	18.40	204.444
30000.00	1061.95	20.00	24.00	22.00	244.444
35000.00	1238.94	24.00	28.00	26.00	288.889
40000.00	1415.93	28.00	32.70	30.35	337.222
50000.00	1769.91	36.50	42.00	39.25	436.111
60000.00	2123.89	47.00	51.00	49.00	544.444
5000.00	176.99	1.00	3.80	2.40	26.667
10000.00	353.98	1.10	9.90	5.50	61.111
15000.00	530.97	1.90	16.50	9.20	102.222
20000.00	707.96	2.50	22.80	12.65	140.556
30000.00	1061.95	4.00	33.00	18.50	205.556
35000.00	1238.94	5.00	40.00	22.50	250.000
40000.00	1415.93	9.00	45.20	27.10	301.111
50000.00	1769.91	14.50	54.00	34.25	380.556
60000.00	2123.89	22.30	63.00	42.65	473.889
5000.00	176.99	4.50	2.00	3.25	36.111
10000.00	353.98	8.20	6.00	7.10	78.889
15000.00	530.97	12.50	9.00	10.75	119.444
20000.00	707.96	15.50	11.50	13.50	150.000
25000.00	884.96	20.00	15.00	17.50	194.444
35000.00	1238.94	28.00	22.00	25.00	277.778
40000.00	1415.93	31.50	25.00	28.25	313.889
50000.00	1769.91	40.00	32.00	36.00	400.000
60000.00	2123.89	50.00	40.00	45.00	500.000
5000.00	176.99	2.70	2.50	2.60	28.889
10000.00	353.98	6.70	6.50	6.60	73.333
20000.00	707.96	12.00	14.00	13.00	144.444
25000.00	884.96	21.30	17.20	19.25	213.889
30000.00	1061.95	26.00	20.00	23.00	255.556
35000.00	1238.94	31.50	24.00	27.75	308.333
40000.00	1415.93	36.50	27.50	32.00	355.556
50000.00	1769.91	46.00	34.50	40.25	447.222
60000.00	2123.89	58.00	43.00	50.50	561.111
20000.00	707.96	12.20	19.00	15.60	173.333
25000.00	884.96	14.80	22.50	18.65	207.222
30000.00	1061.95	18.50	27.00	22.75	252.778
35000.00	1238.94	22.50	31.50	27.00	300.000
40000.00	1415.93	26.50	36.00	31.25	347.222
45000.00	1592.92	31.00	40.50	35.75	397.222
50000.00	1769.91	35.00	46.00	40.50	450.000
60000.00	2123.89	43.80	54.50	49.15	546.111
70000.00	2477.88	53.00	64.00	58.50	650.000
80000.00	2831.86	63.50	74.00	68.75	763.889

FC = 4329.2035

MODULUS OF ELASTICITY = E = 4.5374

CONCRETE CYLINDER COMPRESSION TEST RESULTS

GROUP 3600 14DAYS

P	STRESS	L	R	MEAN	STRAIN
10000.00	353.98	5.50	6.50	6.00	66.667
15000.00	530.97	8.00	11.00	9.50	105.556
20000.00	707.96	11.00	15.20	13.10	145.556
30000.00	1061.95	16.00	22.00	19.00	211.111
40000.00	1415.93	21.50	29.50	25.50	283.333
50000.00	1769.91	28.50	38.00	33.25	369.444
60000.00	2123.89	34.70	45.80	40.25	447.222
70000.00	2477.88	42.00	54.50	48.25	536.111
80000.00	2831.86	49.50	63.00	56.25	625.000
5000.00	176.99	1.00	3.00	2.00	22.222
10000.00	353.98	2.00	6.50	4.25	47.222
15000.00	530.97	6.00	11.00	8.50	94.444
20000.00	707.96	9.00	15.00	12.00	133.333
30000.00	1061.95	14.00	21.50	17.75	197.222
40000.00	1415.93	18.50	29.00	23.75	263.889
50000.00	1769.91	25.00	37.00	31.00	344.444
60000.00	2123.89	30.00	44.00	37.00	411.111
70000.00	2477.88	37.00	53.50	45.25	502.778
80000.00	2831.86	45.00	62.00	53.50	594.444
5000.00	176.99	2.50	4.00	3.25	36.111
10000.00	353.98	4.00	9.00	6.50	72.222
15000.00	530.97	6.00	14.00	10.00	111.111
20000.00	707.96	9.00	18.00	13.50	150.000
30000.00	1061.95	13.00	27.50	20.25	225.000
40000.00	1415.93	18.00	35.80	26.90	298.889
50000.00	1769.91	24.00	45.00	34.50	383.333
60000.00	2123.89	30.20	54,50	42.35	470.556
80000.00	2831.86	40.20	72.00	56.10	623.333
5000.00	176.99	4.00	4.00	4.00	44.444
10000.00	353.98	7.50	7.00	7.25	80.556
15000.00	530.97	11.00	11.00	11.00	122.222
20000.00	707.96	16.00	15.00	15.50	172.222
30000.00	1061.95	23.00	23.00	23.00	255.556
40000.00	1415.93	31.50	31.50	31.50	350.000
60000.00	2123.89	48.00	45.00	46.50	516.667
70000.00	2477.88	57.00	56.00	56.50	627.778
80000.00	2831.86	68.00	65.00	66.50	738.889
5000.00	176.99	2.50	5.00	3.75	41.667
10000.00	353.98	4.50	10.50	7.50	83.333
15000.00	530.97	7.00	15.00	11.00	122.222
20000.00	707.96	9.00	20.00	14.50	161.111
30000.00	1061.95	13.50	29.50	21.50	238.889
40000.00	1415.93	19.50	39.50	29.50	327.778
50000.00	1769.91	26.00	49.50	37.75	419.444
60000.00	2123.89	32.50	58.00	45.25	502.778
70000.00	2477.88	40.00	68.00	54.00	600.000
80000.00	2831.86	47.50	80.00	63.75	708.333

FC = 4750.4425
MODULUS OF ELASTICITY = E = 4.7445

CONCRETE CYLINDER COMPRESSION TEST RESULTS

GROUP 3600 21DAYS

P	STRESS	L	R	MEAN	STRAIN
5000.00	176.99	2.50	3.10	2.80	31.111
10000.00	353.98	5.60	5.90	5.75	63.889
15000.00	530.97	8.00	9.50	8.75	97.222
20000.00	707.96	11.50	13.00	12.25	136.111
30000.00	1061.95	17.00	20.50	18.75	208.333
40000.00	1415.93	24.50	29.00	26.75	297.222
50000.00	1769.91	30.00	37.50	33.75	375.000
60000.00	2123.89	35.50	46.00	40.75	452.778
70000.00	2477.88	42.00	54.20	48.10	534.444
80000.00	2831.86	48.50	63.00	55.75	619.444
5000.00	176.99	3.90	2.00	2.95	32.778
10000.00	353.98	8.50	3.50	6.00	66.667
15000.00	530.97	12.50	6.00	9.25	102.778
20000.00	707.96	16.00	9.00	12.50	138.889
30000.00	1061.95	23.00	16.00	19.50	216.667
40000.00	1415.93	29.00	22.50	25.75	286.111
50000.00	1769.91	36.00	31.00	33.50	372.222
60000.00	2123.89	42.00	39.00	40.50	450.000
70000.00	2477.88	49.00	48.00	48.50	538.889
80000.00	2831.86	56.00	57.50	56.75	630.556
5000.00	176.99	2.00	4.00	3.00	33.333
10000.00	353.98	3.50	8.00	5.75	63.889
15000.00	530.97	6.00	13.00	9.50	105.556
20000.00	707.96	9.50	16.50	13.00	144.444
30000.00	1061.95	16.00	23.00	19.50	216.667
40000.00	1415.93	23.50	29.50	26.50	294.444
50000.00	1769.91	30.00	36.50	33.25	369.444
60000.00	2123.89	38.50	42.00	40.25	447.222
70000.00	2477.88	47.00	50.00	48.50	538.889
80000.00	2831.86	58.00	56.00	57.00	633.333
5000.00	176.99	4.00	2.00	3.00	33.333
10000.00	353.98	7.00	5.50	6.25	69.444
15000.00	530.97	10.00	9.00	9.50	105.556
20000.00	707.96	13.00	13.00	13.00	144.444
30000.00	1061.95	18.00	20.80	19.40	215.556
40000.00	1415.93	24.00	29.00	26.50	294.444
50000.00	1769.91	30.00	37.00	33.50	372.222
60000.00	2123.89	37.00	46.00	41.50	461.111
70000.00	2477.88	44.00	54.00	49.00	544.444
80000.00	2831.86	52.00	63.00	57.50	638.889
5000.00	176.99	3.60	1.00	2.30	25.556
10000.00	353.98	10.80	4.00	7.40	82.222
15000.00	530.97	14.40	8.00	11.20	124.444
20000.00	707.96	18.00	11.00	14.50	161.111
30000.00	1061.95	24.50	17.00	20.75	230.556
40000.00	1415.93	32.50	24.00	28.25	313.889
50000.00	1769.91	40.00	30.00	35.00	388.889
60000.00	2123.89	50.00	35.00	42.50	472.222
70000.00	2477.88	58.00	42.00	50.00	555.556
80000.00	2831.86	70.00	48.00	59.00	655.556

FC = 5168.1416

MODULUS OF ELASTICITY = E = 4.8823

CONCRETE CYLINDER COMPRESSION TEST RESULTS

GROUP 3600 28DAYS

P	STRESS	L	R	MEAN	STRAIN
5000.00	176.99	3.20	2.20	2.70	30.000
10000.00	353.98	6.00	6.00	6.00	66.667
15000.00	530.97	8.80	10.00	9.40	104.444
20000.00	707.96	11.00	14.00	12.50	138.889
30000.00	1061.95	16.00	21.00	18.50	205.556
40000.00	1415.93	22.00	29.00	25.50	283.333
50000.00	1769.91	28.00	38.00	33.00	366.667
60000.00	2123.89	34.00	46.00	40.00	444.444
70000.00	2477.88	41.00	54.00	47.50	527.778
80000.00	2831.86	48.00	62.00	55.00	611.111
5000.00	176.99	2.00	3.70	2.85	31.667
10000.00	353.98	4.50	7.60	6.05	67.222
15000.00	530.97	7.00	12.20	9.60	106.667
20000.00	707.96	9.00	17.50	13.25	147.222
30000.00	1061.95	13.50	27.00	20.25	225.000
40000.00	1415.93	18.00	36.50	27.25	302.778
50000.00	1769.91	24.00	47.00	35.50	394.444
60000.00	2123.89	29.00	56.00	42.50	472.222
70000.00	2477.88	35.00	66.00	50.50	561.111
80000.00	2831.86	41.00	77.00	59.00	655.556
5000.00	176.99	1.80	3.60	2.70	30.000
10000.00	353.98	5.00	6.40	5.70	63.333
15000.00	530.97	6.20	12.00	9.10	101.111
20000.00	707.96	7.10	16.50	11.80	131.111
30000.00	1061.95	11.70	26.00	18.85	209.444
40000.00	1415.93	16.20	33.20	24.70	274.444
50000.00	1769.91	21.60	41.40	31.50	350.000
60000.00	2123.89	28.00	49.00	38.50	427.778
70000.00	2477.88	34.40	57.00	45.70	507.778
80000.00	2831.86	40.00	66.00	53.00	588.889
5000.00	176.99	2.50	2.20	2.35	26.111
10000.00	353.98	5.00	6.40	5.70	63.333
15000.00	530.97	8.50	11.50	10.00	111.111
20000.00	707.96	11.20	15.50	13.35	148.333
30000.00	1061.95	17.00	23.80	20.40	226.667
40000.00	1415.93	23.00	32.00	27.50	305.556
50000.00	1769.91	30.00	41.00	35.50	394.444
60000.00	2123.89	38.00	49.00	43.50	483.333
70000.00	2477.88	44.00	57.00	50.50	561.111
80000.00	2831.86	49.00	64.00	56.50	627.778
5000.00	176.99	2.00	3.20	2.60	28.889
10000.00	353.98	4.00	8.10	6.05	67.222
15000.00	530.97	6.00	14.00	10.00	111.111
20000.00	707.96	8.00	19.00	13.50	150.000
30000.00	1061.95	13.00	27.00	20.00	222.222
40000.00	1415.93	19.00	35.50	27.25	302.778
50000.00	1769.91	26.50	44.00	35.25	391.667
60000.00	2123.89	32.50	51.50	42.00	466.667
70000.00	2477.88	40.50	60.50	50.50	561.111
80000.00	2831.86	48.50	69.00	58.75	652.778

FC = 5497.3451

MODULUS OF ELASTICITY = E = 4.9509

CONCRETE CYLINDER COMPRESSION TEST RESULTS

GROUP 6000 2DAYS

P	STRESS	L	R	MEAN	STRAIN
5000.00	176.99	2.80	2.40	2.60	28.889
10000.00	353.98	6.70	6.50	6.60	73.333
20000.00	707.96	12.00	14.00	13.00	144.444
25000.00	884.96	21.60	17.50	19.55	217.222
30000.00	1061.95	26.00	20.00	23.00	255.556
35000.00	1238.94	31.20	24.30	27.75	308.333
40000.00	1415.93	36.50	27.00	31.75	352.778
50000.00	1769.91	46.00	34.00	40.00	444.444
60000.00	2123.89	58.50	43.00	50.75	563.889
70000.00	2477.88	51.00	64.00	57.50	638.889
5000.00	176.99	1.20	3.70	2.45	27.222
10000.00	353.98	1.20	9.90	5.55	61.667
15000.00	530.97	1.90	16.00	8.95	99.444
20000.00	707.96	2.50	22.50	12.50	138.889
30000.00	1061.95	4.00	33.00	18.50	205.556
35000.00	1238.94	5.00	40.00	22.50	250.000
40000.00	1415.93	9.30	45.20	27.25	302.778
50000.00	1769.91	14.50	54.00	34.25	380.556
60000.00	2123.89	22.60	63.00	42.80	475.556
70000.00	2477.88	53.00	67.00	60.00	666.667
5000.00	176.99	3.00	2.00	2.50	27.778
10000.00	353.98	5.80	6.10	5.95	66.111
20000.00	707.96	13.60	15.30	14.45	160.556
25000.00	884.96	16.80	20.00	18.40	204.444
30000.00	1061.95	20.20	24.00	22.10	245.556
35000.00	1238.94	24.00	28.00	26.00	288.889
40000.00	1415.93	28.00	32.70	30.35	337.222
50000.00	1769.91	36.00	42.00	39.00	433.333
60000.00	2123.89	47.00	51.00	49.00	544.444
70000.00	2477.88	53.50	64.00	58.75	652.778
5000.00	176.99	4.50	2.00	3.25	36.111
10000.00	353.98	8.00	6.00	7.00	77.778
15000.00	530.97	12.50	9.00	10.75	119.444
20000.00	707.96	15.50	11.50	13.50	150.000
25000.00	884.96	20.00	15.00	17.50	194.444
35000.00	1238.94	29.00	22.10	25.55	283.889
40000.00	1415.93	31.50	25.00	28.25	313.889
50000.00	1769.91	40.00	32.00	36.00	400.000
60000.00	2123.89	50.00	40.00	45.00	500.000
70000.00	2477.88	52.00	66.00	59.00	655.556
20000.00	707.96	12.20	19.00	15.60	173.333
25000.00	884.96	14.80	22.50	18.65	207.222
30000.00	1061.95	18.00	27.50	22.75	252.778
35000.00	1238.94	22.50	31.50	27.00	300.000
40000.00	1415.93	26.50	36.00	31.25	347.222
45000.00	1592.92	30.00	41.50	35.75	397.222
50000.00	1769.91	35.00	46.00	40.50	450.000
60000.00	2123.89	43.00	54.50	48.75	541.667
70000.00	2477.88	53.00	64.00	58.50	650.000
80000.00	2831.86	64.50	74.00	69.25	769.444

FC = 4481.4159

MODULUS OF ELASTICITY = E = 4.4746

CONCRETE CYLINDER COMPRESSION TEST RESULTS

GROUP 6000 4DAYS

P	STRESS	L	R	MEAN	STRAIN
5000.00	176.99	2.60	2.50	2.55	28.333
10000.00	353.98	5.20	6.70	5.95	66.111
15000.00	530.97	8.00	11.00	9.50	105.556
20000.00	707.96	11.40	15.00	13.20	146.667
25000.00	884.96	14.60	20.70	17.65	196.111
30000.00	1061.95	17.60	24.80	21.20	235.556
35000.00	1238.94	21.20	29.80	25.50	283.333
40000.00	1415.93	24.30	33.90	29.10	323.333
45000.00	1592.92	28.00	39.20	33.60	373.333
50000.00	1769.91	32.20	44.50	38.35	426.111
10000.00	353.98	2.00	11.00	6.50	72.222
15000.00	530.97	3.30	16.00	9.65	107.222
20000.00	707.96	5.50	21.50	13.50	150.000
25000.00	884.96	7.30	26.50	16.90	187.778
35000.00	1238.94	10.50	36.00	23.25	258.333
40000.00	1415.93	12.50	41.50	27.00	300.000
45000.00	1592.92	14.60	47.10	30.85	342.778
50000.00	1769.91	16.50	52.20	34.35	381.667
5000.00	176.99	2.00	3.50	2.75	30.556
10000.00	353.98	5.00	8.00	6.50	72.222
15000.00	530.97	7.80	12.00	9.90	110.000
20000.00	707.96	11.00	17.00	14.00	155.556
25000.00	884.96	14.00	21.00	17.50	194.444
30000.00	1061.95	17.00	25.00	21.00	233.333
35000.00	1238.94	19.00	29.00	24.00	266.667
40000.00	1415.93	23.50	34.00	28.75	319.444
45000.00	1592.92	27.00	37.50	32.25	358.333
50000.00	1769.91	30.50	42.50	36.50	405.556
10000.00	353.98	6.50	6.20	6.35	70.556
15000.00	530.97	9.20	10.00	9.60	106.667
20000.00	707.96	12.00	14.00	13.00	144.444
25000.00	884.96	14.00	18.00	16.00	177.778
30000.00	1061.95	16.80	21.00	18.90	210.000
35000.00	1238.94	19.00	25.00	22.00	244.444
40000.00	1415.93	21.00	30.00	25.50	283.333
45000.00	1592.92	24.00	33.20	28.60	317.778
50000.00	1769.91	27.00	38.00	32.50	361.111
5000.00	176.99	3.50	2.50	3.00	33.333
15000.00	530.97	11.20	6.50	8.85	98.333
20000.00	707.96	16.10	9.00	12.55	139.444
25000.00	884.96	19.20	11.00	15.10	167.778
30000.00	1061.95	23.20	13.50	18.35	203.889
35000.00	1238.94	27.30	15.50	21.40	237.778
40000.00	1415.93	31.70	18.00	24.85	276.111
50000.00	1769.91	40.00	23.20	31.60	351.111
60000.00	2123.89	48.20	28.40	38.30	425.556

FC = 5253.0973
MODULUS OF ELASTICITY = E = 4.8772

CONCRETE CYLINDER COMPRESSION TEST RESULTS

GROUP 6000 7DAYS

P	STRESS	L	R	MEAN	STRAIN
5000.00	176.99	2.00	3.20	2.60	28.889
10000.00	353.98	4.00	7.00	5.50	61.111
15000.00	530.97	6.00	11.20	8.60	95.556
20000.00	707.96	9.00	14.30	11.65	129.444
30000.00	1061.95	13.50	26.00	19.75	219.444
40000.00	1415.93	18.50	34.20	26.35	292.778
50000.00	1769.91	24.00	44.00	34.00	377.778
60000.00	2123.89	29.00	53.20	41.10	456.667
70000.00	2477.88	35.00	62.10	48.55	539.444
80000.00	2831.86	41.00	70.20	55.60	617.778
10000.00	353.98	2.50	7.00	4.75	52.778
15000.00	530.97	4.50	12.50	8.50	94.444
20000.00	707.96	6.50	16.50	11.50	127.778
30000.00	1061.95	10.50	25.50	18.00	200.000
40000.00	1415.93	14.50	35.00	24.75	275.000
50000.00	1769.91	19.00	44.50	31.75	352.778
60000.00	2123.89	24.00	53.50	38.75	430.556
70000.00	2477.88	27.50	62.50	45.00	500.000
80000.00	2831.86	35.00	74.00	54.50	605.556
10000.00	353.98	3.00	8.50	5.75	63.889
15000.00	530.97	5.50	12.00	8.75	97.222
20000.00	707.96	9.50	14.50	12.00	133.333
30000.00	1061.95	16.00	20.80	18.40	204.444
40000.00	1415.93	24.00	27.00	25.50	283.333
50000.00	1769.91	34.00	31.00	32.50	361.111
60000.00	2123.89	41.50	39.00	40.25	447.222
70000.00	2477.88	50.50	43.50	47.00	522.222
80000.00	2831.86	61.50	49.50	55.50	616.667
5000.00	176.99	2.50	3.90	3.20	35.556
10000.00	353.98	3.00	10.00	6.50	72.222
15000.00	530.97	4.00	14.50	9.25	102.778
20000.00	707.96	6.20	21.00	13.60	151.111
30000.00	1061.95	11.50	29.50	20.50	227.778
40000.00	1415.93	16.00	38.80	27.40	304.444
50000.00	1769.91	21.50	48.00	34.75	386.111
60000.00	2123.89	26.80	57.00	41.90	465.556
70000.00	2477.88	32.80	68.00	50.40	560.000
80000.00	2831.86	38.50	78.00	58.25	647.222
5000.00	176.99	3.00	2.00	2.50	27.778
10000.00	353.98	6.50	5.50	6.00	66.667
15000.00	530.97	10.00	8.50	9.25	102.778
20000.00	707.96	13.00	12.00	12.50	138.889
30000.00	1061.95	20.50	19.00	19.75	219.444
40000.00	1415.93	29.00	27.50	28.25	313.889
50000.00	1769.91	37.50	35.50	36.50	405.556
60000.00	2123.89	45.00	42.50	43.75	486.111
70000.00	2477.88	54.00	50.50	52.25	580.556
80000.00	2831.86	62.40	58.50	60.45	671.667

FC = 6407.0796

MODULUS OF ELASTICITY = E = 5.0008

CONCRETE CYLINDER COMPRESSION TEST RESULTS

GROUP 6000 14DAYS

P	STRESS	L	R	MEAN	STRAIN
5000.00	176.99	4.10	3.60	3.85	42.778
10000.00	353.98	7.20	6.00	6.60	73.333
15000.00	530.97	11.00	9.00	10.00	111.111
20000.00	707.96	14.40	11.20	12.80	142.222
30000.00	1061.95	21.50	16.50	19.00	211.111
40000.00	1415.93	29.20	22.50	25.85	287.222
50000.00	1769.91	37.10	29.20	33.15	368.333
60000.00	2123.89	45.00	34.50	39.75	441.667
70000.00	2477.88	53.00	42.00	47.50	527.778
80000.00	2831.86	64.00	51.00	57.50	638.889
5000.00	176.99	3.00	1.50	2.25	25.000
10000.00	353.98	5.00	2.50	3.75	41.667
15000.00	530.97	7.50	4.00	5.75	63.889
20000.00	707.96	10.00	6.50	8.25	91.667
30000.00	1061.95	16.00	10.50	13.25	147.222
40000.00	1415.93	22.00	15.00	18.50	205.556
50000.00	1769.91	29.50	21.00	25.25	280.556
60000.00	2123.89	36.50	28.00	32.25	358.333
70000.00	2477.88	43.00	34.50	38.75	430.556
80000.00	2831.86	50.00	42.00	46.00	511.111
5000.00	176.99	3.50	1.50	2.50	27.778
10000.00	353.98	7.00	4.00	5.50	61.111
15000.00	530.97	10.00	7.00	8.50	94.444
20000.00	707.96	12.00	10.00	11.00	122.222
30000.00	1061.95	17.00	16.50	16.75	186.111
40000.00	1415.93	22.00	25.00	23.50	261.111
50000.00	1769.91	27.00	34.00	30.50	338.889
60000.00	2123.89	32.00	42.00	37.00	411.111
70000.00	2477.88	37.00	51.00	44.00	488.889
80000.00	2831.86	43.00	61.00	52.00	577.778
5000.00	176.99	4.00	1.50	2.75	30.556
10000.00	353.98	12.00	3.00	7.50	83.333
15000.00	530.97	16.00	5.00	10.50	116.667
20000.00	707.96	21.50	7.00	14.25	158.333
30000.00	1061.95	31.00	11.00	21.00	233.333
40000.00	1415.93	41.00	17.00	29.00	322.222
50000.00	1769.91	50.00	22.00	36.00	400.000
60000.00	2123.89	58.00	28.00	43.00	477.778
70000.00	2477.88	68.00	34.00	51.00	566.667
80000.00	2831.86	82.00	40.00	61.00	677.778
5000.00	176.99	4.00	3.00	3.50	38.889
10000.00	353.98	7.00	6.50	6.75	75.000
15000.00	530.97	10.00	10.00	10.00	111.111
20000.00	707.96	13.50	13.00	13.25	147.222
30000.00	1061.95	19.50	19.50	19.50	216.667
40000.00	1415.93	27.00	26.00	26.50	294.444
50000.00	1769.91	35.50	32.00	33.75	375.000
60000.00	2123.89	43.50	37.50	40.50	450.000
70000.00	2477.88	52.50	44.00	48.25	536.111
80000.00	2831.86	61.00	50.00	55.50	616.667

FC = 6615.9292

MODULUS OF ELASTICITY = E = 5.2876

CONCRETE CYLINDER COMPRESSION TEST RESULTS

GROUP 6000 21DAYS

P	STRESS	L	R	MEAN	STRAIN
5000.00	176.99	2.00	2.50	2.25	25.000
10000.00	353.98	5.00	6.00	5.50	61.111
15000.00	530.97	8.10	9.20	8.65	96.111
20000.00	707.96	9.00	14.40	11.70	130.000
30000.00	1061.95	11.70	23.00	17.35	192.778
40000.00	1415.93	15.20	30.50	22.85	253.889
50000.00	1769.91	19.20	40.00	29.60	328.889
60000.00	2123.89	22.50	49.50	36.00	400.000
70000.00	2477.88	26.50	57.00	41.75	463.889
80000.00	2831.86	31.00	67.20	49.10	545.556
5000.00	176.99	2.00	2.00	2.00	22.222
10000.00	353.98	6.00	6.00	6.00	66.667
15000.00	530.97	9.50	9.50	9.50	105.556
20000.00	707.96	13.00	13.00	13.00	144.444
30000.00	1061.95	17.20	18.00	17.60	195.556
40000.00	1415.93	24.00	22.50	23.25	258.333
50000.00	1769.91	31.00	29.80	30.40	337.778
60000.00	2123.89	39.50	34.00	36.75	408.333
70000.00	2477.88	48.20	42.00	45.10	501.111
80000.00	2831.86	58.30	48.50	53.40	593.333
5000.00	176.99	3.20	2.00	2.60	28.889
10000.00	353.98	7.20	4.50	5.85	65.000
15000.00	530.97	11.00	8.00	9.50	105.556
20000.00	707.96	15.00	11.00	13.00	144.444
30000.00	1061.95	22.50	16.50	19.50	216.667
40000.00	1415.93	30.00	22.50	26.25	291.667
50000.00	1769.91	39.00	28.00	33.50	372.222
60000.00	2123.89	49.00	33.50	41.25	458.333
70000.00	2477.88	58.00	40.00	49.00	544.444
80000.00	2831.86	68.00	45.50	56.75	630.556
5000.00	176.99	3.50	1.00	2.25	25.000
10000.00	353.98	9.00	3.00	6.00	66.667
15000.00	530.97	12.00	6.00	9.00	100.000
20000.00	707.96	16.50	9.00	12.75	141.667
30000.00	1061.95	23.00	15.00	19.00	211.111
40000.00	1415.93	30.50	22.00	26.25	291.667
50000.00	1769.91	38.50	29.00	33.75	375.000
60000.00	2123.89	45.50	35.50	40.50	450.000
70000.00	2477.88	54.50	43.00	48.75	541.667
80000.00	2831.86	63.50	50.50	57.00	633.333
5000.00	176.99	2.00	2.50	2.25	25.000
10000.00	353.98	5.40	6.10	5.75	63.889
15000.00	530.97	8.10	8.30	8.20	91.111
20000.00	707.96	12.00	12.20	12.10	134.444
30000.00	1061.95	17.00	19.00	18.00	200.000
40000.00	1415.93	22.50	23.50	23.00	255.556
50000.00	1769.91	30.00	30.60	30.30	336.667
60000.00	2123.89	34.50	37.00	35.75	397.222
70000.00	2477.88	42.00	45.00	43.50	483.333
80000.00	2831.86	45.00	55.00	50.00	555.556

FC = 7327.4336

MODULUS OF ELASTICITY = E = 5.3316

CONCRETE CYLINDER COMPRESSION TEST RESULTS

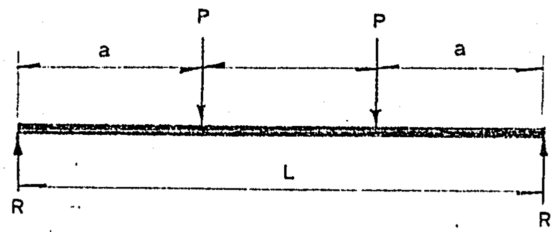
GROUP 6000 28DAYS

P	STRESS	L	R	MEAN	STRAIN
5000.00	176.99	2.40	2.00	2.20	24.444
10000.00	353.98	5.40	5.50	5.45	60.556
15000.00	530.97	9.00	10.00	9.50	105.556
20000.00	707.96	11.00	13.00	12.00	133.333
30000.00	1061.95	16.00	20.00	18.00	200.000
40000.00	1415.93	22.00	26.00	24.00	266.667
50000.00	1769.91	28.00	33.50	30.75	341.667
60000.00	2123.89	34.00	40.00	37.00	411.111
70000.00	2477.88	40.50	47.50	44.00	488.889
80000.00	2831.86	47.00	55.00	51.00	566.667
5000.00	176.99	2.30	2.10	2.20	24.444
10000.00	353.98	6.70	5.00	5.85	65.000
15000.00	530.97	11.00	7.50	9.25	102.778
20000.00	707.96	14.50	11.00	12.75	141.667
30000.00	1061.95	22.50	14.80	18.65	207.222
40000.00	1415.93	30.50	19.00	24.75	275.000
50000.00	1769.91	40.00	23.50	31.75	352.778
60000.00	2123.89	48.00	28.50	38.25	425.000
70000.00	2477.88	56.50	33.00	44.75	497.222
80000.00	2831.86	66.00	38.00	52.00	577.778
5000.00	176.99	2.80	2.50	2.65	29.444
10000.00	353.98	6.20	5.20	5.70	63.333
15000.00	530.97	9.20	8.20	8.70	96.667
20000.00	707.96	12.20	11.00	11.60	128.889
30000.00	1061.95	18.50	16.50	17.50	194.444
40000.00	1415.93	26.00	22.20	24.10	267.778
50000.00	1769.91	34.50	28.50	31.50	350.000
60000.00	2123.89	42.00	33.80	37.90	421.111
70000.00	2477.88	50.00	39.00	44.50	494.444
80000.00	2831.86	58.50	46.00	52.25	580.556
5000.00	176.99	1.00	4.00	2.50	27.778
10000.00	353.98	3.00	7.00	5.00	55.556
15000.00	530.97	7.00	11.00	9.00	100.000
20000.00	707.96	10.00	13.00	11.50	127.778
30000.00	1061.95	18.00	19.00	18.50	205.556
40000.00	1415.93	26.00	24.50	25.25	280.556
50000.00	1769.91	34.00	29.50	31.75	352.778
60000.00	2123.89	43.00	35.00	39.00	433.333
70000.00	2477.88	52.50	40.00	46.25	513.889
80000.00	2831.86	62.00	45.00	53.50	594.444
5000.00	176.99	3.20	1.50	2.35	26.111
10000.00	353.98	8.10	2.70	5.40	60.000
15000.00	530.97	12.00	5.00	8.50	94.444
20000.00	707.96	15.00	8.00	11.50	127.778
30000.00	1061.95	22.00	14.00	18.00	200.000
40000.00	1415.93	29.50	20.00	24.75	275.000
50000.00	1769.91	36.00	25.00	30.50	338.889
60000.00	2123.89	44.00	31.00	37.50	416.667
70000.00	2477.88	50.00	40.00	45.00	500.000
80000.00	2831.86	62.00	44.00	53.00	588.889

FC = 7592.9203
MODULUS OF ELASTICITY = E = 5.3700

APPENDIX B

Calculations of Stiffness of Beams : -



$$\delta_{center} = \frac{Pa}{24EI}(3L^2 - 4a^2)$$

$$= \frac{P(L/3)}{24EI}\left[3L^2 - 4(L/3)^2\right]$$

$$= \frac{PL}{72EI}(3L^2 - 4L^2/9)$$

$$= \frac{23PL^3}{648EI} = \frac{23(33)^3 P}{648EI} = 1,275 \frac{P}{EI}$$

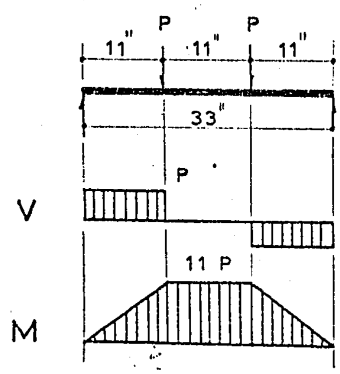
$$EI = 1,275 \frac{P}{\delta}$$

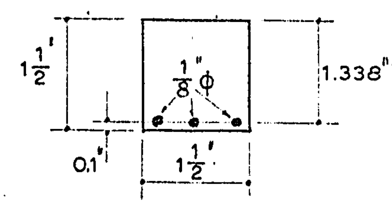
7 days	$EI = 1,275 \frac{37.5}{0.0308} = 1,550,000$
14 days	$EI = 1,275 \frac{25}{0.01955} = 1,680,000$
21 days	$EI = 1,275 \frac{25}{0.0176} = 1,810,000$
28 days	$EI = 1,275 \frac{50}{0.0333} = 1,915,000$

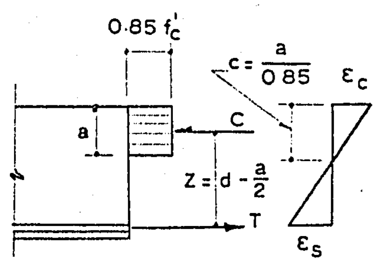
APPENDIX C

Ultimate Load P_u :

(a) Flexural Fail :-







$A_s = 3 \times 0.01225 = 0.0368$ sq.in.

$d = 1\frac{1}{2} - 0.1 - 1/16 = 1.338''$

$p = A_s/bd = \dfrac{0.0368}{1.5(1.338)} = 0.01835$

$p \leq 0.75\ p_b = 0.75 \left[\dfrac{0.85\ K_1\ f'_c}{f_y} \left(\dfrac{87{,}000}{87{,}000 + f_y} \right) \right]$

where $K_1 = 0.85$

7 days $\qquad f'_c = 4{,}688.8$ psi. , $f_y = 86{,}000$ psi.

$0.75\ P_b = \left[\dfrac{0.85(0.85)4{,}688.8}{86{,}000} \times \dfrac{87{,}000}{87{,}000 + 86{,}000} \right] 0.75$

$= (0.0393 \times 0.502)\ 0.75$

$= 0.0148 < 0.01835$ not O.K.

$A_s = 0.0148(1.5)(1.338) = 0.0297$ sq.in.

$C = T$

$0.85\ f'_c\ ab = A_s\ f_y$

$3{,}980\ ab = 0.0297 \times 86{,}000 = 2{,}550$

$a = \frac{2{,}550}{3{,}980(1.5)} = \frac{2{,}550}{5{,}960} = 0.428$

$c = \frac{0.428}{0.85} = 0.504$

$\varepsilon_s = \frac{f_y}{E_s} = \frac{86{,}000}{29{,}100{,}000} = 0.00296$

$\varepsilon_c = \frac{0.00296(0.504)}{1.338 - 0.624} = 0.00212 < 0.003$ O.K.

$M_u = T\ Z = 2{,}550(1.086) = 2{,}770$

$11\ P = 2{,}770$

$P = 251$ lb.

14 days $f'_c = 5{,}620$ psi. , $f_y = 86{,}000$ psi.

$0.75\ P_b = (0.724 \times \frac{5{,}620}{8{,}600} \times \frac{8{,}700}{17{,}300})\ 0.75$

$= 0.0178 < 0.01835$ not O.K.

$C = T$

$A_s = 0.0178(1.5)(1.338) = 0.0357$

$0.85\ f'_c\ ab = A_s\ f_y = 0.0375(86{,}000) = 3{,}070$

$a = \frac{3{,}070}{7{,}160} = 0.428$

$c = \frac{0.428}{0.85} = 0.504$

$\varepsilon_s = 0.00296$

$\varepsilon_c = \frac{0.00296(0.504)}{(1.338 - 0.52)} = 0.00182 < 0.003$ O.K.

$M_u = 11P = TZ = 3,070(1.124) = 3,450$

$P = 314$ lb.

21 days $f'_c = 6,163.1$ psi. , $f_y = 86,000$ psi.

$0.75\ P_b = (0.0519 \times 0.502)\ 0.75 = 0.0195 > 0.01835$ O.K.

$a = \frac{3,160}{7,850} = 0.402$

$c = 0.474$

$\varepsilon_c = \frac{0.00296 \times 0.474}{1.338 - 0.474} = 0.00162 < 0.003$ O.K.

$11P = 3,160\ (1.137) = 3,590$

$P = 326$ lb.

28 days $f'_c = 6,721.8$ psi. , $f_y = 86,000$ psi.

$0.75\ P_b = 0.0213 > 0.01835$ O.K.

$a = 0.369$

$c = 0.434$

$\varepsilon_c = 0.00103 < 0.003$ O.K.

$M_u = 11P = 3,160(1.154) = 3,650$

$P = 332$ lb.

(b) Shear Fail : -

$v_{cr} = \frac{V_{cr}}{bd} = 1.9\sqrt{f'_c}$

$V_{cr} = 1.9\sqrt{f'_c}\ bd$

$= 1.9\ (1.5)\ (1.338)\sqrt{f'_c}$

$= 3.81\sqrt{f'_c}$

7 days $V = P_u = 3.81\sqrt{4,688.8}$

$P_u = 261$ lb.

14 days $V = P_u = 3.81\sqrt{5,620}$

$P_u = 286$ lb.

21 days $V = P_u = 3.81\sqrt{6,163.1}$

$P_u = 299$ lb

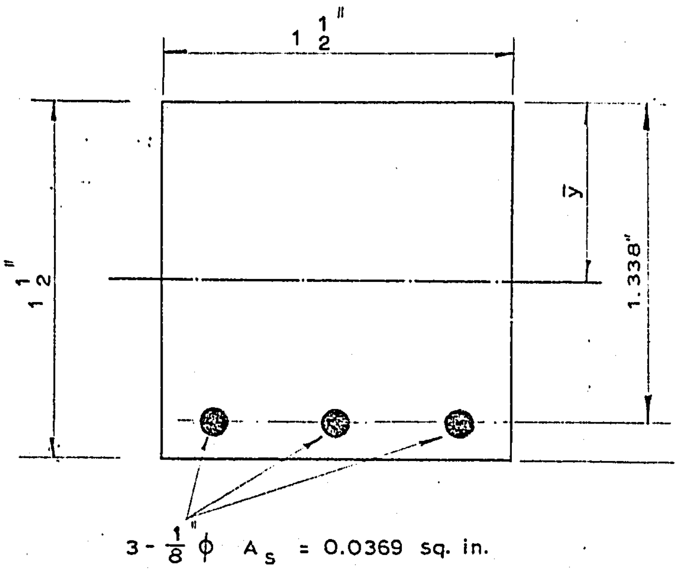
28 days $V = P_u = 3.81\sqrt{6,721.8}$

$P_u = 313$ lb.

APPENDIX D

Calculation of Moments of Inertia:

(a) Uncracked Section:-



7 days

$$E_c = 57{,}500\sqrt{f'_c} = 57{,}500\sqrt{4{,}688.8} = 3{,}940{,}000 \text{ psi.}$$

$$n = \frac{E_s}{E_c} = \frac{29{,}100{,}000}{3{,}940{,}000} = 7.4$$

$$A_s(n-1) = 0.0368(6.4) = 0.236 \text{ sq. in.}$$

$$1.5\,\bar{y}\,(\frac{\bar{y}}{2}) = \frac{(1.5-\bar{y})}{2}\,1.5 + 0.236(1.338-\bar{y})$$

$$\bar{y} = 0.805''$$

$$I = \frac{1.5(1.5)^3}{12} + (1.5)(1.5)(0.055)^2 + 0.236(0.533)^2$$

$$= 0.4961 \text{ in}^4$$

14 days

$$E_c = 57{,}500\sqrt{5{,}620} = 4{,}310{,}000 \text{ psi}$$

$$n = \frac{29,100,000}{4,310,000} = 6.75$$

$$A_s(n-1) = 0.211 \text{ sq. in.}$$

$$\bar{y} = \frac{1.973}{2.461} = 0.8''$$

$$I = 0.422 + 0.0056 + 0.0611 = 0.4887 \text{ in}^4$$

21 days

$$E_c = 57,500\sqrt{6,163} = 4,500,000 \text{ psi}$$

$$n = 6.46$$

$$A_s(n-1) = 0.201 \text{ sq. in.}$$

$$\bar{y} = \frac{1.959}{2.451} = 0.799''$$

$$I = 0.422 + 0.0054 + 0.0605 = 0.4879 \text{ in}^4$$

28 days

$$E_c = 57,500\sqrt{6,721.8} \doteq 4,700,000 \text{ psi}$$

$$n = 6.2$$

$$A_s(n-1) = 0.191 \text{ sq. in.}$$

$$\bar{y} = \frac{1.946}{2.441} = 0.796 \text{ ''}$$

$$I = 0.422 + 0.00475 + 0.056 = 0.48275 \text{ in}^4$$

(b) Cracked Section:-

7 days

$$E_c = 3,940,000 \text{ psi} \quad , n = 7.4$$

$$A_s(n-1) = 0.236 \text{ sq. in.}$$

$$1.5\,\bar{y}\,\frac{\bar{y}}{2} = \frac{(1.338 - \bar{y})^2}{2}\,1.5 + 0.236\,(1.338 - \bar{y})$$

$$\bar{y} = 0.74''$$

$$I = \frac{1.5(1.338)^3}{12} + (1.5)(1.338)(0.071)^2 + 0.236(0.598)^2$$

$$= 0.299 + 0.0101 + 0.0843 = 0.3934 \text{ in}^4$$

14 days

$E_c = 4{,}310{,}000$ psi.

$n = 6.75$

$A_s(n-1) = 0.211$ sq. in.

$$\bar{y} = \frac{1.623}{2.211} = 0.735 ''$$

$$I = 0.299 + 0.00872 + 0.0768 = 0.38452 \text{ in}^4$$

21 days

$E_c = 4{,}500{,}000$ psi.

$n = 6.46$

$A_s(n-1) = 0.201$ sq. in.

$$\bar{y} = \frac{1.608}{2.201} = 0.729 ''$$

$$I = 0.299 + 0.00723 + 0.0745 = 0.3805 \text{ in}^4$$

28 days

$E_c = 4{,}700{,}000$ psi.

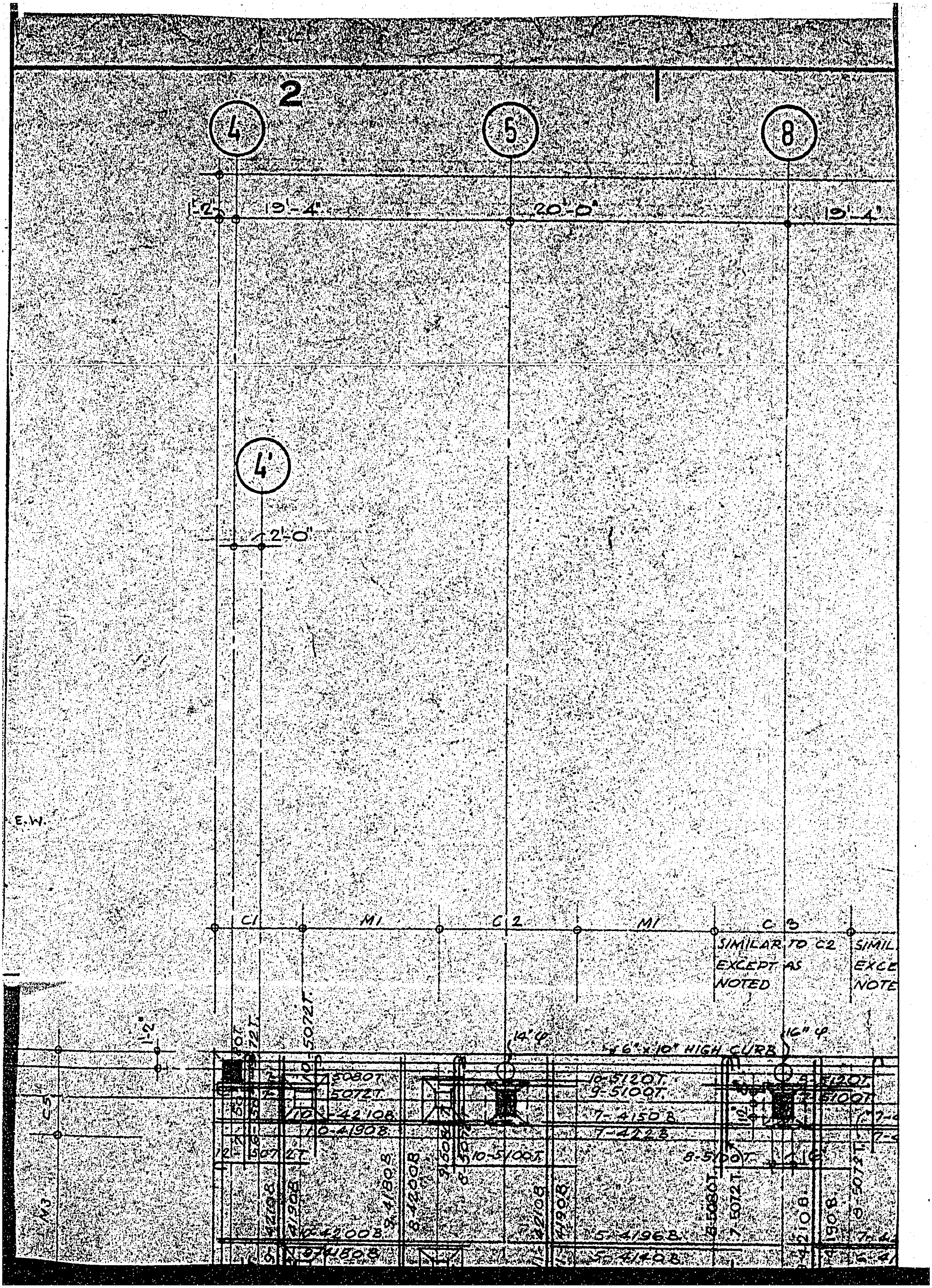
$n = 6.2$

$A_s(n-1) = 0.191$ sq. in.

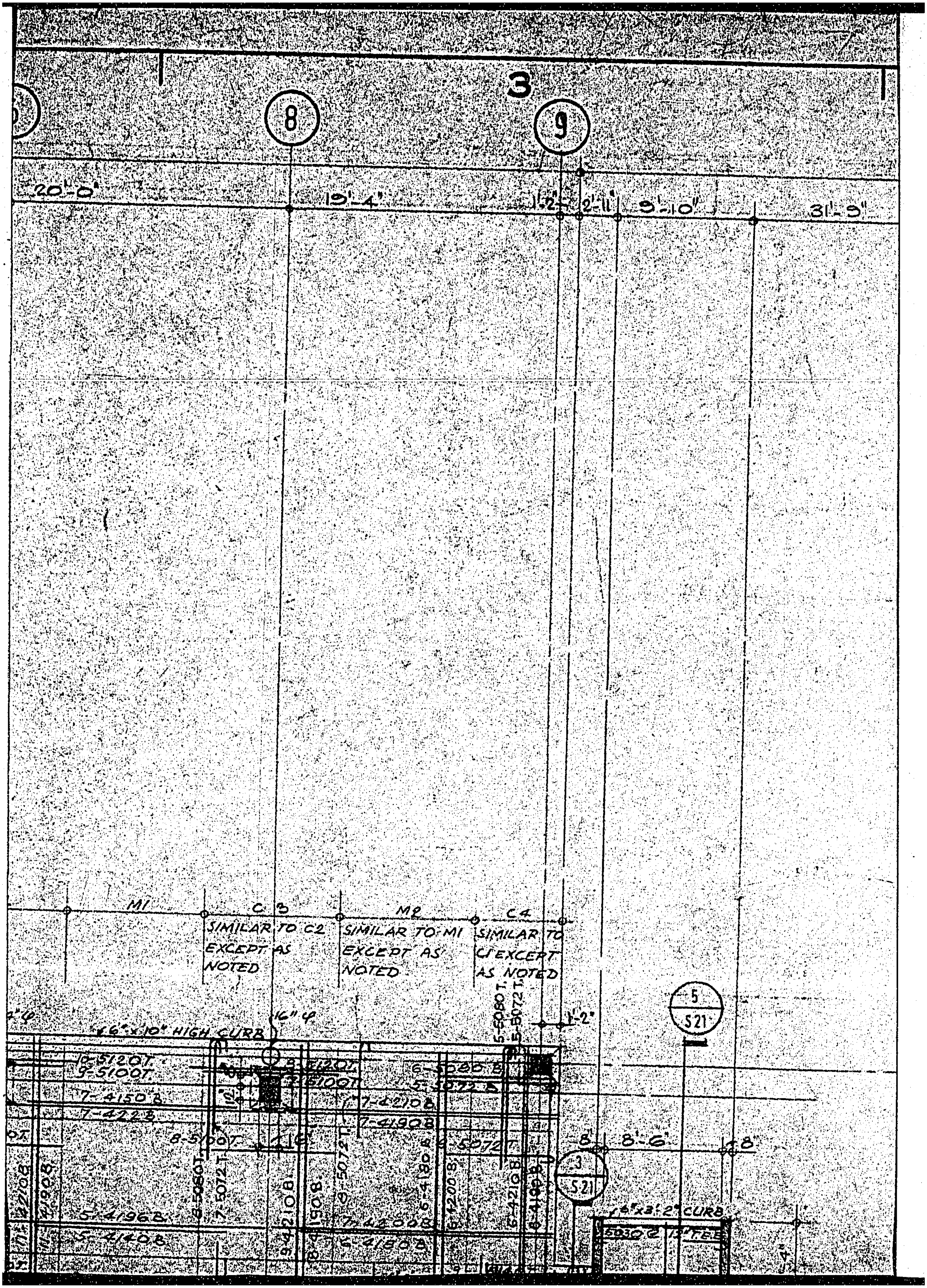
$$\bar{y} = \frac{1.596}{2.191} = 0.728 ''$$

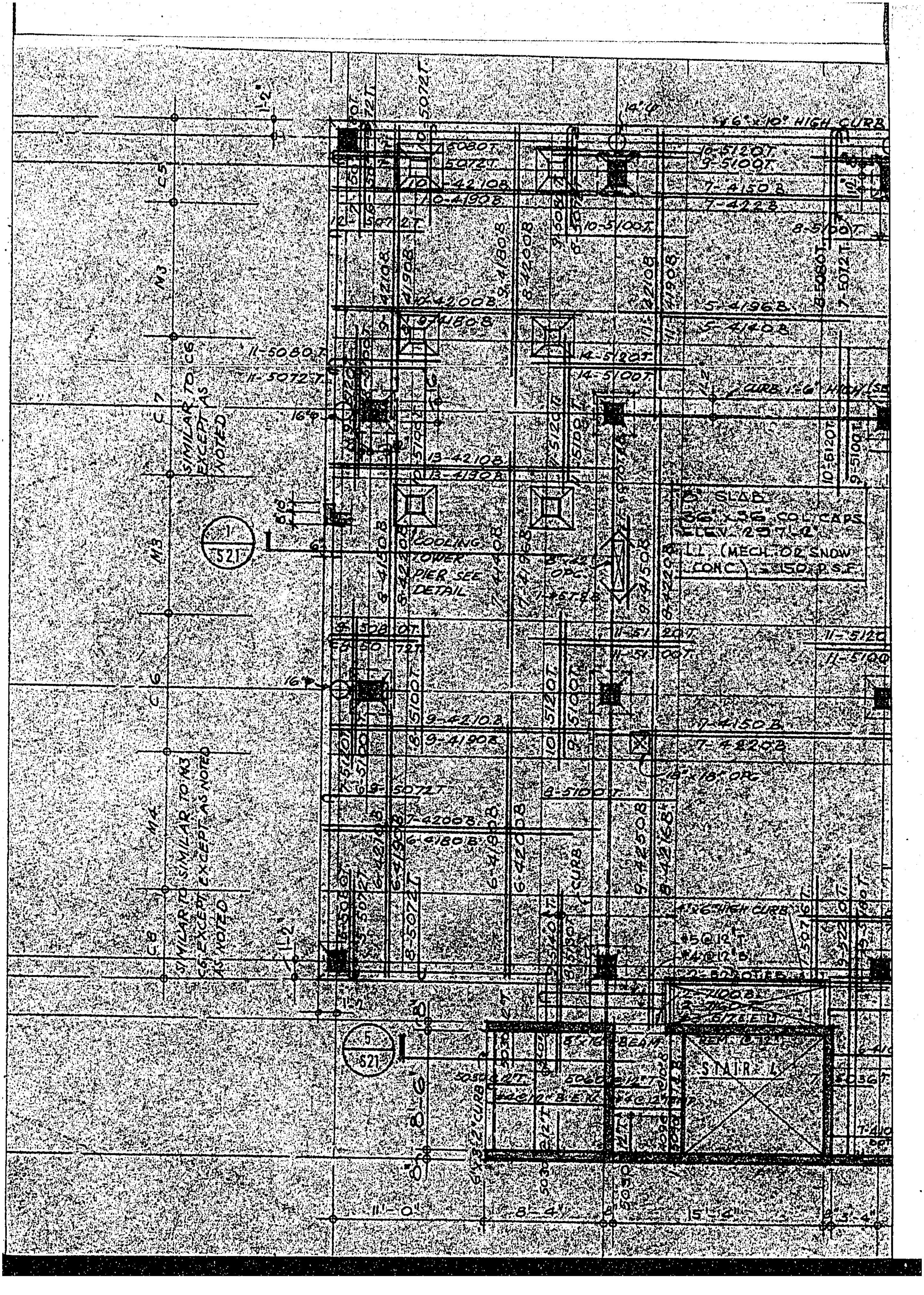
$$I = 0.299 + 0.00699 + 0.071 = 0.3770 \text{ in}^4$$

APPENDIX E

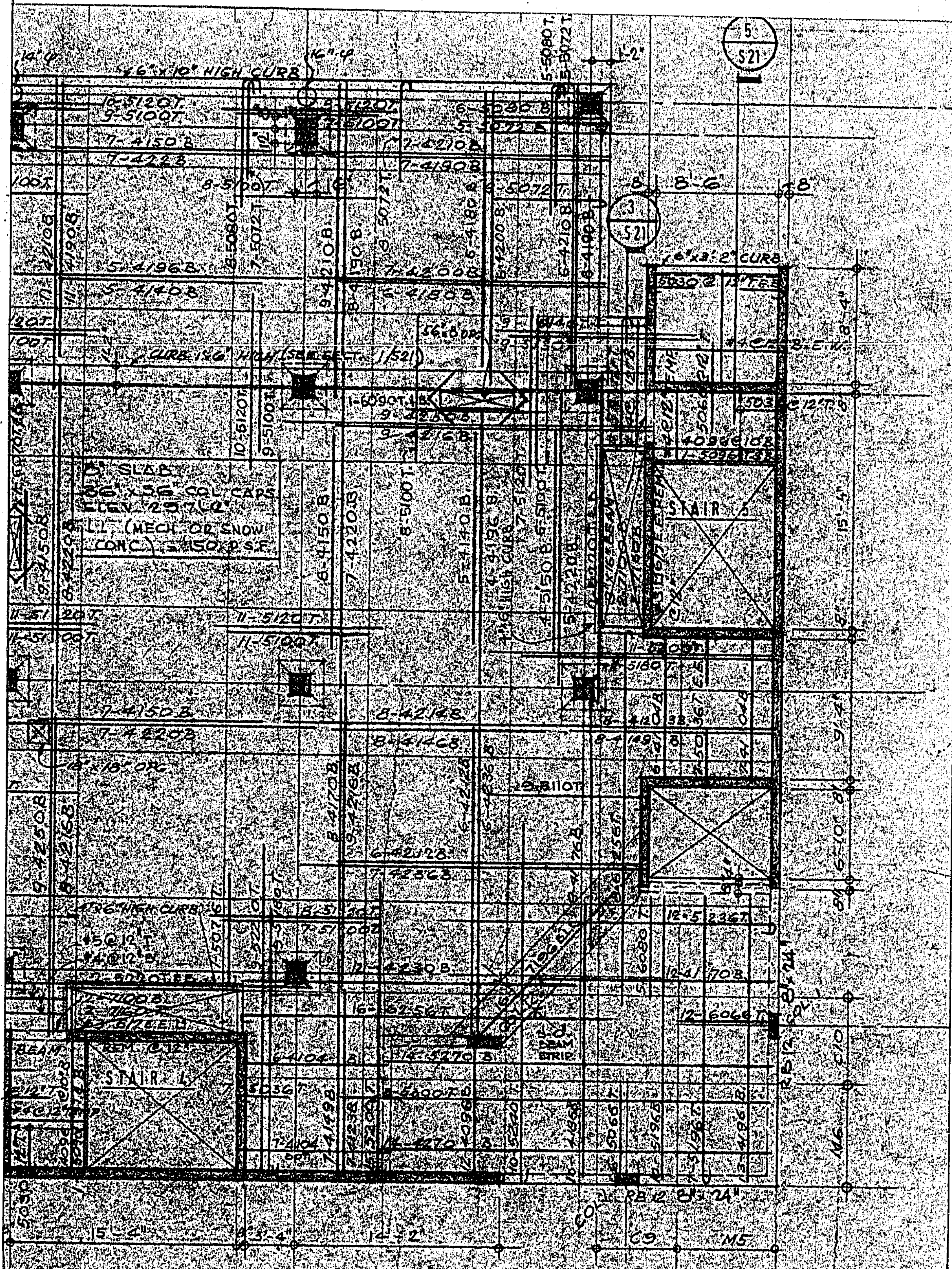


2
4
5
8
1'-2"
19'-4"
20'-0"
19'-4"
4'
2'-0"
E.W.
C1
M1
C2
M1
C3
SIMILAR TO C2
EXCEPT AS
NOTED
14" φ
6" x 10" HIGH CURB
16" φ
10-5120T
9-5100T
7-4150B
7-422B
8-5100T
10-5100T
5-4196B
5-4140B
5080T
5072T
10-4210B
10-4190B
9-4180B
8-4200B
C5
N3
1'-2"


3
8
9
20'-0"
19'-4"
1'-2"
2'-11"
9'-10"
31'-9"
M1
C3
M2
C4
SIMILAR TO C2 EXCEPT AS NOTED
SIMILAR TO M1 EXCEPT AS NOTED
SIMILAR TO C1 EXCEPT AS NOTED
5
S21
6"x10" HIGH CURB
16"φ
1'-2"
10-5120T
3-5100T
7-4150B
7-4210B
7-4190B
6-5080B
5-5072B
6-5072T
5-5080T
5-5072T
8'-6"
3
S21
5-4196B
5-4140B
7-4200B
6-4180B
6"x3'-2" CURB



6" x 10" HIGH CURB
CURB 1'-6" HIGH
SIMILAR TO C6 EXCEPT AS NOTED
SIMILAR TO M3 EXCEPT AS NOTED
SIMILAR TO C6 EXCEPT AS NOTED
COOLING TOWER PIER SEE DETAIL
8" SLAB
36" x 36" COL. CAPS
ELEV. 297'-2"
L.L. (MECH. OR SNOW CONC.) = 150 P.S.F.
18" x 18" OPG.
4" x 26" HIGH CURB
STAIR
11'-0"
8'-4"
15'-8"


6"x10" HIGH CURB
S21
8'-6"
6"x3'-2" CURB
CURB 1'-6" HIGH (SEE SECT. 1/S21)
SLAB
36"x36" COL. CAPS
L.L. (MECH. OR SNOW CONC.) = 150 P.S.F.
STAIR 5
15'-4"
18"x18" OPG.
STAIR 4
BEAM
5'-4"
14'-2"
C9
M5