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DYNAMIC CHARACTERISTICS OF NEW ZEALAND HEAVY FLOORS

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PREFACE

This study documents the second phase of a research programme undertaken by BRANZ to prepare design information for occupant-induced floor vibrations. Vibration tests were conducted on heavy floor systems (those constructed with a concrete slab or concrete topping) to obtain their damping and frequency parameters.

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AUDIENCE

This report is intended for structural engineers, architects, floor manufacturers, floor designers and building managers. Other workers in the field of engineering research may also find this work useful.

NOTE

The work reported here includes the assessment of some named proprietary systems.

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DYNAMIC CHARACTERISTICS OF NEW ZEALAND HEAVY FLOORS

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KEYWORDS

From Construction Industry Thesaurus - BRANZ edition: Acceleration; Damping; Displacement; Design Methods; Fast Fourier Transform; Floor Systems; Frequency; Heel Drop Test; Occupant-induced; Impulse Hammer; Standard Heel Drop Test; Vibrations; Walking.

ABSTRACT

This work follows on from earlier work which was a literature survey of design methods for floors subjected to occupant-induced vibrations. Although design methods were recommended, dynamic parameters (damping, frequency) obtained from tests carried out overseas may not be suitable for New Zealand. This report describes tests to quantify dynamic parameters of New Zealand heavy floors so that recommended design methods can be used. In this report heavy floors are those constructed with a concrete topping or concrete slab.

Human heel drop was used to excite all tested floors. Some floors were also excited with a modular-tuned impulse hammer. In two of the precast rib floors, a mechanical impactor calibrated to simulate standard heel drops (i.e., the "standard heel drop" impactor) was used. Measured fundamental frequency of floors was significantly higher than those calculated, and there was not a good agreement between measured and calculated peak accelerations. Impulsive response design methods appeared suitable for New Zealand application. Consideration of resonant response resulting from walking is generally not warranted in New Zealand because of the higher inherent stiffness of New Zealand floors.

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DYNAMIC CHARACTERISTICS OF NEW ZEALAND HEAVY FLOORS

1.0 INTRODUCTION

Currently in New Zealand, there are few reported cases of floors with problems caused by occupant-induced vibrations. Traditional design procedures that limit the static stiffness of floors have been adequate in preventing troublesome floor vibrations (due to the weight of the floor and supporting structure). New Zealand Standard NZS 4203:1984 Code of Practice for General Structural Design and Design Loadings for Buildings (SANZ, 1984) recommends limiting the vertical static deflection of floors, under the design live load, to 0.4% of the span. However, current practices within the New Zealand construction industry are following overseas trends to minimise costs by greater utilisation of long span and lighter weight floors using higher strength materials. These floors are more flexible and may lead to an increase in the number of vibration problems.

Sole use of the static stiffness criterion is inadequate for such floors. In the previous phase of this project (Lim, 1991) a number of design methods were recommended for both light and heavy-weight floors. Light-weight floors are those constructed using a timber deck and having timber, steel, or composite steel-timber or composite-wood supporting joists. It was also suggested that dynamic parameter (damping, frequency) values obtained from overseas tests may not be suitable for New Zealand floors.

The main objective of the current work was to conduct vibration tests on heavy floors (those constructed with a concrete topping or concrete slab) to obtain their dynamic parameters, enabling application of the recommended design methods. Other objectives were to study the effects that partitions had on the damping of the floors and to compare the various design methods.

2.0 EXPERIMENTAL RESULTS

2.1 Description of Floors

A number of New Zealand heavy floor systems were tested in situ between March and June 1990.

Apart from the "Dycore" floor system, all floors had a ceiling and mechanical equipment installed. Floors that were occupied were also carpeted. Partitions were constructed mostly of lined timber framing. Partitions extended at least to ceiling height and were fixed to the floor as well as "return walls". Some partitions may have been attached to the ceiling framing members also.

Figures 1(a)-1(i) show the floor plans and sections through some of the floors tested. The types of floor systems studied were:

(a) In situ slab

The two-way slab was 114 mm thick and cast in place. It was supported by concrete encased steel frames at 5.4 and 8.1 m centres. Concrete encased secondary steel beams were positioned at the midspan of the 8.1 m beams, reducing slab panels to centreline dimensions of 4.05x5.4 m. One unoccupied but partitioned floor was studied. Another floor studied had smaller secondary beams and was unoccupied and unpartitioned.

(b) Precast "Unispan"

Each precast "Unispan" unit was 75 mm thick by 1.2 m wide and supported by reinforced concrete frames. The floor areas of the two locations tested were 6.0x6.5 m and 4.55x5.3 m. An in situ concrete topping of 70 mm was cast over the Unispan. Both floor areas were unoccupied and considered to be unpartitioned, even though adjacent areas were occupied and partitioned.

(c) Precast rib

Four reinforced concrete-framed buildings with Stahlton precast rib floor systems were investigated. Tests were done on a 10.05 m span of the first building. Precast rib with an overall depth of 300 mm (200 mm rib depth) was used. The floor was partitioned but unoccupied.

The second building had a clear floor span of 6.4 m and was supported by a rib of 160 mm depth. Tests were conducted at the same plan position on two different floor levels. One floor was unoccupied and unpartitioned; the other was occupied and partitioned.

Two locations of one floor were tested in both of the last two buildings. At the time of testing, the floor of an 8 m span building was partially completed with workmen working on the floor. This floor had been partitioned. The floor of the other building, which was partitioned and occupied had a span of 8.4 m. Precast ribs with an overall depth of 250 mm (150 mm rib depth) were used. The 8.4 m span building had only two spans, i.e., a building width of about 17 m.

(d) Composite steel-concrete

Secondary steel beams (310UB40), with a span of 8.6 m and at 2.5 m centres supported the 0.75 mm High Strength "Hibond" tray deck. Primary steel girders (360UB51), with a 7.2 m span, supported the secondary beams. The in situ concrete was 65 and 120 mm thick, at the crest and trough of the tray deck section, respectively. Tests were conducted at the same plan position of two different floor levels, two test locations per floor. One floor was unpartitioned and unoccupied; the other was partitioned and occupied.

(e) Precast "Dycore"

Precast Dycore, 200 mm thick by 1200 mm wide, with a 100 mm thick in situ concrete topping was used to span 8.4 m. Precast concrete beams (800 mm deep x 600 mm wide) and cast in situ reinforced concrete columns (500 mm square at 8.4 m centres both ways) supported the Dycore units. The floor was an empty carpark.

(f) Precast double tees

Precast cropped double tee units understood to have supplied by Stresscrete; 450 mm deep and with a 65 mm concrete topping, were used to span 14.9 m. These were supported by reinforced concrete frames and shear walls. The floor was partitioned and occupied.

2.2 Test Procedure

The floors were impacted primarily with human "heel drop" and a "modular-tuned hammer" (PCB Piezotronics Incorporated, 1989). In some instances, a "standard heel drop" impactor was used.

In the heel drop test, a 78 kg person raised his heels by about 50 mm, shifted his body weight onto his heels, then dropped his heels; thereby impacting the floor. Three heel drops were imparted at each test location.

Comparative tests were done by the same person in the laboratory on a 100 kN loadcell (sensitivity of 1 Volt/10 kN). The average peak force measured was 1.80 kN (cf. 2.72 kN for the standard), and the average impulse after 50 ms was 48 Ns (cf. 68 Ns). These were 66 and 71% respectively of the simulated standard heel drop derived by Lenzen and Murray (1969).

In the hammer impact test, a person impacted the floor using a 5 kg modular-tuned hammer with a force transducer (sensitivity of 0.22 Volts/kN) installed. Where used, the hammer was hit three times at each location.

Modal tuning is used for the following three reasons:

(a) it eliminates spurious responses in the frequency spectrum resulting in a nearly flat spectrum over a wide frequency range, i.e., a single impact is capable of exciting all resonances within that frequency band;

(b) it reduces double hits of the hammer;

(c) and it eliminates hammer resonance from the test results.

The main disadvantage of the hammer impact test is that it has the additional weight of the person impacting the floor. The force input also varies by the amount of force exerted on the hammer.

The "standard heel drop" impactor (Figure 2a), was fabricated using the specification of Murray (1990). The impactor was calibrated to simulate the peak force and impulse of the standard heel drop. The impactor was triggered by using a light stick to prop the impactor, then removing the stick by pulling an attached string. The string, which was a minimum of 3 m long, prevented additional weight (of the person controlling the impactor) being added to the point of floor excitation. The peak force and impulse resulting from the "standard heel drop" impactor was also measured in the laboratory. The average peak force of 40 tests was 3.06 kN (cf. 2.72 kN) and the average impulse at 50 ms was 75 Ns (cf. 68 Ns). Three "standard heel drop" impacts were imparted on the test locations of the 8.0 and 8.4 m span precast rib floor system.

An integrated-circuit piezoelectric accelerometer (PCB Piezotronics Incorporated, 1989) was positioned adjacent to the point of impact, to measure the vertical acceleration at the midspan of each floor. In some instances, the accelerometer measured the acceleration of the floor when the impact was applied at an adjacent bay or at a supporting beam. The accelerometer (with a built-in amplifier), has a sensitivity of 1.143 Volts/g, where g is the acceleration due to gravity. In carpeted floors, the accelerometer was screwed to a rigid aluminium block with pins on its underside and the pins protruded through the carpet to the top of the floor. The accelerometer sat directly on not-carpeted floors.

Output signals from the accelerometer and modular-tuned hammer were low-pass filtered at 50 Hz before data were recorded on an IBM compatible personal computer (Figure 2b). Data were collected using a waveform scroller board controlled under "CODAS" (Dataq Instruments, 1988) software. The floor response as measured by the accelerometer was also displayed in "real-time" on the computer. Data were collected at 250 samples/s per channel.

Depending on the author's perception, each floor was "rated" either "Satisfactory" (S) or "Noticeable" (N) in response to a person walking on the floor. A "Noticeable" rating meant that vibrations were either "Borderline" or "Unsatisfactory". Rating the floor was done to compare the author's perception with the response predicted using the design methods described in section 3.2. In occupied floors the rating given by the author was similar to that felt by occupants of floors. This subjective method, however imprecise, was the best available at the time. Because of this subjectivity, however, the "Unsatisfactory" rating was not used. Note that the accelerometer was not sensitive enough to record walking acceleration levels.

2.3 Data Analysis and Results

Signal analysis of the data was carried out using the "DADiSP" (DSP Development Corporation, 1987) software. Peak accelerations from heel drops and hammer impacts, and peak force of the hammer, were determined directly from the signals of the floor response (Figure 2c, 2d).

A Fast Fourier Transform (FFT) of each signal was performed using 512 samples (Figure 2e). Zeros were added at the end of the signal if the actual sample length was less than 512 points. If energy leakage needed to be reduced before the FFT operation the signal was multiplied with the "Hanning" window, described by Blackman and Tukey (1959). At 250 samples/s, a resolution error of approximately 0.5 Hz (i.e., 250/512) resulted. The "DADiSP" fourier transform of data produces a "real" and "imaginary" component for each data point. Because "DADiSP" plots only the "real" part, and stores the "imaginary" value of the amplitude, actual acceleration amplitude of each signal of the FFT operation was obtained by performing the square root of the sums of squares of the "real" and the "imaginary" values. The fundamental natural frequency and the damping of each floor was obtained by using this modified FFT plot.

Two methods were used to obtain the floor damping (as a % of the critical damping). Firstly, damping was evaluated in the time domain from the decay of the acceleration response curve (Figures 2c and 2d), (Chui and Smith (1986)).

$$D = \ln (a_o/a_n)/2\,n \qquad .... [1a]$$

where a_o = peak acceleration amplitude of the first peak and
a_n = acceleration amplitude of the n^{th} peak.
n = number of cycles

The second method was the "half-power bandwidth" method in the frequency domain. This is illustrated in Figure 2e where the damping, (Chui and Smith (1988b)).

$$D = (P2-P1)/(P2+P1) \quad .... \text{[1b]}$$

Peak displacement of each signal was obtained by performing an inverse FFT. The signal was filtered at 2.5 times the measured fundamental natural frequency, and divided by 4 before performing the inverse FFT operation (Rainer, 1986).

Table 1 summarises types of floor construction and the average frequency and damping of the three signals at each test location. At each location and for each type of impact, the maximum "peak acceleration" and "peak displacement" of the three signals (rather than the average) were plotted. Note that maximum "peak displacement" was derived from the signal that produced the maximum "peak acceleration", i.e., the maximum "peak displacement" was never recorded from one signal and the maximum "peak acceleration" from another.

3.0 CALCULATION PHASE

3.1 Dynamic Properties of Floors

The parameters affecting discomfort felt by people on vibrating floors are frequency, peak acceleration, peak displacement, and damping. CSA (1984).

3.1.1 Frequency

For isotropic floors simply supported on four edges, the fundamental natural frequency (f) may be determined from,

$$f_o = \pi(L^2 + B^2)/(2L^2B^2)\sqrt{D^*/M} \quad .... \text{[2] CSA (1984)}$$

The fundamental natural frequency (f_o) of one-way floors can be approximated by

$$f_o = K'\sqrt{gEI/(WL^3)} \quad .... \text{[3] CSA (1984)}$$

The floor frequency (f) in one-way floor systems supported on flexible steel girders can be approximated by Dunkerly's formula,

$$\frac{1}{f^2} = \frac{1}{f_o^2} + \frac{1}{f_g^2} \quad .... \text{[4] CSA (1984)}$$

where f_o is found from equation 3 and
g = acceleration due to gravity = 9810 mm/s ;
K' = 1.57 for simply supported beams,
= 0.56 for cantilevered beams,
= 2.45 for fixed/pinned support,
= 3.56 for floors fixed at both ends,
= can be obtained from Figure 3, for two or three spans continuous beams with unequal spans;
I = the uncracked transformed moment of inertia of the floor section based on a Tee-beam model (mm);
$D^* = Et_c^3 / [12(1- \mu 2)]$
E = 200000 Mpa for steel,
= 30340 Mpa for concrete assuming a density of 2400 kg/m ;
W = the dead load plus 10% of live load, supported by the floor in N;
t_c = the concrete thickness or the equivalent concrete thickness in m;
μ = concrete's poisson ratio = 0.3;
L and B are the span and width of the floor in mm for equation [2], and in m for equation [3];
M = the mass of the floor system in kg/m^3;
f_g = fundamental frequency of the supporting girder.

Murray (1989) recommended that 10-25% of the design live load be included in the weight when calculating the fundamental frequency while Wyatt (1989) recommended only 10%. (Transient vibration in offices is most noticeable when the floors are lightly loaded). The value of 10% was used when applying the design methods in this report.

A superimposed dead load of 0.4 kPa was included in the calculations of fundamental frequency to take into account the weight of carpets, ceilings, mechanical equipments, partitions, facade cladding and furniture. A dead load of 0.25 kPa was assumed for the supporting beams in floors in which the supporting beams participated in the floor response, such as the composite steel-concrete floor. Calculation of the fundamental frequency of the floors tested is shown in Appendix A.

3.1.2 Peak acceleration

An approximation of peak acceleration (a_o, in %g) from heel drop impact for floors with spans greater than 7 m, and frequency less than 10 Hz is given by Canadians Standards Association ((CSA), 1984) as,

$$a_o = 60 f_o / W B_1 L \qquad \text{..........} [5]$$

where W = the weight of floor plus 10% live load, in kPa;
L = the span of the beam in m;
B_1 = width of the equivalent beam in m = 40 t_c, where t_c is the equivalent concrete thickness in floors on rigid supports.

The CSA method should not be used if either the floor span or fundamental frequency is outside the above limitations.

3.1.3 Peak Displacement

The initial/peak displacement of a floor system due to a heel-drop impact is given by Murray (1989) as,

$$A_o = A_{ot} / N_{eff} \qquad \text{..........} [6]$$

$$N_{eff} = 2.97 - S/(17.3t_c) + L4/200EI) \qquad \text{..........} [7]$$

Clifton (1989) provides an alternative expression for N_{eff} for all secondary beam spacings:

where A_o = initial displacement of the floor system due to heel drop;
A_{ot} = initial displacement of a single tee beam due to heel drop impact
= DLF d_s;
DLF = dynamic load factor from Table 5;
d_s = static deflection due to a 2.67 kN force;
N_{eff} = number of effective tee beams;
s = secondary beam spacing greater than 750 mm. Murray provided an alternative expression to obtain N_{eff} for s less than 750 mm (E, I, L, t defined as above).

3.1.4 Estimated damping

From Murray's (1989) recommendations, a finished floor with ceilings and ducts would have a minimum damping of 4% (2% for the bare floor, and 1% each from ceiling and ducts). Partitions attached to the floor system and which are spaced not more than five secondary floor beams (or the effective floor width), would add a further 10-20% damping. Appendix G (CSA, 1984) recommends damping values of 3, 6 and 12% for bare composite floors, finished floors with ceilings and ducts, and for finished floors with partitions, respectively.

Allen (1990) noted that the measured damping values obtained from heel drop impact include components for geometric dispersion and viscous damping. The geometric component of damping does not provide a means of energy dissipation under resonant response, and hence does not reduce the annoyance potential of vibrations. After modal analysis, he further noted that this component was half of the total damping estimated from heel drop measurements. Therefore, when considering resonant response in floors, he recommends damping values of 1.5% for bare floors; 3% for finished floors with ceilings, ductwork and mechanical fittings; and 4.5% for finished floors with full-height partitions. Wyatt (1989) also recommended the same damping values when considering resonant response.

The contradictory damping values, given in the last two paragraphs, could arise because the Murray and CSA values are for transient or impulsive responses, whereas the Allen and Wyatt values are for resonant response problems. Damping values used in calculations in this report were 1.5%, 3.0% and 4.5% when considering resonant response as mentioned by Allen (1990); and 2.0%, 4.0% and 6.0% when considering impulsive response for a bare floor based on Murray's (1989) recommendations; a finished floor with ceiling and ducts, and with partitions installed respectively.

Table 2 compares the calculated and measured values of the floor systems under human heel drop and from the "standard heel drop" impactor.

3.2 Application of Design Methods

Since all but one of the floors investigated were in office buildings, only the response to walking vibrations was evaluated. The Dycore carparking floor was assumed to be under office occupancy so a comparison could be made with the other floor systems.

Wyatt's (1989) design methods for assessing the likely vibrational behaviour of floors in steel framed buildings both to impulsive and resonant response were only applied for the composite-steel concrete floor situation.

Floors rated satisfactory when evaluated using any design method are not necessarily satisfactory under in-service conditions. Rather, the design methods are aimed to check potential vibration problems, and remedial measures (including those described by Lim 1991) should be taken.

3.2.1 Impulsive response

Appendix G of CAN3-S16.1-M84 (CSA, 1984) outlines an empirical design method for assessing the required damping in floors. The dashed lines in Figure 4 relate to acceleration limits obtained from heel impact tests, and are used as semi-empirical criteria for evaluating the response of a floor to transient walking vibrations. The solid line in Figure 4 is the recommended acceleration limit for continuous vibrations. The greater the damping values the greater the acceleration limits the floor can have under walking vibrations.

Fundamental frequency and peak acceleration from heel drop impact were computed for each floor. Using these two parameters, a point was plotted in Figure 4 to obtain the required damping. The floor was considered to be satisfactory when the estimated floor damping was greater than that required from the graph. This method was only applied to floors with a fundamental frequency of less than 10 Hz, since equation [5] is only applicable to such floors.

Murray (1989) formulated an empirical method from tests conducted on composite steel-concrete floors. The method is valid for damping between 4-6% critical, and conservative for damping greater than 6%. Murray (1990) has extended the method to precast concrete floors and found it to be satisfactory. His criterion is very conservative for floors with a fundamental frequency greater than 10 Hz. The method proposes that a floor system will be satisfactory if the estimated damping,

$$D > 1.38A_o f_o + 2.5 \quad \text{..........} \quad [8]$$

To evaluate if vibration problems exist in floors with the fundamental frequency greater than 7 Hz, Wyatt (1989) derived the following design method. The response factor, R, is:

$$R = 30000/MB_2L \quad \text{..................} \quad [9]$$

where M = dead load plus 10% live load in kg/m^2;
B_2 = the lesser of the secondary floor beam spacing or 40 t_c where t_c is the equivalent concrete thickness in m;
L = span in m. For continuous construction, L is the larger of the span under consideration or of an adjacent span.

3.2.2 Resonant response

Wyatt (1989) also presented a method for assessing the response to the harmonic resonant component of walking for floors with fundamental frequency less than 7 Hz. The response factor, R, is:

$$R = 68000C/MS\ L_{eff}\ D \quad \text{.......} \quad [10]$$

where C = the fourier component factor = 0.4 if f_o is between 3 and 4 Hz,
= $1.4-0.25f_o$ if f_o is between 4 and 4.8 Hz,
= 0.2 if f_o is greater than 4.8 Hz;

S and L_{eff} are the effective floor width and span (in m) respectively. The way of estimating these values using Wyatt's Method is shown in Table 6.

The limiting response factor (R), was derived in conjunction with BS 6472 (British Standards Institution, 1984). R should not exceed 4, 8 and 12 for a "special office", "general office" and "busy office", respectively.

Allen (1990) developed an expression for a design that counters against harmonic resonance problems resulting from walking:

$$DW > R_1 \quad _nP/(a_m /g) \quad \text{..........} \quad [11]$$

where
- D = Damping
- W = weight of floor (kN);
- P = weight of a person = 0.7 kN;
- a_m = limiting acceleration, which Allen obtained from ISO 2631 (International Standards Organisation (ISO), 1989);
- R_1 = 0.5. This reduction factor takes into consideration two elements: that full steady-state resonance is not achieved, and it also accounts for a person walking along a beam rather than just moving up and down at midspan;
- $_n$ = dynamic load factor of the n harmonic.

The dynamic load factors of walking are given as:
= 0.5 for the first harmonic frequency of 1.5-2.5 Hz;
= 0.2 for the second harmonic frequency of 3.5-5.5 Hz;
= 0.1 for the third harmonic frequency of 5-7 Hz;
= 0.05 for the fourth harmonic frequency of 7-10 Hz.

The minimum values of DW recommended for satisfactory performance of the floor are 28, 14, 7, and 3.5 for the first, second, third and fourth harmonics of walking respectively.

The width of a floor on rigid supports is given by Allen (1990),

$$B_3 = 2L4\ (D_y/D_x) \leq L \quad \text{..........} \quad [12]$$

where D_y and D_x are the flexural rigidity per unit width perpendicular to the span and along the beam directions.

Table 3 summarises application of these design methods using calculated or estimated values. The in situ slab and Unispan floor were not tabulated because they were outside the scope of the design methods. The 6.4 m span precast rib floor which has a span less than the proposed minimum applicable to equation 5, was included for comparison with the others since the fundamental frequency was determined to be within the critical range outside the span limitation of equation 5.

Appendix B shows the application of the design methods to the 8.4 m span precast rib floor, and Appendix C shows the design methods application to the composite steel-concrete floor, using calculated or estimated values.

4.0 DISCUSSION

4.1 Frequency

There was no significant difference between the measured fundamental frequency of floors obtained from the heel drop, hammer impact, or standard heel drop (within 1.5 Hz, Table 1).

Calculated fundamental frequency of the beam in the in situ slab floor with smaller secondary beams, was smaller than the fundamental frequency of the slab and governed the response of the overall floor system.

Fundamental frequency of the Unispan floor 4.55x5.30 m was calculated assuming simple supports on four sides. This was because a third side was supported by a reinforced concrete shear wall. The calculated frequency using this assumption was very close to that measured.

The measured frequency of the Hibond composite steel-concrete floor was closer to that of the secondary beam than to either the frequency of the primary girder or two-way action (Table 2). This indicates that floor response is governed by the secondary beam. Non-composite action between the primary girder and the slab was assumed in the calculation; this turned out to be very conservative. Clifton (1989) recommends that short lengths of secondary beam section can be placed on top of the primary girder and connected to both the slab and the girder to achieve composite action. When composite action of the primary girder is achieved, it is unlikely that the primary girder would govern the response of the floor system, as the frequency of the primary girder would be increased considerably. Similarly, response of the two-way action could be ignored because the frequency of the two-way system (which is dependent on the frequency of the primary girder) would be increased correspondingly also. In perimeter girders, the frequency would be considerably greater through contribution of a spandrel/cladding system, particularly where there is full-height partition beneath or above the girder.

Field tests indicated that actual fundamental frequency of each of the floors tested was significantly higher than those calculated, with the exception of the in situ slab and Unispan floors where two-way action has been assumed in the calculations (Table 2). It was initially thought that that the fundamental frequency of each of the floors obtained using the FFT analysis technique was the second mode response of the floors. There have been instances overseas where this has been observed, particularly for very stiff floors. However, in this study there are two reasons why this assumption is not correct. Firstly , a less accurate method of determining the fundamental frequency of each signal was obtained by calculating the number of cycles divided by the time elapsed for each signal. This method yielded results similar to that obtained from the FFT. Secondly, similar field tests that assumed simple support conditions carried by other workers on New Zealand floors (Wood, 1990; Burns and Yong, 1990) gave similar results to those observed here. The higher actual measured frequency compared to the calculated values was attributed to the actual floor stiffness being significantly greater than that assumed in the calculation.

In the calculations (except for those for the in situ slab and Unispan floors) it was assumed that floors possessed only one-way action. In reality, each precast unit is tied to its adjacent unit through the in situ concrete topping. Depending on bay width and thickness of the concrete topping, some two-way action will occur. Theoretically, the fundamental frequency of continuous beams with equal spans is equal to the fundamental frequency of a simply supported beam (Rogers, 1959). In practice, a continuous beam, regardless of whether the spans are equal or not, will not exhibit simply supported conditions. The floor or beam actually has a more rigid support condition and this will result in a higher fundamental frequency than suggested by theory. We recommend that support conditions be considered thoroughly when calculating the fundamental frequency of beams/floors in one-way systems. Full-fixity at the support should be considered only where appropriate. The fully built-in condition shall be assumed only when the designer is confident that it can be achieved in the construction. It is unlikely that the fully built-in condition can be achieved at end spans of a building supported by perimeter spandrel beams. This is because spandrel beams (particularly of steel framed construction) would not possess sufficient torsional stiffness.

Stiffness contributions from non-structural components, such as full-height partitions and facade cladding, were also not included in the calculations. A further parameter influencing the stiffness of the floors, which indirectly influenced the calculated frequencies of concrete floors is the concrete modulus of elasticity (E). A 20% allowance for increase in the concrete modulus of elasticity of in-service condition (Bull, 1990) in relation to the 28-day value was assumed in the calculations. However, the derived E value of 30340 MPa is still low compared to Wyatt's (1989) recommended value of 38000 MPa for normal weight concrete. Moreover, the E value used in the frequency calculation assumed concrete with a 28-day compression strength of 25 MPa. In reality, the precast concrete floor units would have been constructed using concrete with a 28-day compression strength of about 40 MPa. Thus it seems reasonable to use the area-weighted concrete modulus of elasticity for calculating fundamental frequency of precast concrete floors. This increases the E value by 30% (fundamental frequency by 14%) for concrete floors, if the area of precast concrete floor element is equal to the area of the topping (i.e., (40+25)/2 divided by 25).

4.2 Peak Acceleration and Peak Displacement

For the floors that were within the scope of equation [5], the measured peak accelerations of the heel drop impact were generally different from those calculated (Table 2). The variations were thought to be a result of the following opposing factors.

Firstly, as well as the fundamental mode, the acceleration signals contained components of higher modes of vibration. This produced a higher peak acceleration than if the fundamental mode alone was present. Low-pass filtering of the signal, at a frequency of twice the calculated fundamental frequency, would reduce but not remove such effects completely. Secondly, although the person producing the heel drop weighed

78 kg, the resulting peak force and impulse were substantially smaller than those of the simulated standard heel drop.

Although there were some variations between peak accelerations of the floors obtained from the human heel drop and the "standard heel drop" impactor, these variations were not significant. Both these types of impacts could therefore be considered to be useful test methods. On the other hand, peak accelerations of the floors obtained from using the impulse hammer ranged from 3.8 to 19.7% g. This was because the impulse hammer could not impart consistent force levels to the floor, reducing its potential for obtaining useful results. In some instances, the displacement obtained from the hammer impact was less than the heel drop impact, even though the acceleration produced by the hammer was significantly greater than that of heel drop impact. This could be because the floor response from the impulse hammer did not represent a sinusoidal function.

The peak displacements shown in Table 2 were not measured directly, but rather determined through manipulation of the acceleration response signals in the frequency domain. Generally, however, the measured peak displacements were small, and of the same order of magnitude as those calculated.

4.3 Damping

In theory, damping values estimated using the "half-power bandwidth" method in the frequency domain and the "logarithmic decay" method in the time domain should be the same. However, the measured values were generally quite different (Table 1). The main difficulty in estimating floor damping using the "logarithmic decay" method was that the signals did not show a smooth exponential decay. The increase in acceleration (Figures 2c, 2d) after the second and third peaks was due to the simultaneously occurring higher modes of floor vibration. Secondly, the floor damping varied, depending on which cycles of the signal were considered. Thus, the damping estimated in the time domain has not been considered and the damping obtained in the frequency domain has been taken as the representative floor damping.

Generally, the damping measured from the hammer impact excitation was different to that derived from the heel drop and was not used. The variation was attributed to the additional weight of the person conducting the impact, the weight of the hammer and the inconsistency in force levels imparted by the hammer.

Only damping from heel drop impacts conducted adjacent to the accelerometer have been included in the following analysis. Damping of unpartitioned floors ranged from 3.7-8.2% critical, with an average of 6.3% and a standard deviation of 1.9%. Damping of the Unispan floor system (12.2% and 12.8%) has not been considered because although test areas were unpartitioned, areas adjacent to the test areas were partitioned, and could have increased the damping available.

Damping of partitioned floors ranged from 7.7-17.1% critical, with an average of 12.6% and a standard deviation of 3.3%.

Whether the floor was partitioned or not, and where the test was conducted in relation to the partitions affected the damping. There were insufficient data to determine if damping is affected by the type of floor system. Table 1 shows that the floors which were partitioned had higher damping than those that were not (3.7% compared to 16.4% critical for the 6.40 m span rib floor; average of 6.7% compared to 14.9% critical for the composite steel-concrete floor). The measured damping was significantly higher where the test location was adjacent to a partition (within a distance of 1 m), than a test location remote from a partition. This is supported by Murray (1989), who observed that partitions must be attached to the floor system and spaced closer than every five secondary floor beams for floor systems to achieve his recommended damping. For example, the higher damping of location 2 (compared to location 1) of the 8.0 m span precast rib floor was because it was near the corner of a partitioned room (16.8% cf. 11.0%). Location 1 was at midspan of the floor and parallel to one partition only. The results of Burns and Yong (1990) indicated an average damping of 11.6% (cf. 11.0%) for location 1 compared to 13.1% (cf. 16.8%) for location 2. Burns and Yong obtained damping values using the decay method.

The wide range of measured damping in both unpartitioned and partitioned floors also indicates that damping cannot be easily quantified. Since damping varied with floor location, in situ tests to measure floor damping should be carried out in at least four locations distributed throughout the floor. The average of these damping values would provide a better representation of overall damping than values from one or two test locations in each floor. This is why these tests do not give actual damping of the floors, but instead provide more of a guideline.

If indeed the viscous component of damping was considered to be 50% of the measured floor damping when considering resonant response (Allen, 1990), a finished floor with mechanical fittings and a ceiling would have a damping of 0.5 x 6.3 = 3.1% critical. Similarly, the viscous damping would be 0.5 x 12.6% = 6.3% for a partitioned floor. These values are slightly greater than the 3.0% and 4.5% damping values recommended at the corresponding stages of construction (Allen, 1990; Wyatt, 1989). It is recommended that 3.0% and 6.0% be adopted for the above two stages of construction for New Zealand heavy floors when considering resonant response. These damping values are also recommended when considering transient response, not only because this avoids confusion, but also because damping is location dependant.

The 6.0% damping for partitioned floor applies only to partitions that extend to at least ceiling height, are fixed to at least three locations (e.g. to the floor and two return walls), and are spaced no more than 4 m apart. The partition spacing of 4 m was derived from the test floors where it was observed that generally this maximum spacing was used. For partition floors that do not fulfil these requirements, a damping of 4.5% should be assumed.

4.4 Different Design Methods

4.4.1 Using calculated/estimated values

Both resonant response design methods (Wyatt, 1989; Allen, 1990) evaluated the composite steel-concrete floor as being "Unsatisfactory" (Appendix

C.2) using estimated values. However, harmonic resonant response can be ignored in New Zealand floors because of their inherant stiffness. This leads to a higher fundamental frequency, negating the "Unsatisfactory" response predicted by the Allen method. Similarly, because the fourier component factor C (see eqn. 10) decreases considerably with higher frequency, an "Unsatisfactory" response as predicted by Wyatt's method can be ignored as well.

Applying the impulsive response design methods (CSA, 1984; Murray, 1989) with an estimated damping of 6.0% in the partitioned floors, the only floor that was predicted to be "Unsatisfactory" was the Stresscrete 6.4 m span precast rib floor (Table 3). The observed rating for this floor, the 8.4 m span precast rib, and the composite steel-concrete floors was "Noticeable". Because a "Noticeable" rating could mean either "Borderline" or "Unsatisfactory", it is postulated that the 6.4 m precast rib floor is actually an "Unsatisfactory" floor, whereas the others lie in the "Borderline" region.

Murray's criteria appeared to be more severe than the CSA method (i.e., required greater damping) for the floors considered (Table 3). The correlation between the dynamic characteristics of the floors measured in the field and those predicted by either method was often poor. It is recommended that Murray's criteria be applied to ribbed floor systems, these being the elements upon which the criteria were derived by the researcher within his US studies. The CSA criteria are generally less conservative in their demand for minimum damping levels, but have a wider scope of application. Both design methods are highly dependent on the fundamental frequency of the floors, a parameter that is difficult to predict for New Zealand floors. Floors with a higher fundamental frequency also have a higher calculated peak acceleration (equation 5). Therefore higher levels of damping are needed to avoid vibration problems using the CSA method. Murray's method also requires a higher damping ratio for shorter span floors with a higher fundamental frequency.

Murray observed that contrary to popular beliefs, long span floors (with lower fundamental frequency) are less susceptible to transient vibration problems. Whereas the 6.40 m span precast rib floor required 5.7% and 8.1% critical damping, the 10.05 m span with a deeper rib size, required only 3.2% and 5.3% damping respectively, using the CSA and Murray's method (Table 3). Moreover, the longer span floor with a greater contributing weight was also less susceptible to harmonic resonance.

4.4.2 Using measured values

The plot of measured peak acceleration against frequency is shown in Figure 5. The acceptable criteria of Murray and of CSA have been included. Murray's values for 4% and 6% damping were derived by assigning a value of 4% or 6% to the left hand side of equation [8], and solving for the displacement amplitude A_o. Assuming a sinusoidal response, the acceptable peak acceleration (a_o), was derived. Figure 5 indicates that the floors tested would generally perform satisfactorily under both the CSA and Murray criteria. Fundamental frequency appears to be the main parameter affecting dynamic performance of the floors. Floors with frequency below about 10 Hz were rated as "Noticeable" under walking vibrations; those greater than 10 Hz were "Satisfactory". Available floor damping did not appear to influence significantly the performance of the floors tested.

Table 4 shows how the design methods compared. Only 50% of the damping measured has been used in the evaluation of the resonant response. The table indicates that in general, the Murray criterion is more severe than the CSA criterion.

The 8.40 m and the 10.05 m span precast rib floor which were partitioned, were predicted to be "Unsatisfactory" by the Murray method. The former was in agreement with the "Noticeable" rating observed (see Table 1). However a rating of "Satisfactory" was observed for the 10.05 m span. The discrepancy may be due to the conservatism of the Murray criterion at the high measured floor fundamental frequency of 11.7 Hz.

The unpartitioned 6.4 m span precast rib and the composite steel-concrete floors were calculated as being "Unsatisfactory" according to the Murray and CSA criteria, and could be considered to be in agreement with the observed floor responses. On the other hand, the 6.40 m and the composite steel-concrete partitioned floors were predicted to be "Satisfactory" even though the floors were rated as "Noticeable" in field tests. These discrepancies could be a result of the definition of the "Noticeable" rating, i.e., the floors are probably "Borderline".

Because New Zealand has traditionally built structures using structural concrete members which are substantially stiffer than steel (both floor and structural framing), a slight reduction in floor stiffness and floor weight could cause floors to be rated as "Noticeable" with regard to people's expectations. Because the design methods were derived in North America from tests on composite-steel concrete floors (particularly the Murray method), the occupants there may have been "conditioned" to those types of buildings and therefore have a greater tolerance to vibrations compared to New Zealanders. Possibly, floors that are "Satisfactory" in North America using the Murray and CSA methods may not be rated as "Satisfactory" here.

A further reason could be that the damping measured and utilised in the design methods to predict the floor response (the Stresscrete 6.4 m precast rib and the composite steel-concrete floors) was higher than the average damping present (i.e., the tests were conducted at highly-damped areas). This indirectly supports the recommendation in section 4.3, of the need to obtain and average damping values from at least four locations.

Generally, the harmonic resonant response was classified as "Satisfactory" using the Allen method when the measured fundamental frequency of the floors were used. These frequencies were significantly higher than those calculated and shifted the response from that of the second (the product of damping and floor weight, DW = 14), to the fourth harmonic (DW = 3.5). Similarly, the Wyatt method also evaluated the partitioned composite steel-concrete floor as "Satisfactory" when using the measured values. Because New Zealand floors exhibit a higher degree of stiffness resulting in higher fundamental frequencies, problems resulting from harmonic resonant response are unlikely.

5.0 FUTURE WORK

Field testing suggested that New Zealanders may have a higher expectation of the floor performance with regard to vibrations than overseas building occupants. This could be because New Zealand structures have been built traditionally with substantial stiffness and mass using structural concrete members. This issue however needs to be studied further and it is recommended that "controlled studies" be made to investigate the acceptable acceleration limits for New Zealand building occupants. Respondents for such an investigation should be drawn from a wide cross section of New Zealand society. Such an investigation should be conducted on light-weight timber floors because respondents would be able to perceive vibrations more easily than in heavy, stiffer floors. Testing of light-weight floors also has the advantage that the relevant dynamic parameters could be measured to verify the design method recommended for use in the previous phase of this project. The parameters to be studied may include:

(a) whether the respondent was sitting, standing or walking;

(b) the span, width and sectional properties of the joists;

(c) respondents response to walking, running, jumping or heel drop excitations;

(d) types of joists, e.g., timber, light-weight steel, composite wood products;

(e) the effects of solid blocking between joists, the presence of a ceiling and the presence of partitions on the acceptability limits for floor damping and acceleration.

Accelerometers with a higher degree of sensitivity (such as 10˙ Volts/g) should however, be used to record floor response from excitations such as walking.

Further field testing on heavy-weight floors such as those described in this report, should be conducted to obtain an information "database". Clarification of the importance of damping in reducing annoying transient vibrations should be carried out.

The information obtained in both light-weight and heavy-weight floors could then be used as inputs to develop a "finite-element" computer model to predict the performance of floors when subjected to vibrations.

6.0 CONCLUSION AND RECOMMENDATIONS

Heel drop impact, hammer impact and standard heel drop impact tests were conducted on New Zealand heavy floors. Hammer impact did not produce consistent excitations and it is recommended that future testing avoids using this method. The fundamental frequency; peak acceleration; peak displacement, and damping of floors were measured.

The measured fundamental frequency of each of the floors was generally significantly higher than those calculated assuming one-way action. This was attributed to the higher actual stiffness resulting from:

(a) significant two-way slab action dominating, through the thickness of the concrete topping;

(b) stiffness contributions from non-structural components such as full-height partitions and facade cladding;

(c) a higher concrete modulus of elasticity than assumed in the calculation.

It is suggested that to attain a calculated fundamental frequency corresponding to the actual, the floor support conditions need to be thoroughly considered. Full-fixity or two-way action may be possible.

The measured damping of the floor systems using the "logarithmic decay" method was generally quite different from that of the "half-power bandwidth" method. Because the "half-power bandwidth" method was more accurate and consistent in the determination of damping, it is preferred for use. The measured floor damping was affected by where the test location was in relation to the partitions. Measured damping was higher where the test location was adjacent to a partition. Damping values of 3.0 and 6.0% are recommended for use in floors with a ceiling and mechanical fittings, and in partitioned floors, respectively. For partitioned floors to achieve the 6.0% damping, they:

(a) must extend to at least ceiling height;

(b) must be fixed to the floor and two return walls;

(c) must not be spaced more than 4 m apart.

If the above requirements are not met, a damping of 4.5% could be assumed for partitioned floors.

Either of the two impulsive response design criteria considered within this study (CSA, 1984; Murray, 1989) may be used to identify the level of damping required to avoid transient vibration problems with heavy floors in New Zealand. The damping vibration demands required to satisfy Murray's criteria are generally more severe that those stipulated by the CSA method. European requirements are more demanding still. It is important that designers be aware of the limitations of application for both the CSA and the Murray method, and ensure that the floors being considered have parameters which fall within the scope limits defined (usually relating to minimum span, and maximum fundamental response frequency). The field test undertaken in this study indicates that the fundamental frequency of most floors was in excess of the value predicted by calculation. It is thus generally unlikely that resonant response of floor systems will be of concern for New Zealand floors.

Some floors were rated as "Noticeable" (meaning either a "Borderline" or "Unsatisfactory" rating) by the occupants even though the application of the Murray and CSA criteria using both the calculated and measured values predicted a satisfactory response. A posssible explanation was that New Zealanders are more sensitive to the vibration of floor than is reported overseas.

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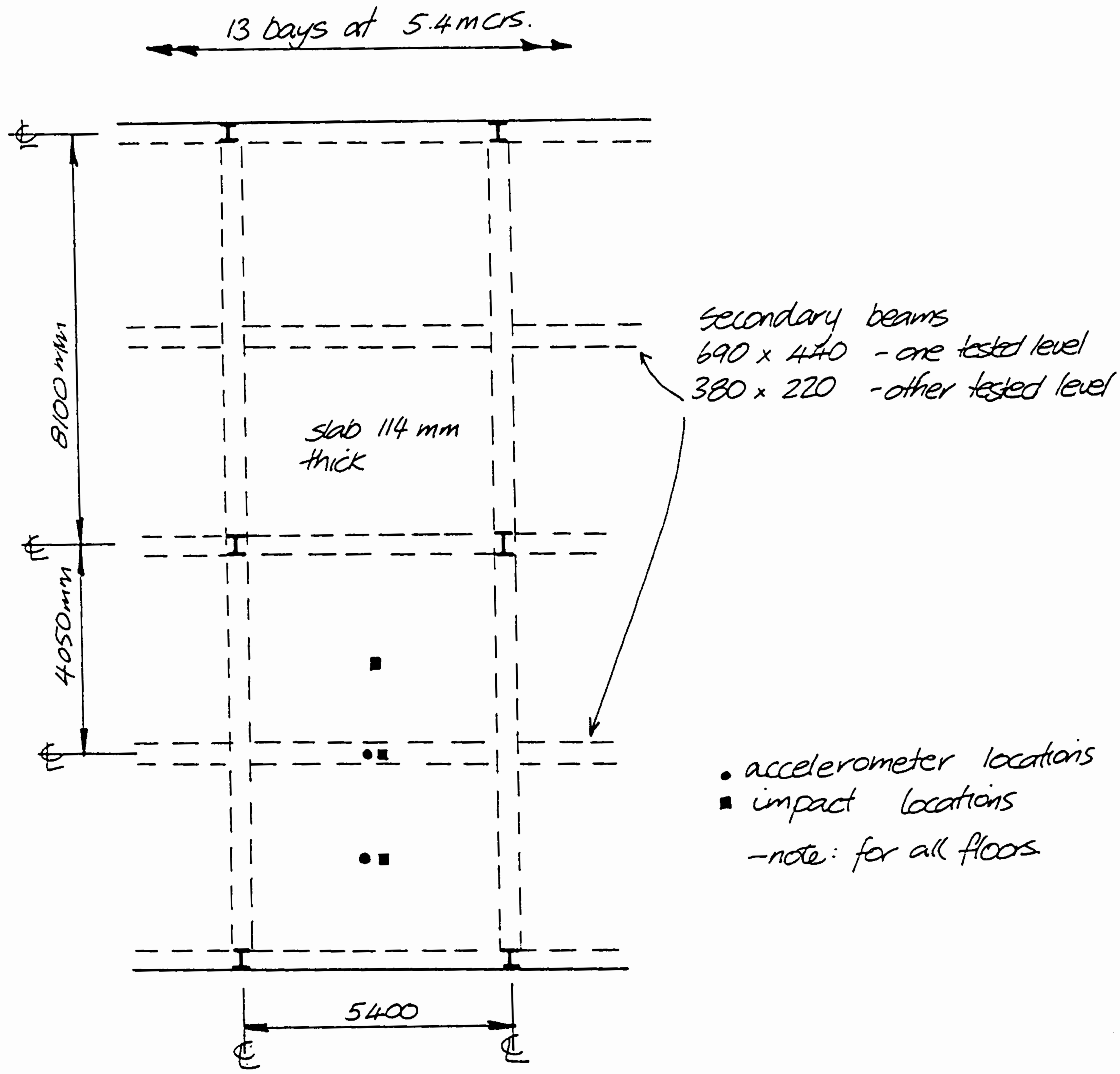
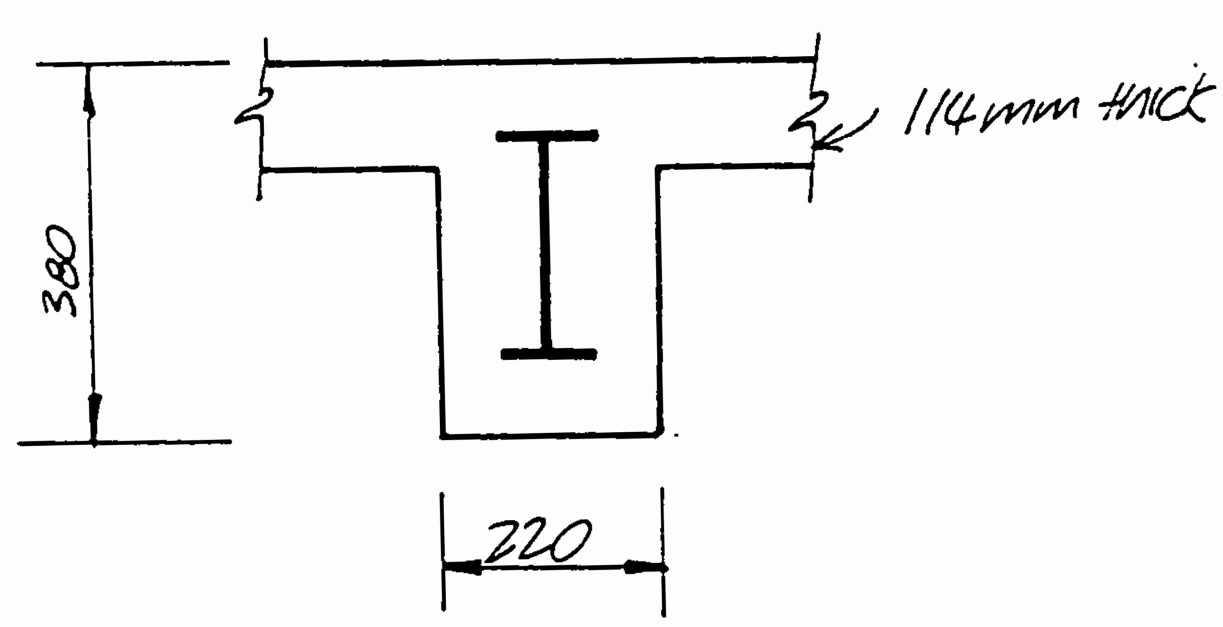


Figure 1a : Typical floor plan of one bay of in-situ floor slab



Section through secondary beam of one level.

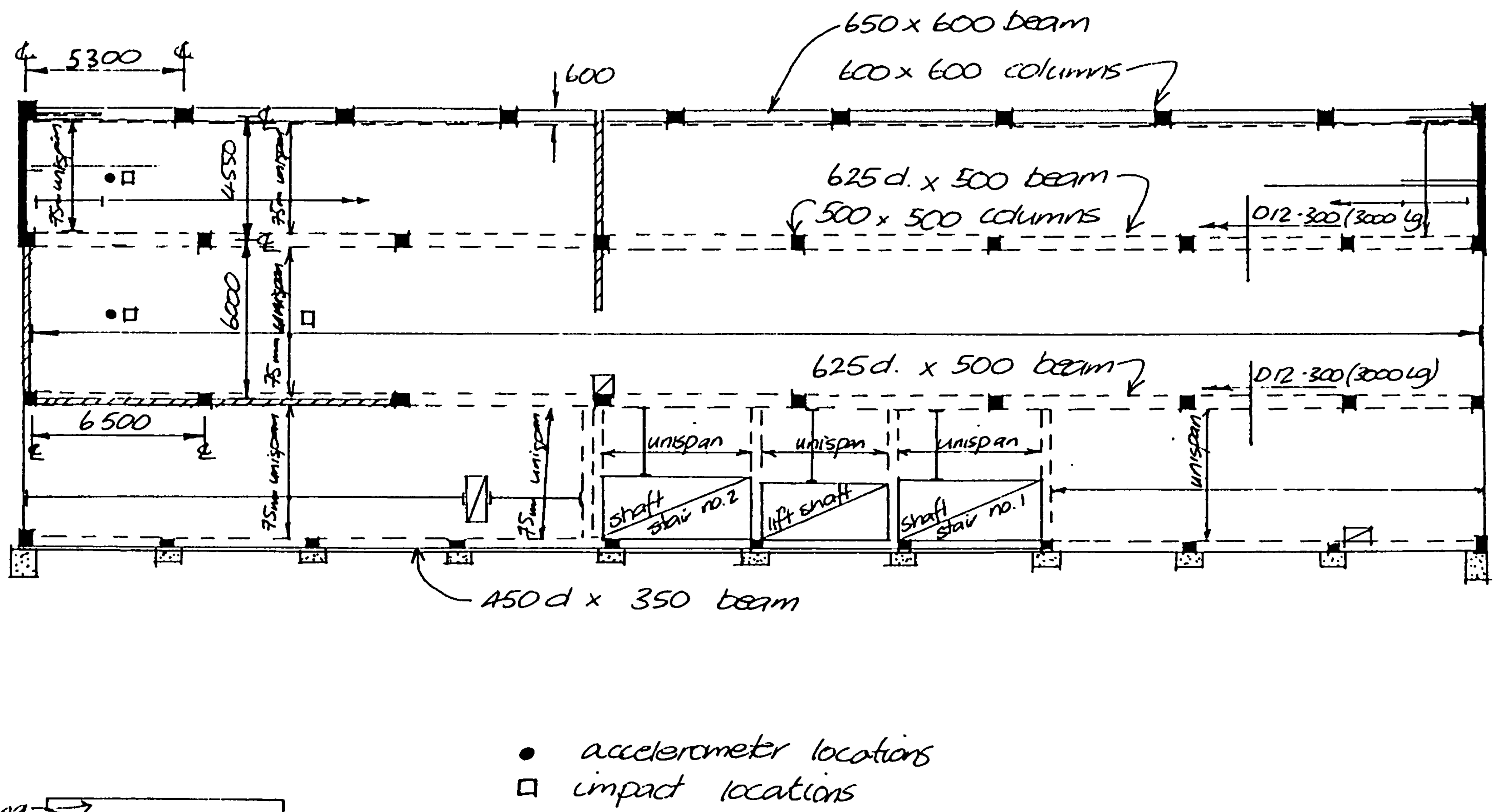
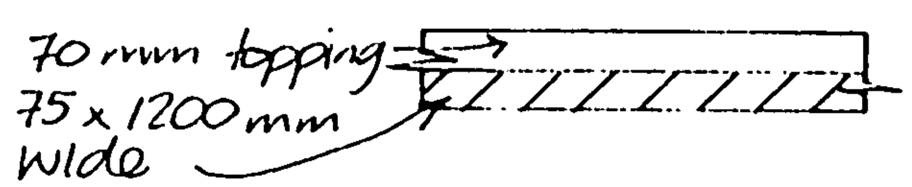


Figure 1b : "Unispan" floor plan



Section through precast "Unispan"

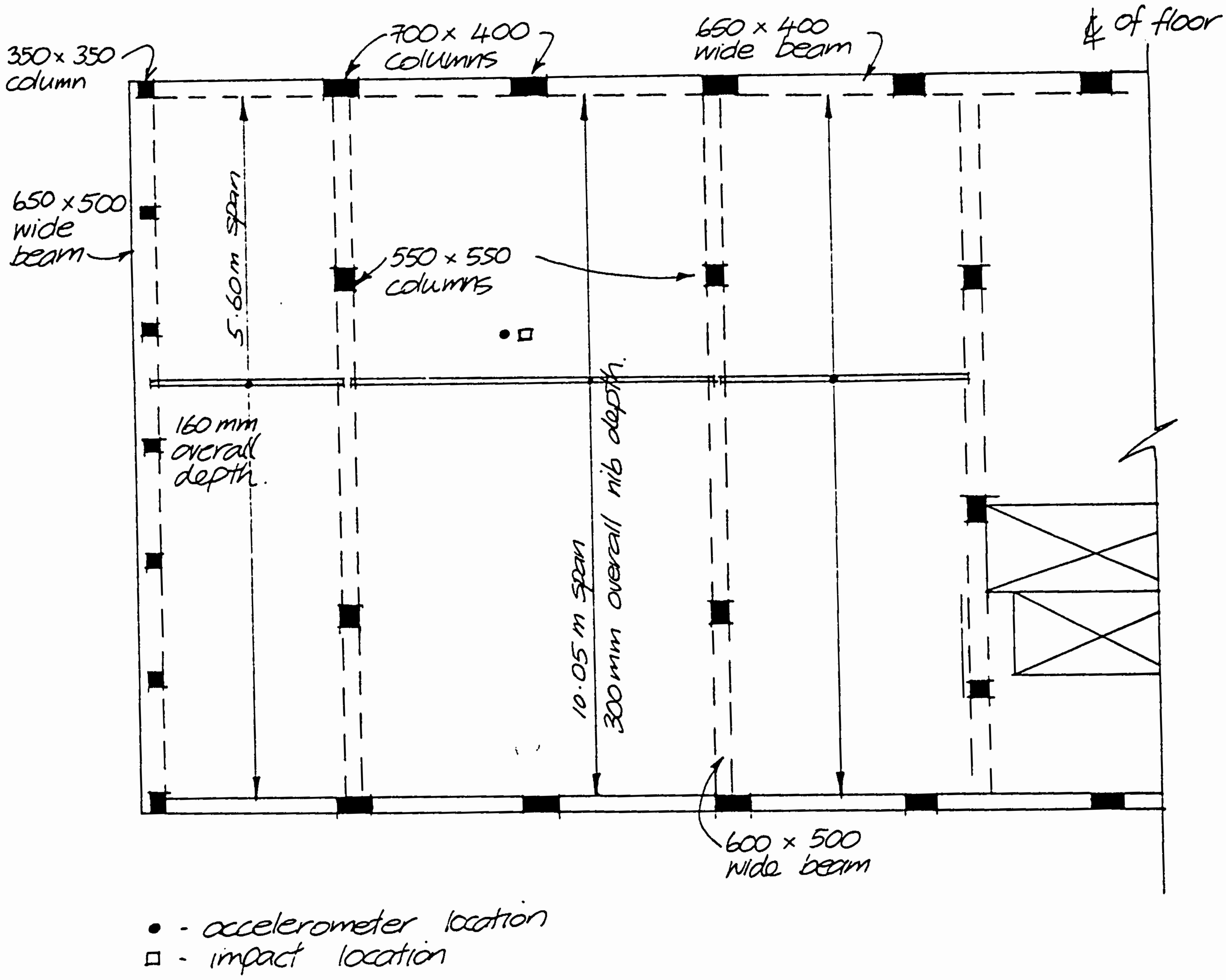


Figure 1c : Floor plan of 10.05 m precast rib floor

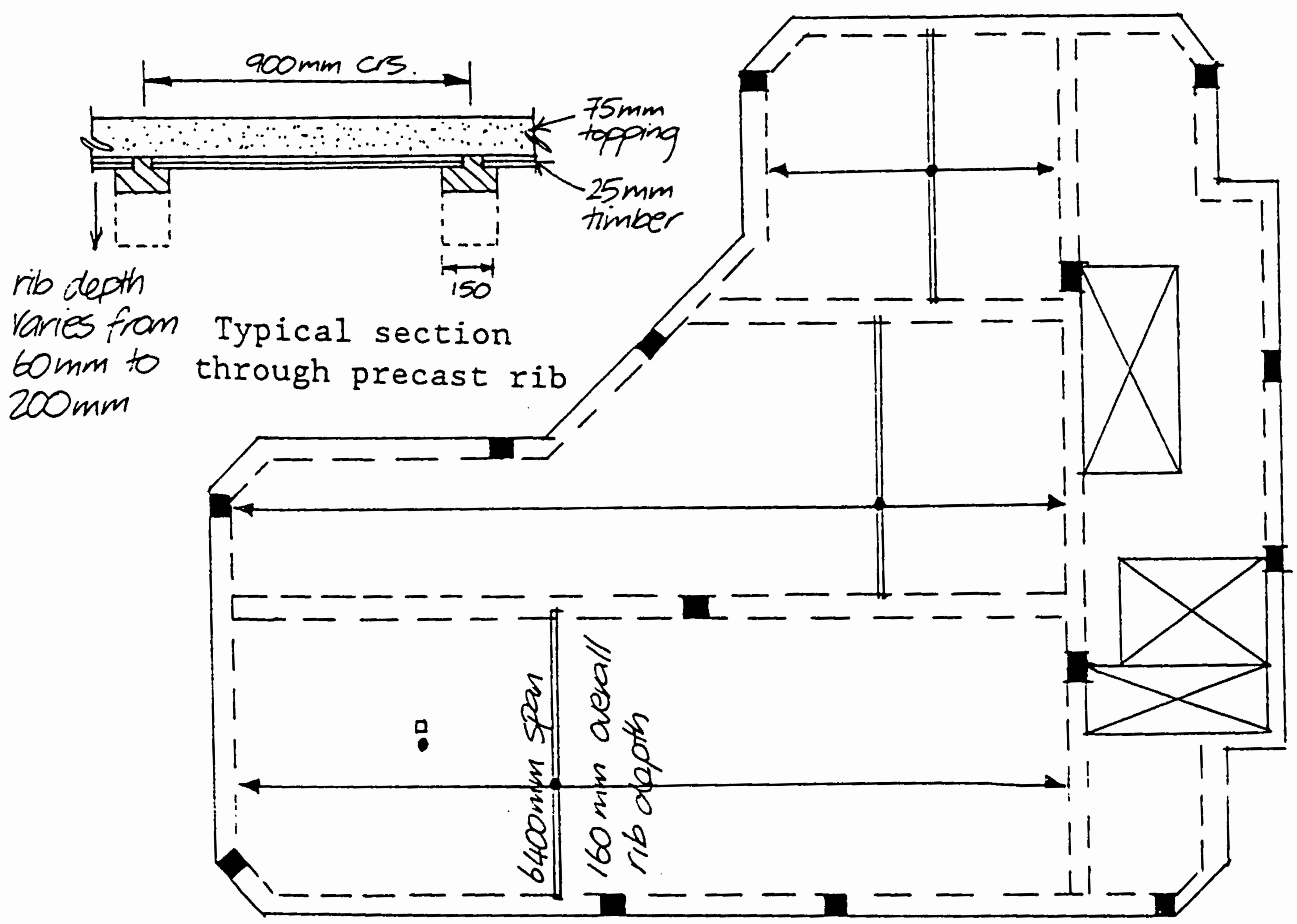


Figure 1d : Floor plan of 6.4 m precast rib floor

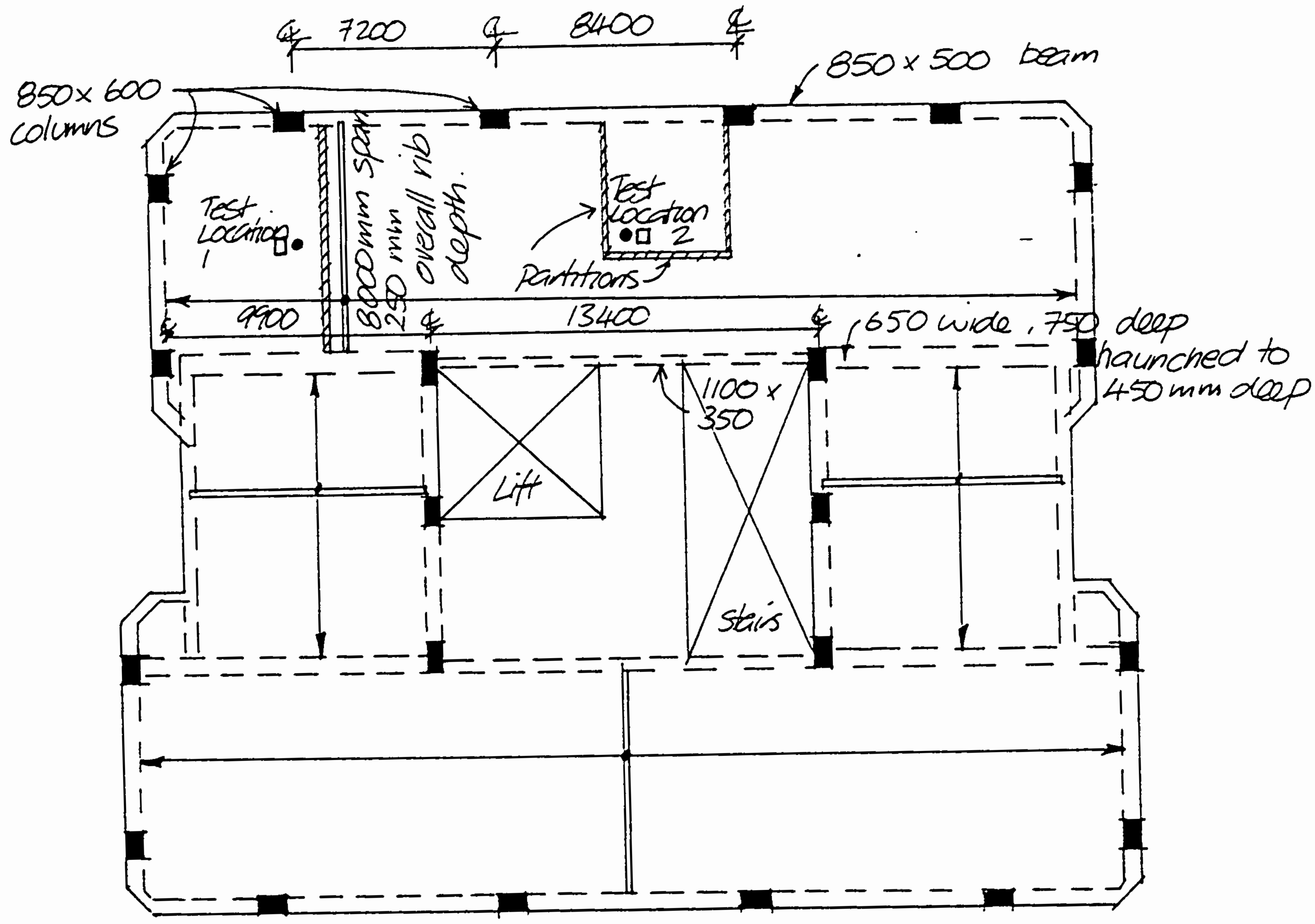


Figure 1e : Floor plan of 8.0 m precast rib floor

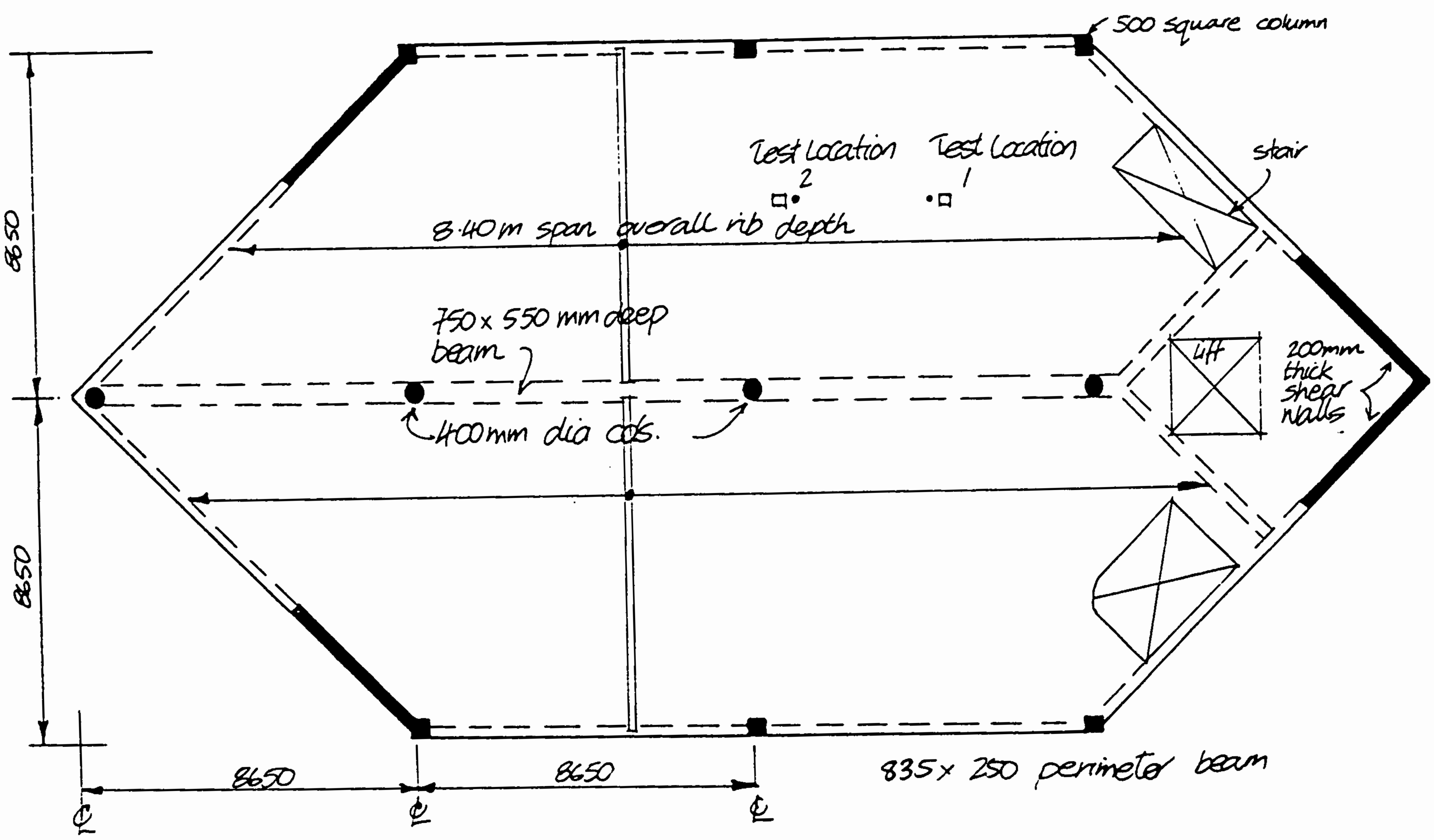
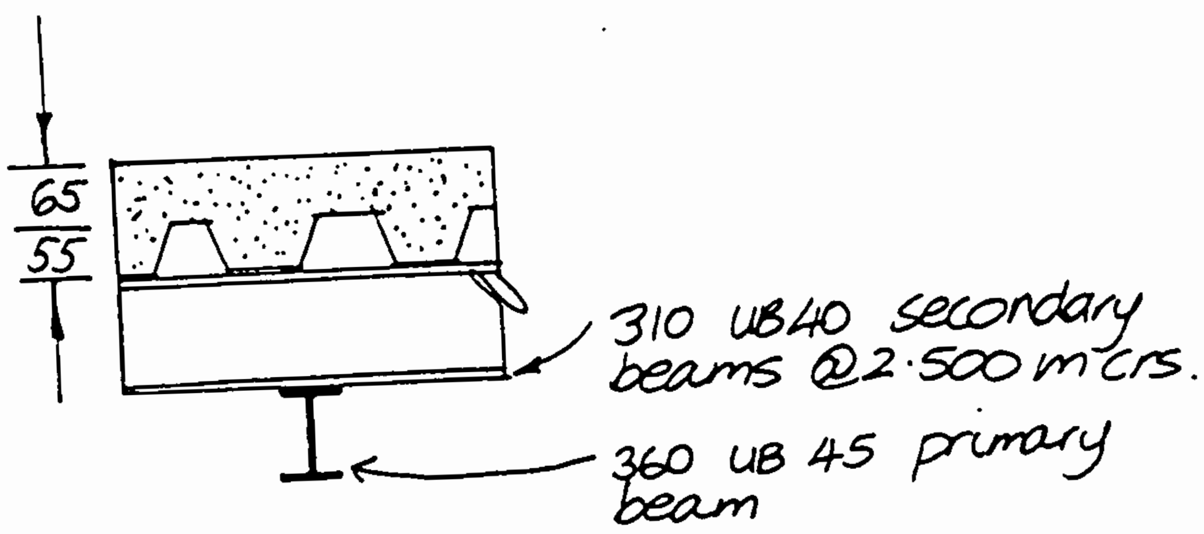


Figure 1f : Floor plan of 8.4 m span precast rib floor



Section A-A

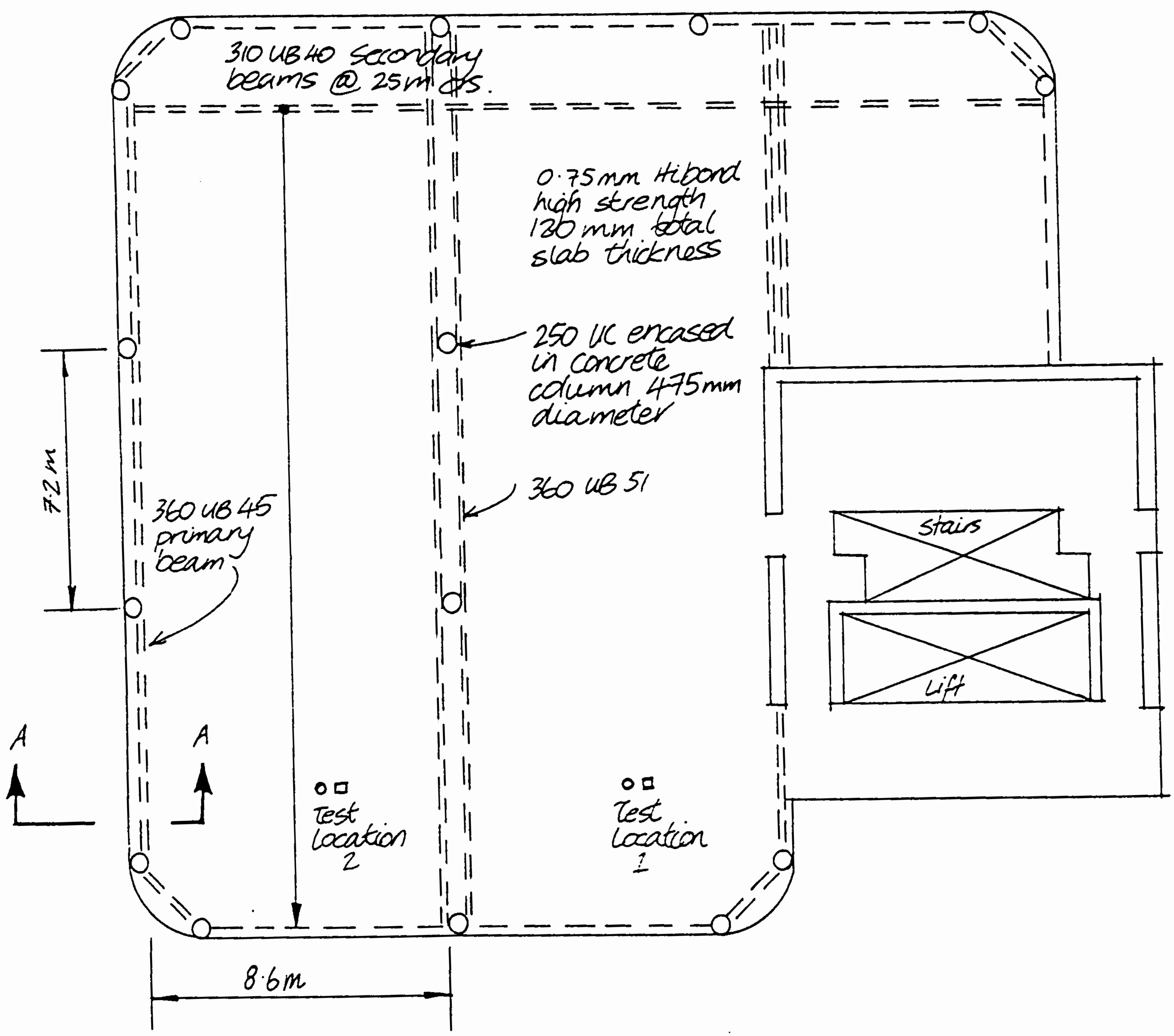


Figure 1g : Floor plan of composite steel-concrete floor

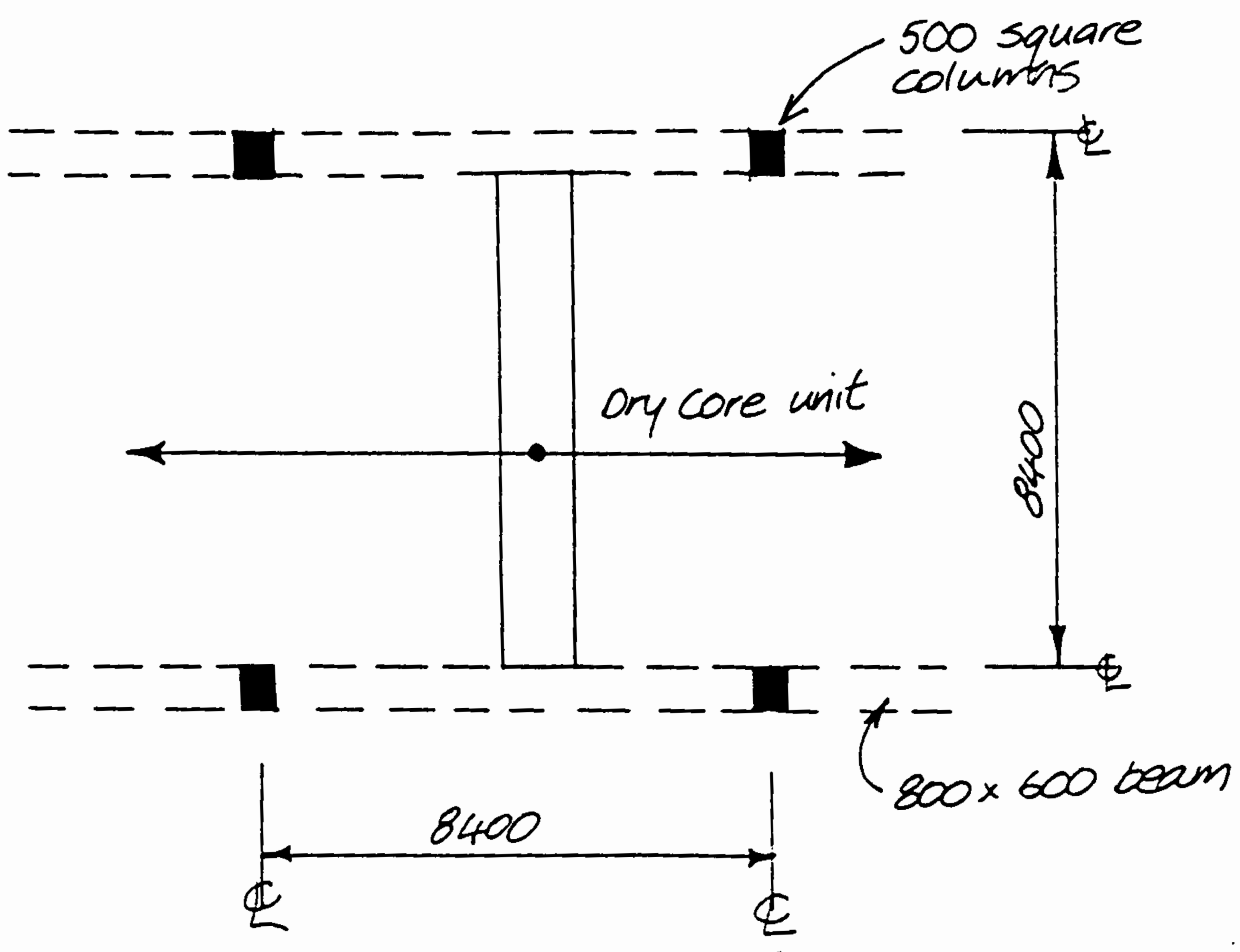
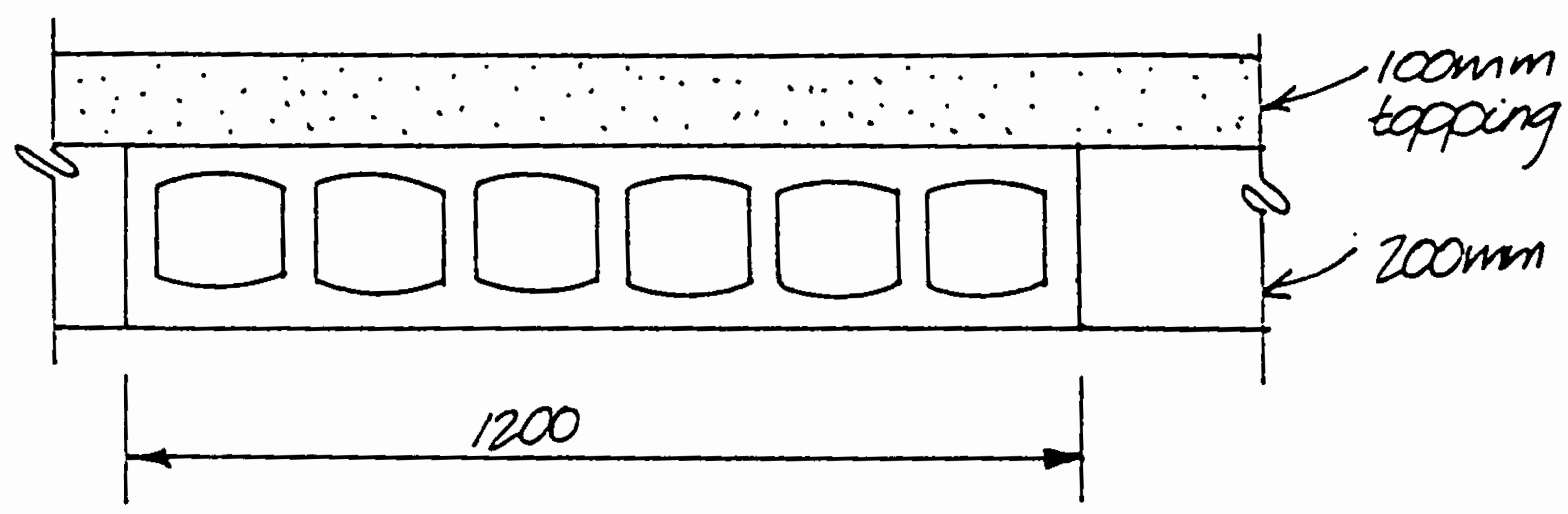
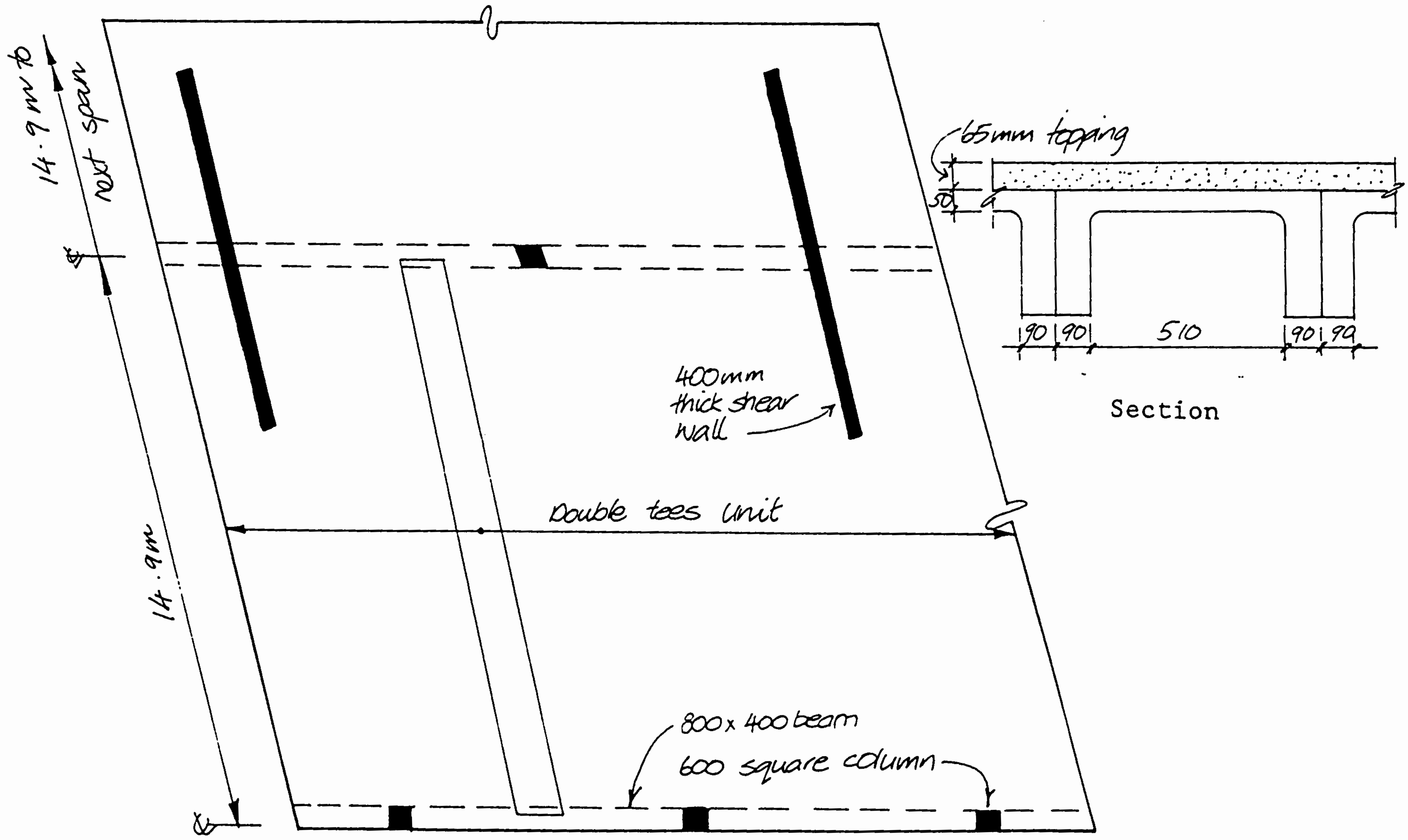


Figure 1h : Section through the precast "Dycore" floor



Section



Section

Figure 1i : Section through the precast double tees (cropped) floor

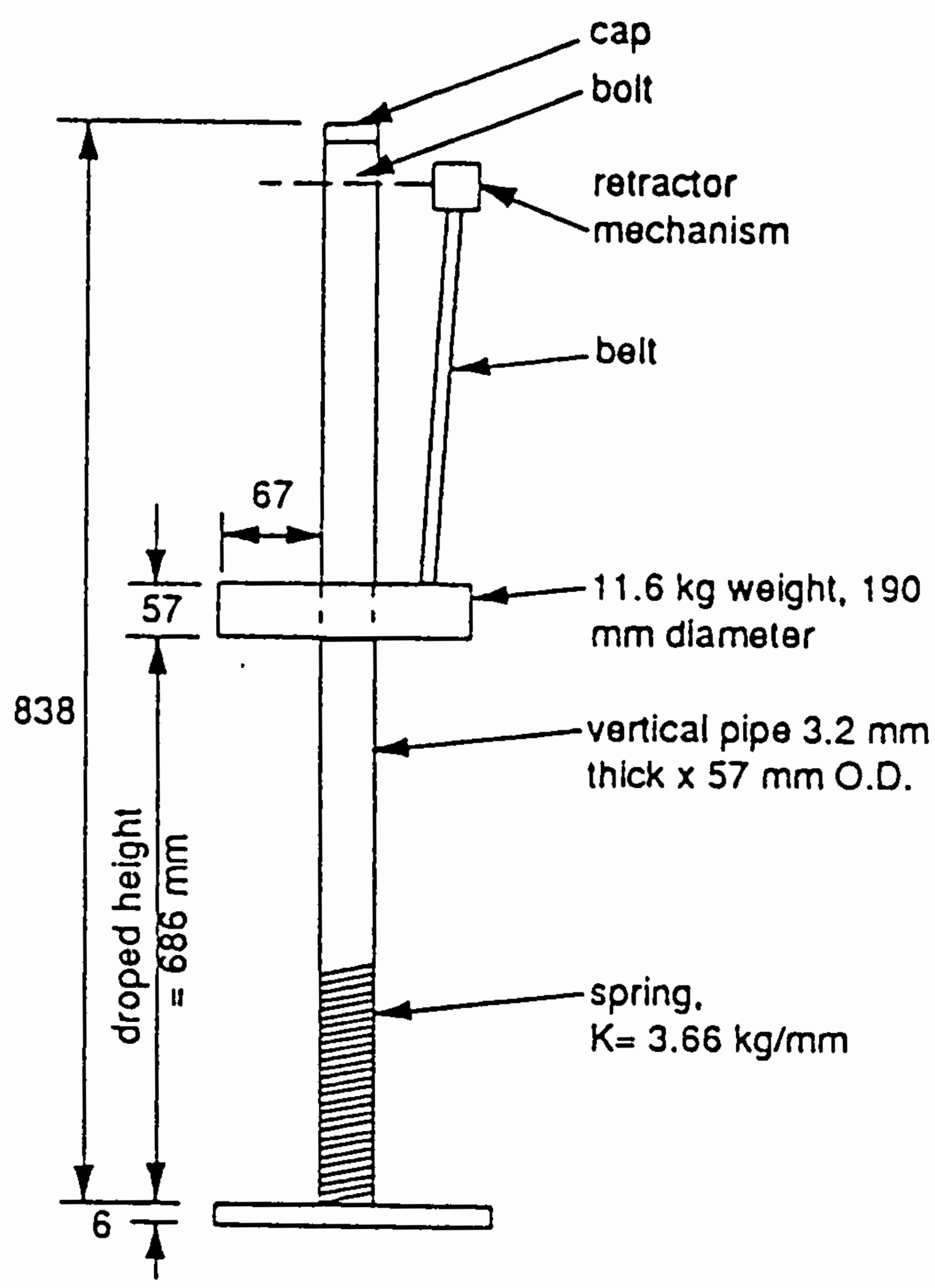


Figure 2a : Standard heel drop impactor from Murray (1990)

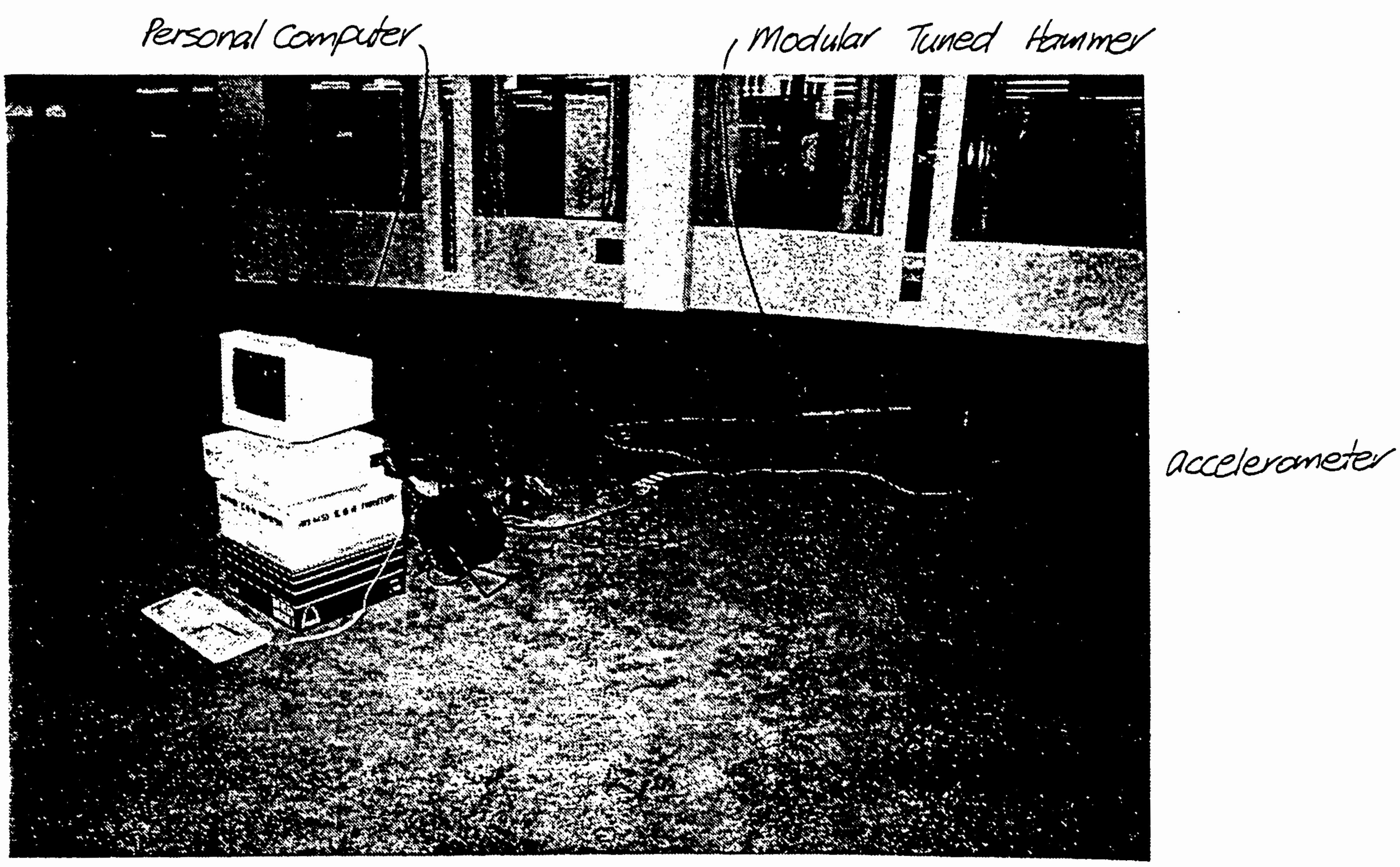


Figure 2b : Equipment for dynamic tests

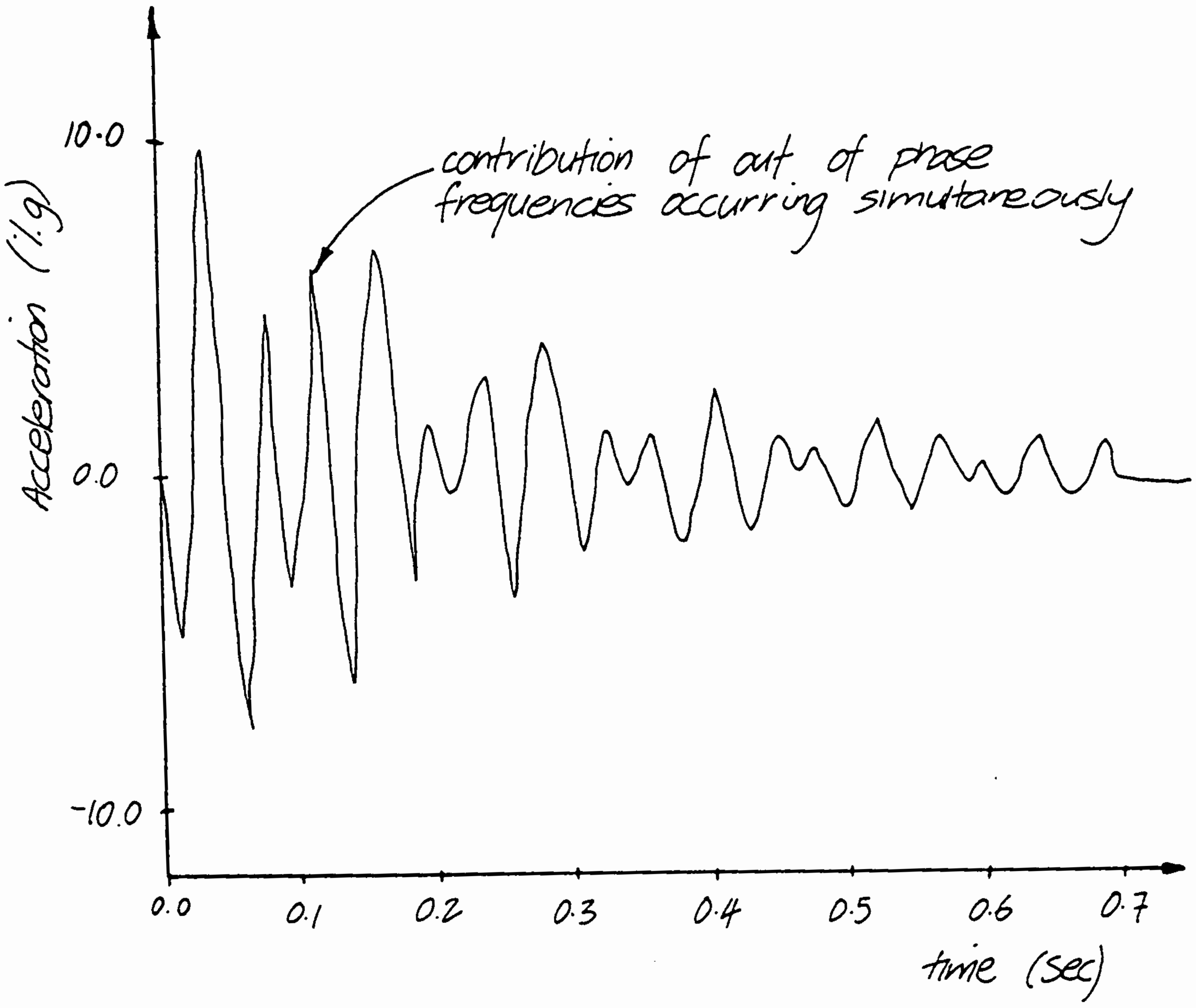


Figure 2c : Acceleration of in-situ slab from heel drop impact

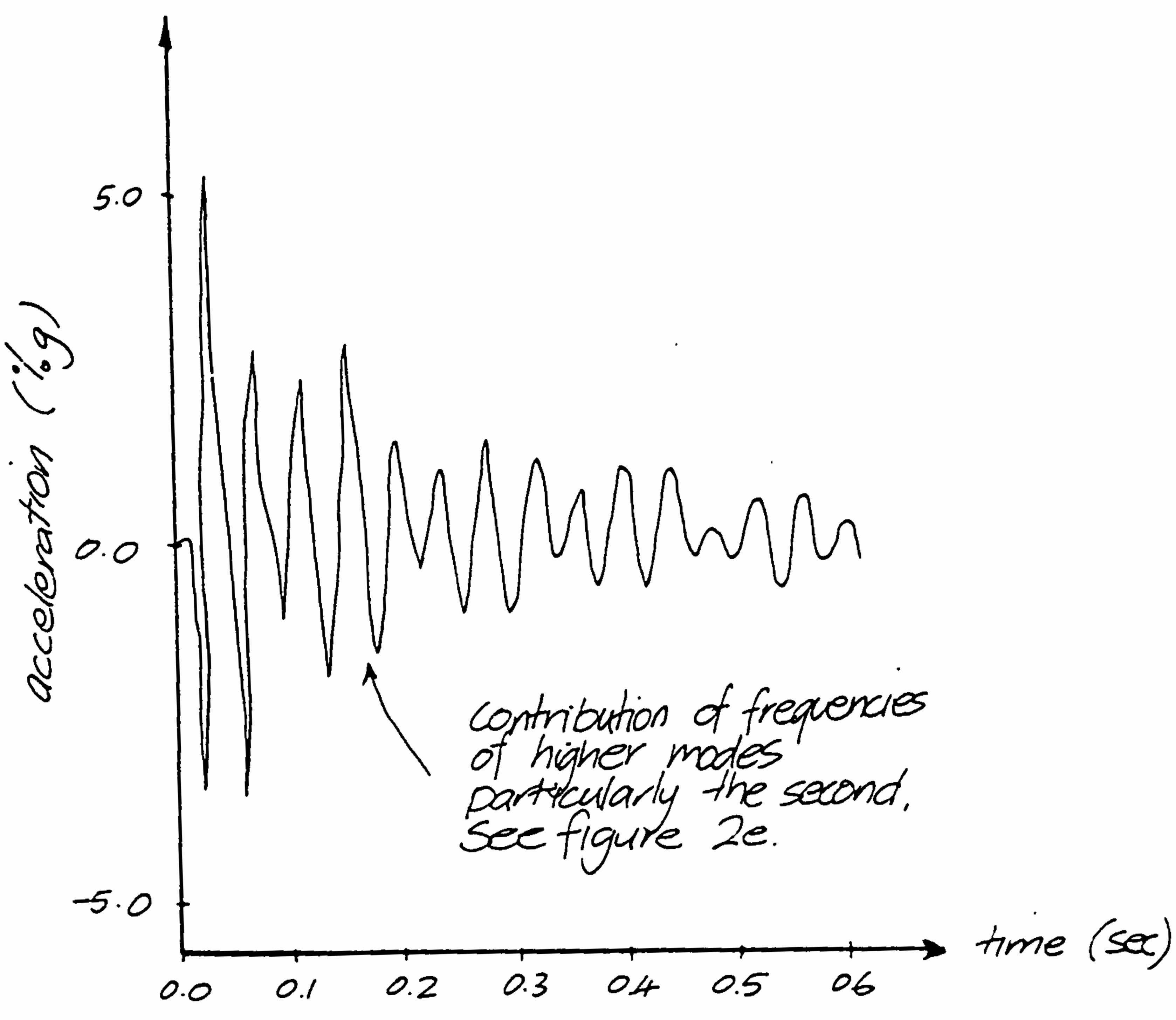


Figure 2d : Acceleration of insitu slab from hammer impact

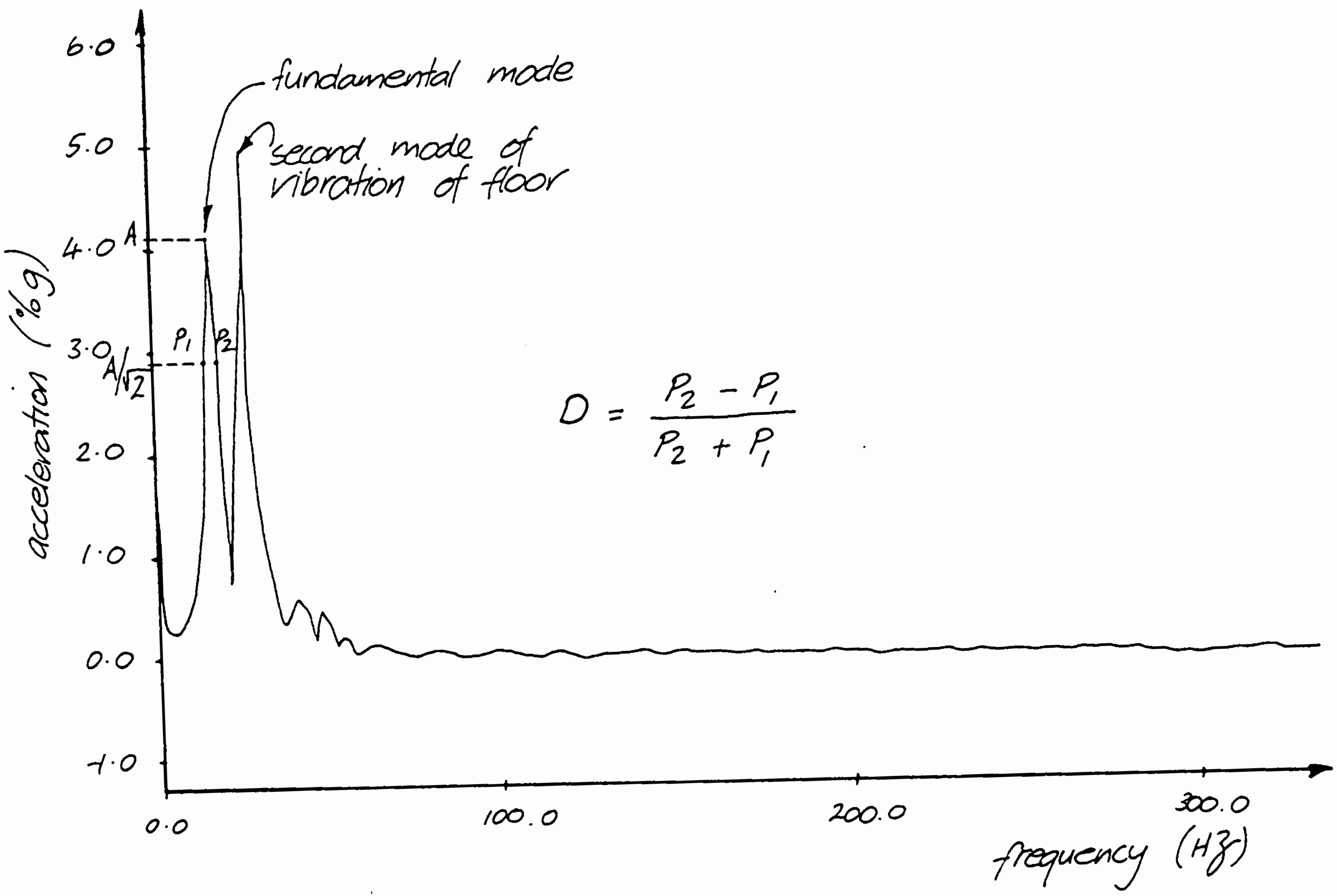


Figure 2e : "FFT" of in-situ slab from heel drop impact

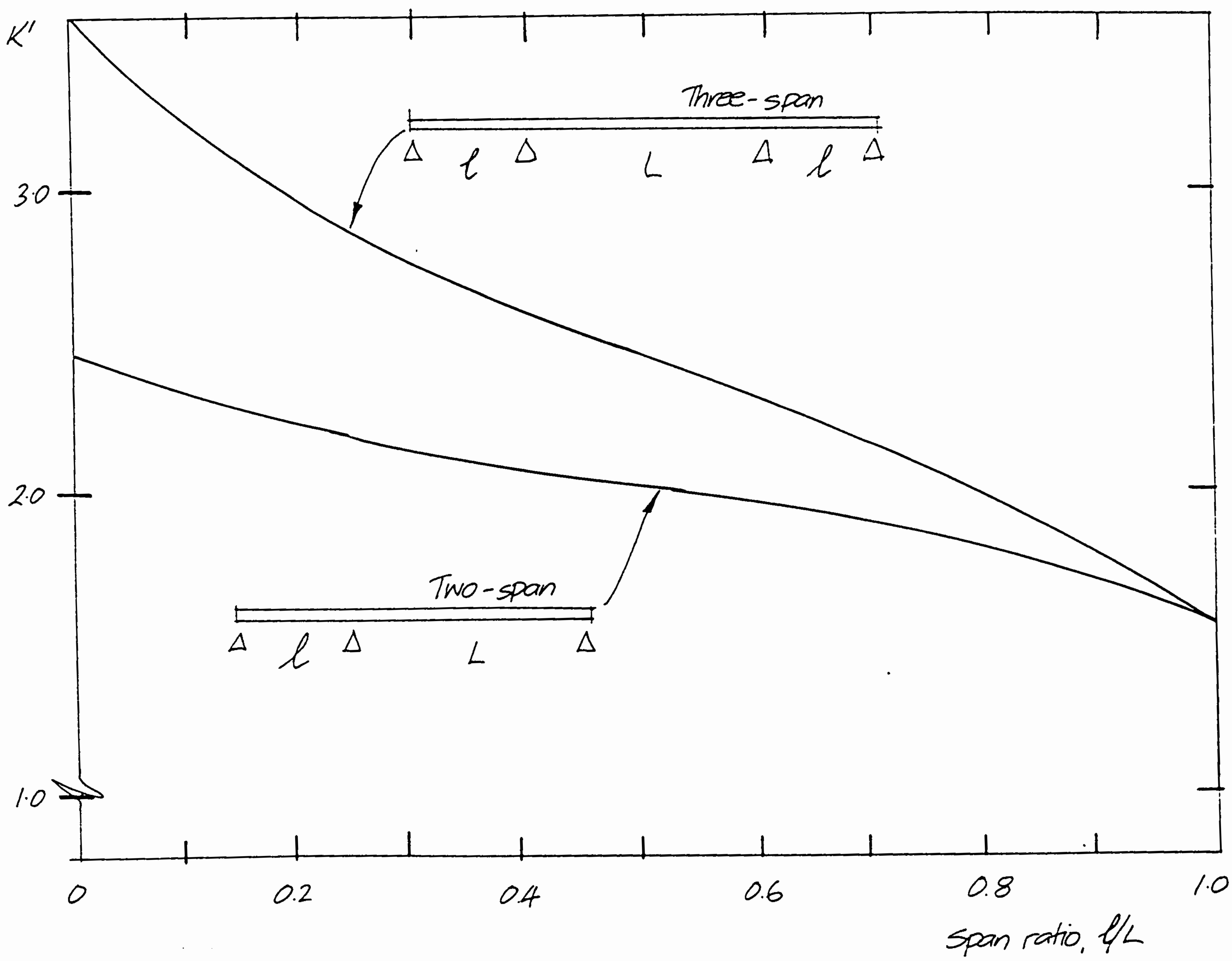


Figure 3 : Frequency factor for continuous beams from Wyatt (1989)

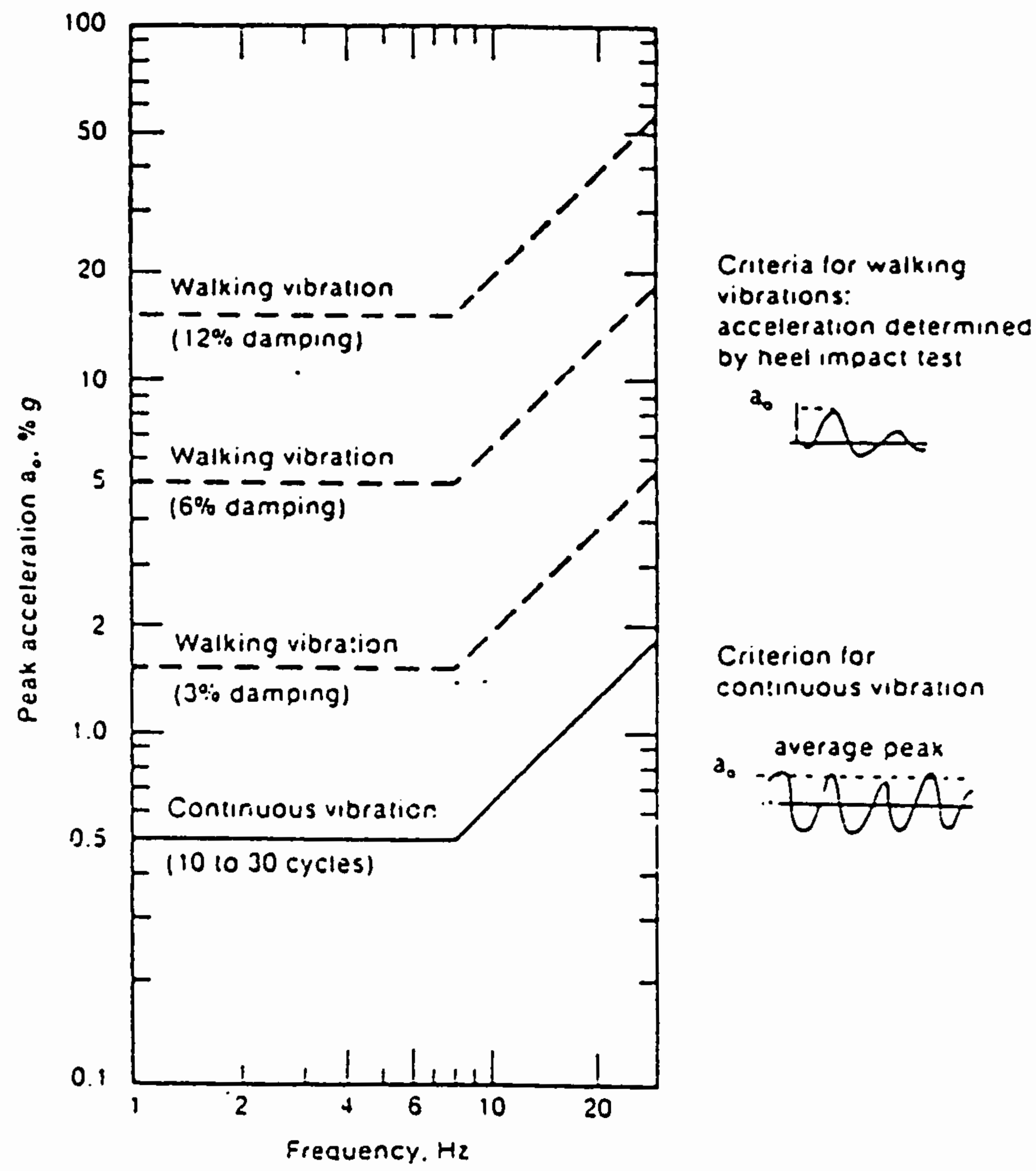


Figure 4 : Annoyance criteria for floor vibrations from CSA (1984)

	FLOOR	MEASURED DAMPING (% CRITICAL)
1	InSitu slab 5·40 x 4·05 P	10·2
2	InSitu slab 5·40	7·8
3	Unispan 6·00 x 6·50	12·2
4	Unispan 4·55 x 5·30	12·8
5	Precast rib 10·05 P	9·9
6	Precast rib 6·40	3·7
7	Precast rib 6·40 P	16·4
8	Precast rib 8·00 P-1	11·0
9	Precast rib 8·00 P-2	16·8
10	Precast rib 8·40 P-1	8·8
11	Precast rib 8·40 P-2	10·6
12	Composite 8·60 P-1	17·1
13	Composite 8·60 P-2	12·7
14	Composite 8·60 -1	8·2
15	Composite 8·60 - 2	5·3
16	Dycore 8·40	6·6
17	Double Tees 14·90 P	15·4

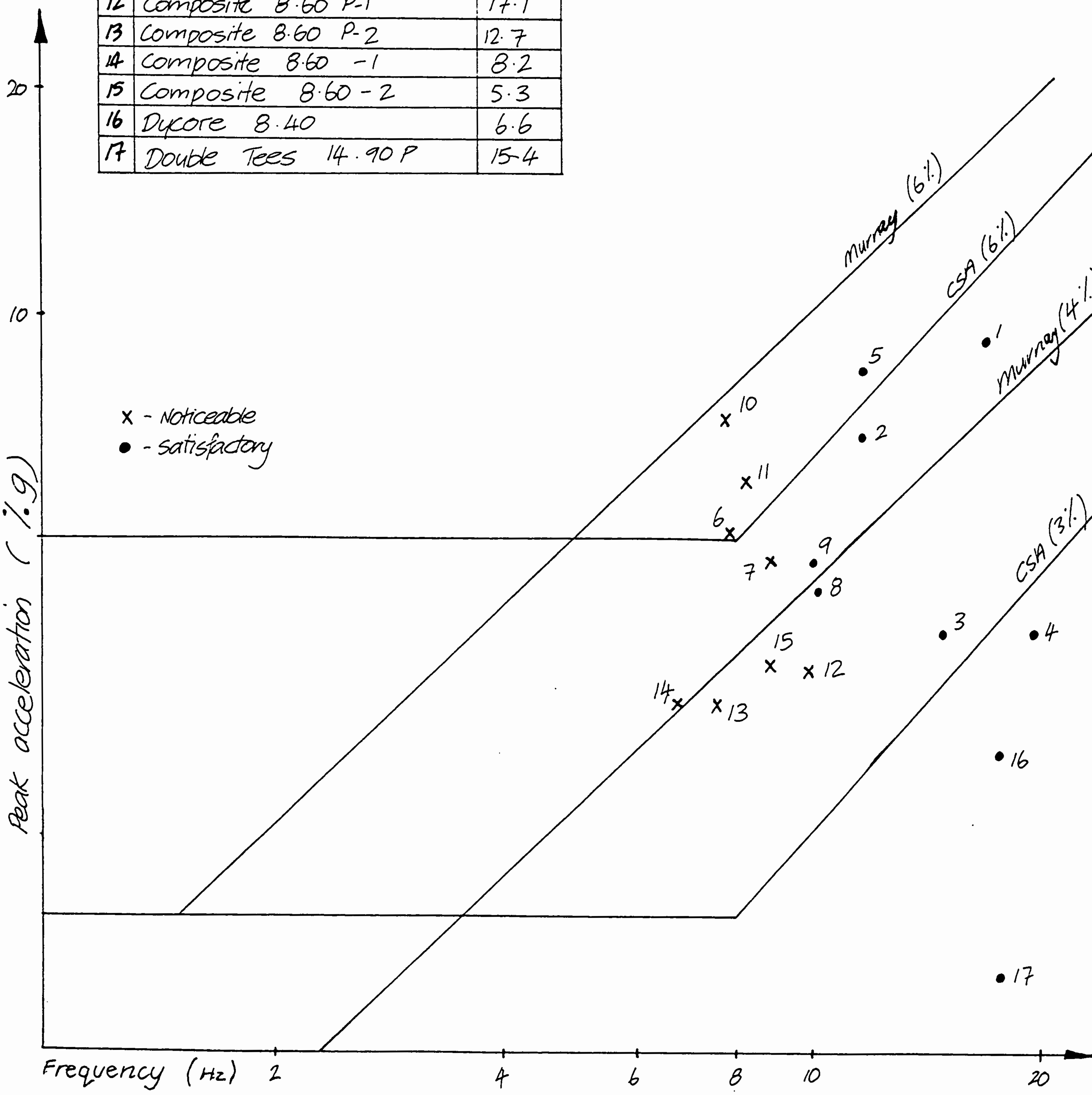


Figure 5 : Plot of measured values under heel drop impact against the Murray and CSA methods

TABLE 1 : MEASURED AND OBSERVED FLOOR VIBRATION CHARACTERISTICS

Floor Type	Span (m)	Impact Type	f_o (Hz)	a_o (%g)	A_o (mm)	Damping (% of critical) Half power Bandwidth	Damping (% of critical) Logarithmic Decay	Rating from walking
Insitu Slab	5.40x 4.05P	HD	15.6	9.4	0.82	10.2	9.7	S
		IH	16.9	5.1	0.41	7.2	9.3	
		HD on adjacent bay	17.6	3.8	0.32	6.2	6.5	
	5.40	HD	11.7	6.8	1.25	7.8	6.2	S
		HD on adjacent bay	13.0	8.1	0.56	12.4	8.9	
		HD on secondary beam	11.7	3.6	0.14	10.5	6.6	
		HD and accelerometer on secondary beam	12.4	3.8	0.41	7.2	5.1	
Precast Unispan	6.00x 6.50	HD	13.7	3.8	0.65	12.2	9.5	S
		HD on adjacent bay	15.6	1.3	0.11	13.3	6.4	
		IH	13.7	10.7	0.43	12.7	5.8	
		IH on supporting beam	13.7	3.8	0.26	13.3	6.3	
	4.55x 5.30	HD	19.5	3.8	0.37	12.8	7.0	S
		IH	21.5	19.7	0.42	5.0	7.6	
		IH on supporting beam	21.5	4.3	0.12	9.5	8.3	
Precast Rib	10.05P	HD	11.7	8.5	0.71	9.9	10.0	S
	6.40	HD	7.8	5.1	0.87	3.7	6.8	N
		IH	7.8	5.1	0.31	9.6	8.8	
	6.40P	HD	8.8	4.7	0.34	16.4	15.6	
		IH	9.1	3.8	0.26	13.9	12.3	
	8.00P	HD-1	10.7	4.3	0.10	11.8	9.9	
		*	11.1	4.7	0.15	-	11.3	

		SHD-1 *	11.2 11.1	3.4 4.7	0.17 0.12	10.3 -	8.4 11.9	S
		IH-1	11.2	9.0	0.14	11.2	13.2	
		HD-2 *	10.3 9.7	4.7 4.3	0.29 0.14	16.7 -	14.6 12.8	
		SHD-2 *	10.3 9.7	3.0 4.5	0.19 0.13	17.0 -	11.1 13.3	
	8.40P	HD-1	7.8	7.3	0.79	7.7	19.2	N
		SHD-1	7.8	5.1	0.68	9.8	14.8	
		HD-2	8.3	4.7	0.57	9.9	11.8	
		SHD-2	8.3	6.0	0.62	11.3	9.8	
Composite steel-concrete	8.60P	HD-1	9.8	3.4	0.27	17.1	13.7	N
		IH-1	9.8	8.5	0.12	12.0	15.7	
		HD-2	6.8	3.0	0.69	12.7	17.5	
		IH-2	7.6	9.0	0.27	19.2	?	
	8.60	HD-1	6.8	3.0	0.83	8.2	4.8	
		IH-1	6.8	11.1	0.42	14.8	?	
		HD-2	8.8	3.4	0.51	5.3	8.6	
Dycore	8.40	HD	17.2	1.3	0.03	6.6	6.4	S
Double Tees	14.90P	HD	16.6	2.6	0.20	15.4	14.2	S

Notation : HD = Heel drop; IH = Impulse hammer; SHD = Standard heel drop;
Number after impact type = test location; S =Satisfactory;
N Noticeable; P = Partitioned; ? = Cannot be determined.
* = Results from Burns and Yong (1990)

TABLE 2 : COMPARISON OF CALCULATED AND MEASURED FLOOR VIBRATION CHARACTERISTICS

Floor Type	Span (m)	Calculated/Estimated				Measured			
		f_o (Hz)	a_o (%g)	A_o (mm)	D (% critical)	f_o	a_o	A_o	D
Insitu Slab	5.40x 4.05P	16.5	NA	NA	6.0	15.6	9.4	0.82	10.2
	5.40	12.5	NA	NA	4.0	11.7	6.8	1.25	7.8
Unispan	6.00	7.6	3.2	0.41	4.0	13.7	3.8	0.65	12.2
	4.55x 5.30	18.9	NA	NA	4.0	19.5	3.8	0.37	12.8
Precast Rib	10.05P	5.1	1.7	0.40	6.0	11.7	8.5	0.71	9.9
	6.40	5.8	4.8	0.70	4.0	7.8	5.1	0.87	3.7
	6.40P	5.8	4.8	0.70	6.0	8.8	4.7	0.34	16.4
	8.00P	4.8	2.3	0.39	6.0	11.0 10.3	4.3 4.7	0.17 0.29	11.0 16.8
	8.40P	4.4	2.0	0.39	6.0	7.8 8.3	7.3 6.0	0.79 0.62	8.8 10.6
Composite steel-concrete	8.60Ps 8.60Pg 8.60Pt	6.2 4.4 3.6	3.7 2.7 2.1	0.24 0.10 0.29	6.0 6.0 6.0	9.8 7.6	3.4 3.0	0.27 0.69	17.1 12.7
	8.60s 8.60g 8.60t	6.2 4.4 3.6	3.7 2.7 2.1	0.24 0.10 0.29	4.0 4.0 4.0	6.8 8.8	3.0 3.4	0.83 0.51	8.2 5.3
Dycore	8.40	6.9	1.1	0.16	2.0	17.2	1.3	0.03	6.6
Double Tees	14.90P	3.6	0.3	0.14	6.0	16.6	2.6	0.20	15.4

Notes:

1. s = secondary beam; g = girder; t = two-way action
2. Only the data from heel drop and standard heel drop impact tests have been used for comparison. The average of the frequency and damping values have been tabulated. The peak acceleration and peak displacement, rather than the average values have been used.

3. Where tests were carried out on more than one location, the results of each location have been tabulated.

4. Where both heel drop and standard heel drop impacts have been conducted, the average damping obtained using the half-power bandwidth method has been tabulated.

TABLE 3 : COMPARISON OF PREDICTIONS FROM VARIOUS DESIGN METHODS - (Input Parameters Being Calculated Dynamic Characteristics)

Floor Type	Span (m)	f_o (Hz)	a_o (%g)	A_o (mm)	W (kN)	Damping required/ Damping estimated CSA	Murray	DW required/ DW calculated Allen
Precast Rib	10.05	5.1	1.7	0.40	335	3.2/6.0 S	5.3/6.0 S	7/15.1 S
	6.40	5.8	4.8	0.70	120	5.7/6.0 S	8.1/6.0 U	7/5.4 U
	8.00	4.8	2.3	0.39	214	4.0/6.0 S	5.1/6.0 S	14/9.6 U
	8.40	4.4	2.0	0.39	236	3.6/6.0 S	4.9/6.0 S	14/10.6 U
Composite	8.60s 8.60g 8.60t	6.2 4.4 3.6	3.7 2.7 2.1	0.24 0.10 0.29	231 231 231	5.1/6.0 S 4.4/6.0 S 3.8/6.0 S	4.6/6.0 S 3.1/6.0 S 3.9/6.0 S	14/10.4 U
Dycore	8.40	6.9	1.1	0.16	381	2.2/2.0 U	4.0/2.0 U	7/7.6 S
Double	14.9	3.6	0.3	0.14	1020	1.2/6.0 S	3.2/6.0 S	14/45.9 S

s = secondary beam; g = girder; t = two-way action; S = satisfactory; U = Unsatisfactory, Damping values of 4.0% and 6.0%; and 3.0% and 4.5% were assumed when considering transient and resonant response for unpartitioned and partitioned floors respectively.

TABLE 4 : COMPARISON OF PREDICTIONS FROM VARIOUS DESIGN METHODS - (Input Parameters Being Measured Dynamic Characteristics)

Floor Type	Span (m)	f_o (Hz)	a_o (%g)	A_o (mm)	W (kN)	Damping required/ Damping measured CSA	Murray	DW required/ DW measured Allen
Precast Rib	10.05P	11.7	8.5	0.71	335	8.8/9.9 S	14.0/9.9 U	3.5/16.6 S
	6.40	7.8	5.1	0.87	120	6.2/3.7 U	5.5/3.7 U	3.5/2.2 U
	6.40P	8.8	4.7	0.34	120	5.8/16.4S	6.2/16.4S	3.5/9.8 S
	8.00P	11.0	4.3	0.17	214	5.5/11.0S	5.1/11.0S	3.5/11.8 S
		10.3	4.7	0.29	214	5.8/16.8S	6.6/16.8S	3.5/18.0 S
	8.40P	7.8	7.3	0.79	236	8.1/8.8 S	11.0/8.8U	3.5/10.4 S
		8.3	6.0	0.62	236	6.9/10.6S	9.6/10.6S	3.5/12.5 S
Composite steel-concrete	8.60P	9.8	3.4	0.27	231	5.1/17.1S	6.2/17.1S	3.5/19.8 S
		7.6	3.0	0.69	231	4.6/12.7S	9.7/12.7S	3.5/14.7 S
	8.60	6.8	3.0	0.83	231	4.6/8.2 U	10.3/8.2U	7.0/9.5 S
		8.8	3.4	0.51	231	5.1/5.3 S	8.7/5.3 U	3.5/6.1 S
Dycore	8.40	17.2	1.3	0.03	381	2.4/6.6 S	3.2/6.6 S	3.5/12.6 S
Double Tees	14.9P	16.6	2.6	0.20	1020	4.4/15.4S	7.1/15.4S	3.5/78.5 S

S = Satisfactory; U = Unsatisfactory

Table 5 Dynamic Factors for Heel-Drop Impact from Murray (1989)					
f,Hz	DLF	f, Hz	DLF	f, Hz	DLF
1.00	0.1541	5.50	0.7819	10.00	1.1770
1.10	0.1695	5.60	0.7937	10.10	1.1831
1.20	0.1847	5.70	0.8053	10.20	1.1891
1.30	0.2000	5.80	0.8168	10.30	1.1949
1.40	0.2152	5.90	0.8282	10.40	1.2007
1.50	0.2304	6.00	0.8394	10.50	1.2065
1.60	0.2456	6.10	0.8505	10.60	1.2121
1.70	0.2607	6.20	0.8615	10.70	1.2177
1.80	0.2758	6.30	0.8723	10.80	1.2231
1.90	0.2908	6.40	0.8830	10.90	1.2285
2.00	0.3058	6.50	0.8936	11.00	1.2339
2.10	0.3207	6.60	0.9040	11.10	1.2391
2.20	0.3356	6.70	0.9143	11.20	1.2391
2.30	0.3504	6.80	0.9244	11.30	1.2494
2.40	0.3651	6.90	0.9344	11.40	1.2545
2.50	0.3798	7.00	0.9443	11.50	1.2594
2.60	0.3945	7.10	0.9540	11.60	1.2643
2.70	0.4091	7.20	0.9635	11.70	1.2692
2.80	0.4236	7.30	0.9729	11.80	1.2740
2.90	0.4380	7.40	0.9821	11.90	1.2787
3.00	0.4524	7.50	0.9912	12.00	1.2834
3.10	0.4667	7.60	1.0002	12.10	1.2879
3.20	0.4809	7.70	1.0090	12.20	1.2925
3.30	0.4950	7.80	1.0176	12.30	1.2970
3.40	0.5091	7.90	1.0261	12.40	1.3014
3.50	0.5231	8.00	1.0345	12.50	1.3058
3.60	0.5369	8.10	1.0428	12.60	1.3101
3.70	0.5507	8.20	1.0509	12.70	1.3143
3.80	0.5645	8.30	1.0588	12.80	1.3185
3.90	0.5781	8.40	1.0667	12.90	1.3227
4.00	0.5916	8.50	1.0744	13.00	1.3268
4.10	0.6050	8.60	1.0820	13.10	1.3308
4.20	0.6184	8.70	1.0895	13.20	1.3348
4.30	0.6316	8.80	1.0969	13.30	1.3388
4.40	0.6448	8.90	1.1041	13.40	1.3427
4.50	0.6578	9.00	1.1113	13.50	1.3466
4.60	0.6707	9.10	1.1183	13.60	1.3504
4.70	0.6835	9.20	1.1252	13.70	1.3541
4.80	0.6962	9.30	1.1321	13.80	1.3579
4.90	0.7088	9.40	1.1388	13.90	1.3615
5.00	0.7213	9.50	1.1434	14.00	1.3652
5.10	0.7337	9.60	1.1519	14.10	1.3688
5.20	0.7459	9.70	1.1583	14.20	1.3723
5.30	0.7580	9.80	1.1647	14.30	1.3758
5.40	0.7700	9.90	1.1709	14.40	1.3793

Table 6 Values for dimensions L_{eff} and S from Wyatt (1988)

Indicative floor layout	Qualifying conditions	L_{eff} (m)	S (m)
Case (1)	RF main beam < 0.2	L	S* but ≤ W*
	RF main beam < 0.2	L	Greater of S* of L but ≤ W*
Case (2)	I = L	2L	As for Case (1) above
	0.8L < I <L	1.7L	
	1 < 0.8L	L	
Case (3)	RF main beam < 0.6	2L	W*
	RF main beam > 0.6	L* but < l max	
Case (4)	$W_2 = W_1$	As for Case (3) above	$2W_1$
	$W_2 > 0.8 W_1$		$1.7W_1$
	$W_2 < 0.8 W_1$		W_1

Cases (1) and (2) of the indicative floor layouts are intended to be applied where the fundamental mode shape is governed by floor beam deflection.

Cases (3) and (4) are for mode shapes governed by main beam deflection.

APPENDIX A: CALCULATION OF FUNDAMENTAL FREQUENCY OF FLOORS

A.1 Insitu Slab - 114 mm thick

A.1.1 Partitioned floor 4.05 × 5.40 m

Dead load (DL) = $23.5 kN/m^3 \times 0.114$ m = 2.68 kPa
Superimposed Dead Load (SDL) = 0.4 kpa (ceiling; carpet, furnitures, etc.)
Live load (0.1 LL) = 0.1×2.5 kPa (for general office) = 0.25 kPa
Therefore DL + SDL + 0.1 LL = 3.33 kPa
Poisson's ratio of concrete, $\mu = 0.3$

Modulus of elasticity of concrete,
$E = 1.2 \times 0.043 \times W^{1.5} \times \sqrt{f'_c}$ from NZS 3101 (SANZ, 1982)

The 1.2 factor above is an allowance for actual concrete compression strength under in-service condition being approximately 40% greater than the 28 day strength (Bull, 1990). Assuming concrete with a weight of 2400 kg/m^3 and a 28 day concrete compression strength of 25 MPa, then
$E = 1.2 \times 0.043 \times 2400^{1.5} \times \sqrt{25} = 30340$ MPa

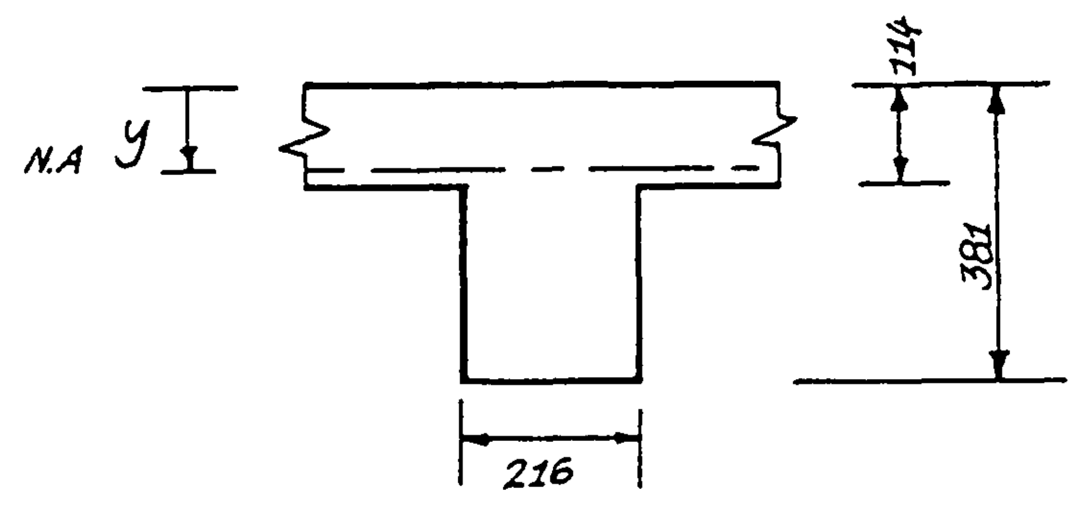
$D* = \frac{30340 \times 10^6 \times 0.114^3}{12 \times (1-0.3^2)} = 4.116 \times 10^6$ Nm

$M = \frac{3.33 \times 10^3}{9.81} = 339.5\ kg/m^2$

Since the slab is simply supported on four edges, from equation [2]
$f_0 = \frac{\pi}{2} \times \frac{5.4^2+4.05^2}{5.4^2 \times 4.05^2} \times \sqrt{\frac{4.116 \times 10^6}{339.5}} = 16.5$ Hz

A.1.2 Unpartitioned floor, secondary beam size = 381 × 216 mm, 5.4 m span

Dead load beam = $\frac{23.5kN/m^3 \times 0.267 \times 0.216}{4.05} = 0.34$kPa
Total load = 3.67 kPa



The distance to the neutral axis,
$y = \frac{114 \times 4050 \times 57 + 267 \times 216 \times 247.5}{114 \times 4050 + 267 \times 216} = 78$ mm

The transformed moment of inertia of the uncracked section,
$I = \frac{114^3 \times 4050}{12} + 114 \times 4050 \times 21^2 + \frac{267^3 \times 216}{12} + 267 \times 216 \times 170^2 = 2.703 \times 10^9$ mm

Since the 5.4 m span beam is continuous with equal spans, from equation [3]
$f_0 = 1.57 \times \sqrt{\frac{9810 \times 30340 \times 2.703 \times 10^9}{3.67 \times 4.05 \times 5400^4}} = 12.5$ Hz

A.2 "Unispan" floor - 145 mm overall

A.2.1 6.0 m span

Dead load (DL) = 23.5 × 0.145 = 3.41 kPa

Superimposed dead load (SDL) = 0.40 kPa
Live load (0.1 LL) = 0.25 kPa
DL + SDL + 0.1 LL = 4.06 kPa

Consider the system as a three span continuous floor with two short spans of 4.55 and a long span of 6.0 m (refer Figure 1b). From Figure 3, $K' = 2.00$

The second moment of inertia,
$I = \frac{145^3 \times 1200}{12} = 3.049 \times 10^8 \ mm^4$

$f_0 = 2.00 \times \sqrt{\frac{9810 \times 30340 \times 3.049 \times 10^8}{4.06 \times 1.2 \times 6000^4}} = 7.6$ Hz

A.2.2 4.55 × 5.30 m floor

Because the floor is also supported on a third edge by the shear wall, consider the floor as being simply supported on all four sides.

$D* = \frac{30340 \times 10^6 \times 0.145^3}{12 \times (1-0.3^2)} = 8.470 \times 10^6$ Nm

$M = \frac{4.06 \times 10^3}{9.81} = 413.9 \ kg/m^2$

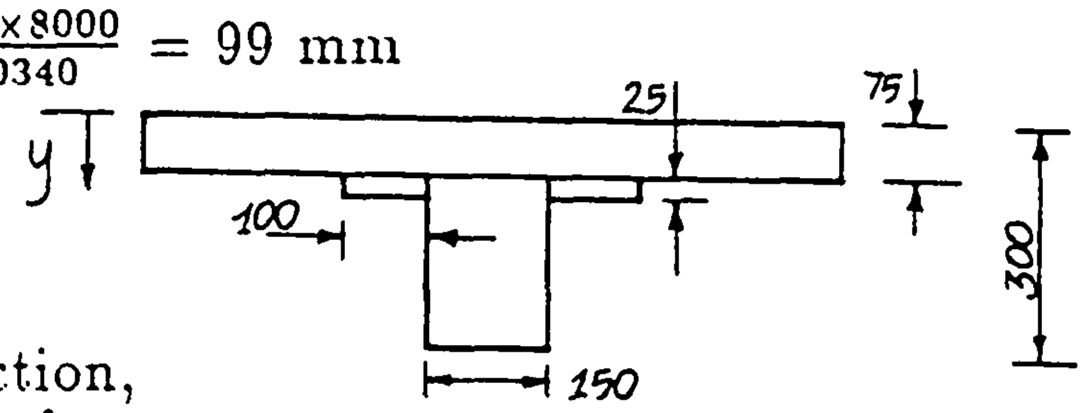
$f_0 = \frac{\pi}{2} \times \frac{4.55^2+5.3^2}{4.55^2 \times 5.3^2} \times \sqrt{\frac{8.470 \times 10^6}{413.9}} = 18.9$ Hz

A.3 Precast rib floor

A.3.1 10.05 m span - overall depth of 300 mm

Dead load (DL) = 2.92 kPa from manufacturer's data
Superimposed dead load (SDL) = 0.4 kPa and Live load (0.1 LL) = 0.25 kPa
DL + SDL + 0.1 LL = 3.57 kPa

Modulus of elasticity of concrete = 30340 MPa
Modulus of elasticity of timber = 8000 MPa
Therefore transformed timber width (to concrete) = $\frac{375 \times 8000}{30340} = 99$ mm



$y = \frac{900 \times 75 \times 37.5 + 198 \times 25 \times 87.5 + 225 \times 150 \times 187.5}{900 \times 75 + 198 \times 25 + 225 \times 150} = 87.5$ mm

The transformed moment of inertia of the uncracked section,
$I = \frac{900 \times 75^3}{12} + 900 \times 75 \times 50^2 + \frac{198 \times 25^3}{12} + 198 \times 25 \times 10^2 + \frac{225^3 \times 150}{12} + 225 \times 150 \times 100^2 = 681 \times 10^6 \ mm^4$

Consider the floor as a two span beam with the ratio of short to long span = $\frac{5.6}{10.05}$ = 0.56. Therefore $K' = 2.05$ from Figure 3.

$f_0 = 2.05 \times \sqrt{\frac{9810 \times 30340 \times 681 \times 10^6}{3.57 \times 0.9 \times 10050^4}} = 5.1$ Hz

A.3.2 6.4 m span - overall depth of 160 mm

Dead load (DL) = 2.27 kpa from manufacturer's data
Superimposed dead load (SDL) = 0.4 kPa and Live load (0.1 LL) = 0.25 kpa
DL + SDL + 0.1 LL = 2.92 kpa

From similar calculations as above,
y = 52.4 mm and I = 114.7×10^6 mm^4

The ratio of short to long span at the point of impact = $\frac{3.2}{6.4}$ = 0.5. Therefore K' = 2.09 and f_0 = 5.8 Hz

A.3.3 8.0 m span - overall depth of 250 mm

Dead load (DL) = 2.70 kPa from manufacturer's data
SDL = 0.4 kPa and Live load (0.1 LL) = 0.25 kPa DL + SDL + 0.1 LL = 3.35 kPa

y = 73.3 mm and I = 395.3×10^6 mm^4

Assuming simply supported conditions because in the bay adjacent to the tested location, the precast rib spanned in the perpendicular direction.
Therefore f_0 = 4.8 Hz

A.3.4 8.4 m span - overall depth of 250 mm

Continuous beam with equal spans, f_0 = 4.4 Hz

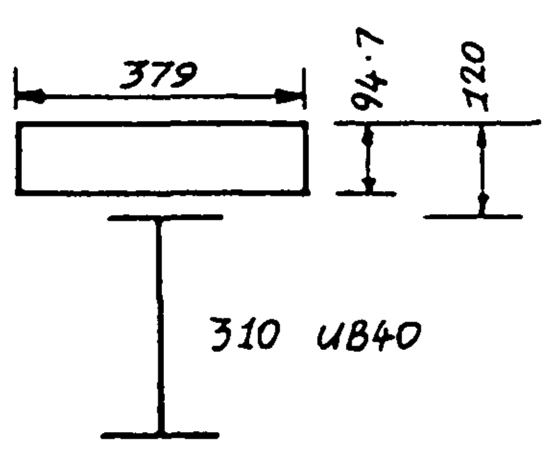
A.4 Composite steel-concrete floor (refer Figure 1g)

Dead load floor = 2.23 kPa
Dead load primary and secondary floor beams = 0.25 kPa
Superimposed dead load = 0.40 kPa and Live load (0.1 LL) = 0.25 kPa
Therefore DL (floor and beams) + SDL + 0.1 LL = 3.13 kPa

The equivalent concrete thickness, $t_c = \frac{2.23}{23.5}$ = 94.7 mm

Modulus of elasticity of concrete = 30340 MPa
Modulus of elasticity of steel = 200000 MPa
Transformed concrete width (to steel) = $\frac{2500 \times 30340}{200000}$ = 379 mm

Area of 310UB40 (Universal Beam) = 5150 mm^2
The second moment of inertia of the 310UB40, I = 85.2×10^6 mm^4
The second moment of inertia of the 360UB45 (primary beam in the interior of building away from the perimeter) I = 141.6×10^6 mm^4



$$y = \frac{379 \times 94.7 \times 47.3 + 5150 \times (152 + 120)}{379 \times 94.7 + 5150} = 75.5 \text{ mm}$$

$I = 85.2 \times 10^6 + 5150 \times 196.5^2 + \frac{94.7^3 \times 379}{12} + 94.7 \times 379 \times 28.2^2 = 339.4 \times 10^6\ mm^4$

Simply supported secondary beams,

$f_0 = 1.57 \times \sqrt{\frac{9810 \times 200000 \times 339.4 \times 10^6}{3.13 \times 2.5 \times 8600^4}} = 6.19$ Hz

The interior primary beam has been considered because the perimeter primary beam would be considerably stiffened by the presence of the spandrel/cladding system. Continuous primary beam of equal spans and assume that there is no composite action with the slab,

$f_0 = 1.57 \times \sqrt{\frac{9810 \times 200000 \times 141.6 \times 10^6}{3.13 \times 4.3 \times 7200^4}} = 4.35$ Hz

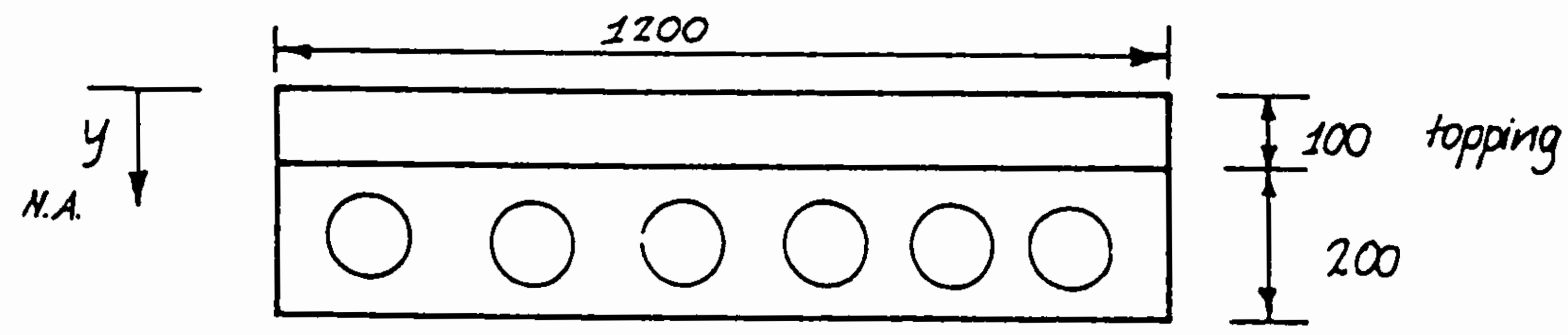
Consider two-way action of system, from Dunkerly's formula (equation [4]),

$\frac{1}{f^2} = \frac{1}{6.19^2} + \frac{1}{4.02^2}$, therefore f = 3.6 Hz

A.5 Dycore floor

Dead load Dycore = 2.40 kPa and Dead load concrete topping = 2.35 kPa
Superimposed dead load = 0.40 kPa and Live load (0.1 LL) = 0.25 kPa
Therefore DL (Dycore and topping) + SDL + 0.1 LL = 5.40 kPa

Area of Dycore = $0.1192 \times 10^6\ mm^4$
The second moment of inertia of "Dycore", I = $6.50 \times 10^8\ mm^4$



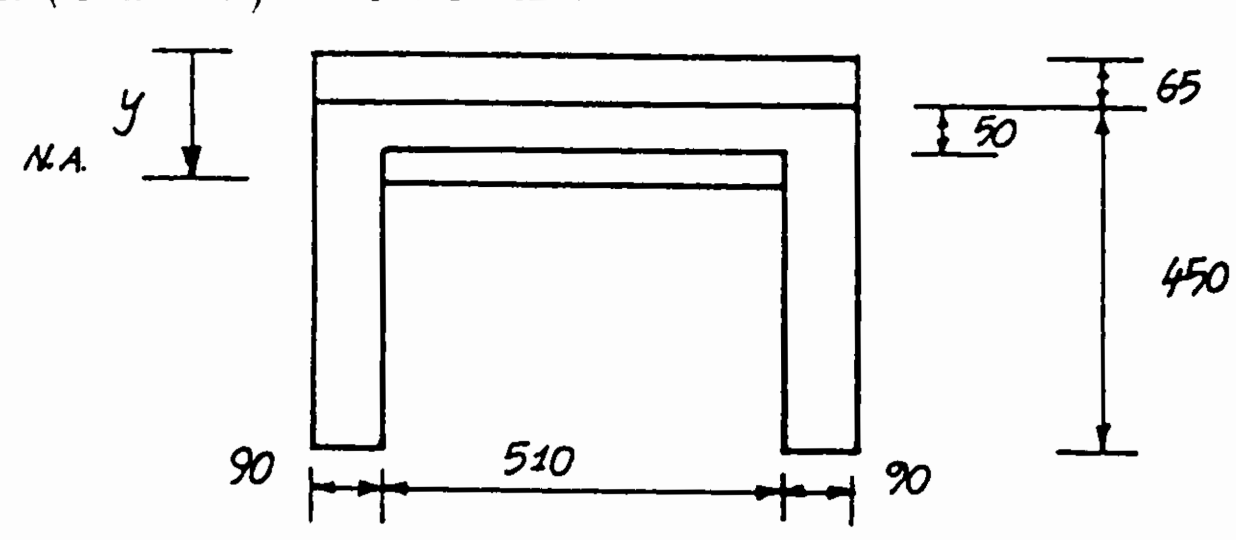
$y = \frac{200 \times 0.1192 \times 10^6 + 1200 \times 100 \times 50}{0.1192 \times 10^6 + 1200 \times 100} = 124.7$ mm

$I = 6.5 \times 10^8 + \frac{1200 \times 100^3}{12} + 1200 \times 100 \times 74.7^2 + 0.1192 \times 10^6 \times 75.3$
$= 2.095 \times 10^9\ mm^4$

Continuous beam with equal spans,
$f_0 = 1.57 \times \sqrt{\frac{9810 \times 30340 \times 2.095 \times 10^9}{5.40 \times 1.2 \times 8400^4}} = 6.9$ Hz

A.6 Double Tees floor (cropped unit)

Dead load = $\frac{23.5 \times (0.115 \times 0.69 + 2 \times 0.09 \times 0.4)}{0.69} = 5.15$ kPa
Superimposed dead load = 0.4 kPa and Live load (0.1 LL) = 0.25 kPa
Therefore DL + SDL + 0.1 LL = 5.80 kPa



$y = \frac{690\times115\times57.5+400\times180\times315}{690\times115+400\times180} = 180$ mm

$I = \frac{690\times115^3}{12} + 115 \times 690 \times 122.5^2 + \frac{2\times400^3\times90}{12} + 400 \times 180 \times 135^2 = 3.55 \times 10^9\ mm^4$

Continuous beam with equal spans,
$f_0 = 1.57 \times \sqrt{\frac{9810\times30340\times3.55\times10^9}{5.80\times0.69\times14900^4}} = 3.6$ Hz

APPENDIX B: APPLICATION OF THE DESIGN METHODS TO THE 6.4 M SPAN PRECAST RIB FLOOR USING THE CALCULATED/ESTIMATED VALUES.

B.1 Impulsive Response (see section 3.2.1)

B.1.1 CSA method

$f_0 = 5.8$ Hz from Appendix A.3.2

$a_0 = \frac{60 f_0}{W B_1 L}$ (from equation [5]),

$a_0 = \frac{60 \times 5.8}{2.92 \times (40 \times 2.27/23.5) \times 6.4} = 4.8\%$g

From Figure 4, damping required = 5.7%

B.1.2 Murray method

The static deflection under a 2.67 kN point load,
$d_s = \frac{2670 \times 6400^3}{48 \times 30340 \times 116.5 \times 10^6} = 4.125$ mm

DLF = 0.8168 from Table 5

$A_{0t} = 0.8723 \times 4.125 = 3.369$ mm and from equation [7],

$$N_{eff} = 2.97 - \frac{0.9}{17.3 \times (2.27/23.5)} + \frac{6400^4}{200 \times 30340 \times 116.5 \times 10^6} = 4.805$$

Therefore $A_0 = 3.369/4.805 = 0.701$ mm

From equation [8], $D > 1.38 \times 0.701 \times 5.8 + 2.5 = 8.1\%$. If an available damping of 6.0% was assumed, then the floor is predicted to be unsatisfactory to impulsive response using both methods.

B.2 Resonant Response (refer section 3.2.2)

B.2.1 Allen method

Since f_0 is 5.8 Hz, check for resonance to the third harmonic of walking frequency, i.e., $DW > 7$.

$W = 2.92 \times 6.4 \times B_3$ kN
where $B_3 = 2L \times \sqrt[4]{\frac{D_y}{D_x}} <= L$ from equation [10]

Assume conservatively that only the topping thickness contributes to the flexural rigidity in the direction perpendicular to the rib span,

$$B_3 = 2L \times \sqrt[4]{\frac{900 \times 75^3}{12 \times 116.5 \times 10^6}} = 1.444L > L. \text{ Therefore } B_3 = L = 6.4 \text{ m}$$

W = 120 kN and DW = 0.045x103 = 5.4< 7, i.e., the floor is unsatisfactory to the third harmonic of walking vibrations.

APPENDIX C: APPLICATION OF THE DESIGN METHODS TO THE COMPOSITE STEEL CONCRETE FLOOR USING THE CALCULATED/ESTIMATED VALUES.

C.1 Impulsive Response (see section 3.2.1)

C.1.1 CSA method
C.1.1.1 Secondary beam

$f_0 = 6.2$ Hz from Appendix A.4
$a_0 = \frac{60\times6.2}{3.13\times(40\times0.0947)\times8.6} = 3.7\%$ g

From Figure 4, damping required = 5.1%.

C.1.1.2 Primary girder

$f_0 = 4.3$ Hz

$a_0 = \frac{60\times4.4}{3.13\times4.3\times7.2} = 2.7\%$ g

From Figure 4, damping required = 4.4%.

C.1.1.3 Two-way action

Using Dunkerly's equation, f = 3.6 Hz

$a_0 = \frac{60\times3.6}{3.13\times(40\times0.0947)\times8.6} = 2.1\%$ g

From Figure 4, damping required = 3.8%.

The impulsive response is therefore governed by that of the secondary beam.

C.1.2 Murray method
C.1.2.1 Secondary beam

$d_s = \frac{2670\times8600^4}{48\times200\times10^3\times339.4\times10^6} = 0.521$ mm

$f_0 = 6.2$ Hz and from Table 5, DLF = 0.8615

$A_{0t} = 0.8615\times0.521 = 0.449$ mm

$N_{eff} = 2.97 - \frac{2.5}{17.3\times0.0947} + \frac{8600^4}{200\times200\times339.4\times10^9} = 1.847$

$A_0 = 0.243$ mm and $D > 1.38\times0.243\times6.2+2.5 = 4.6\%$

C.1.2.2 Primary girder

Assuming that the support conditions of the primary girder is 80% of fixed ended conditions, $d_g = 0.8 \times \frac{Pl^3}{192EI}$, i.e.,

$d_g = 0.8 \times \frac{2670 \times 7200^3}{192 \times 200 \times 10^3 \times 141.6 \times 10^6} = 0.147$ mm

$f_0 = 4.4$ Hz and DLF $= 0.6448$

Since the effective number of primary girders, $N_{efp} = 1.0$,
$A_{0t} = A_0 = 0.6448 \times 0.147 = 0.095$ mm

$D > 1.38 \times 0.095 \times 4.4 + 2.5 = 3.1\%$

C.1.2.3 Two-way Action

f = 3.6 Hz

The initial amplitude due to heel drop of the two-way system = the heel drop amplitude of the secondary beam + half the heel drop amplitude of the primary girder, $A_0 = 0.243 + \frac{0.095}{2} = 0.291$ mm

$D > 1.38 \times 0.291 \times 3.6 + 2.5 = 3.9\%$

Therefore, the secondary beam again governs the response to impulsive loading. If the estimated available damping in the floor is 6.0%, then the floor is satisfactory using Murray's method.

C.1.3 Wyatt method

Wyatt's method is not required to be checked for impulsive response since the fundamental frequency of the floor is less than 7 Hz, and resonant response dominates. This method is however shown below as a worked example.

The response factor, $R = \frac{30000}{MB_2L}$ where B_2 is the lesser of the secondary beam spacing or 40 times the equivalent concrete thickness. $B_2 = 2.5$ m in this case.

$M = \frac{3.13 \times 1000}{9.81} = 319\ kg/m^2$

Therefore $R = \frac{30000}{319 \times 2.5 \times 8.6} = 4.4 < 8$, hence the floor is satisfactory to impulsive response according to Wyatt's method.

C.2 Resonant Response (refer section 3.2.2)

C.2.1 Allen method

Even though the response of the secondary beam governs the impulsive response (as shown above), check for the second harmonic response of walking resulting from two-way action of the floor since $f = 3.6$ Hz, i.e., DW is required to be greater than

14 for the floor to be satisfactory.

$W = 3.13 \times B_3 \times L = 3.13 \times 8.6 \times 8.6 = 231$ kN;
$DW = 0.045 \times 231 = 10.4 < 14$, hence the floor is predicted to be unsatisfactory.

C.2.2 Wyatt method

The response factor, $R = \frac{68000C}{MSLD}$ where C = 0.4; L = 8.6 m,
$M = 313 kg/m^2$ D = 0.045

The deflection of the secondary floor beams under the imposed loading,
$d = \frac{5 \times 3.13 \times 2.5 \times 8600^4}{384 \times 200 \times 339.4 \times 10^9} = 8.21$ mm

The deflection of the primary girder under the imposed loading,

$d_g = 0.8 \times \frac{3.13 \times 4.3 \times 7200^4}{192 \times 200 \times 10^3 \times 141.6 \times 10^6} = 5.32$ mm

The relative flexibility of the main beam, $RF = \frac{5.32}{5.32+8.21} = 0.39$, and therefore S = greater of S* or L_{eff} but less than W* (refer Table 6).

$L_{eff} = 7.2$ m and W* = 25.4 m.

From Wyatt, $S* = 4.5 \times \sqrt[4]{\frac{EI_1}{Mf_0^2}}$ where

EI_1 is the dynamic flexural rigidity of slab $= \frac{30340 \times 10^6 \times 0.0947^3}{12} = 2.147 \times 10^6$ Nm

$S* = 4.5 \times \sqrt[4]{\frac{2.147 \times ^6}{319 \times 3.6^2}} = 21.5$ m

$R = \frac{68000 \times 0.4}{319 \times 21.5 \times 8.6 \times 0.045} = 10.2 > 8$, hence floor is also predicted to be unsatisfactory to resonant response using the Wyatt method.

Dynamic characteristics of New Zealand heavy floors.
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