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MEASURED CONTACT PRESSURES BELOW RAFT
SUPPORTING A STIFF BUILDING

by

W. J. Eden, G. C. McRostie and J. S. Hall

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Measured Contact Pressures Below Raft Supporting a Stiff Building¹

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Contact pressures between supporting clay and the foundation raft of a very stiff, 15-storey building indicate that pressures near the edge of the raft were up to three times those near the center. This condition persisted over a 2-year period. The difference in pressure is discussed in the light of the stiffness of the building. The raft design method suggested by an A.C.I. Committee in 1966 is compared with the measured performance and found to be reasonable for the raft itself but questionable for the foundation walls above the raft.

Les contraintes au contact entre l'argile portante et le radier de fondation d'un édifice très rigide de 15 étages indiquent que les contraintes près du bord du radier sont jusqu'à trois fois plus grandes que près du centre. Cette condition a persisté pendant une période de 2 ans. L'auteur examine cette différence de contrainte à la lumière de la rigidité de l'édifice. En comparant avec le fonctionnement mesuré la méthode de conception suggérée en 1966 par un comité de l'American Concrete Institute, on constate que celle-ci est convenable pour le radier même mais douteuse pour les murs de fondation au-dessus du radier.

The structural design of reinforced concrete raft foundations has been for many years one of the least satisfactory areas of design. Geotechnical engineers have tended to neglect it after making estimates of the immediate and long term settlements produced by an infinitely flexible surface load; and structural engineers have tended to start an analysis with the assumption of a soil modulus constant at all stress levels and unchanging with time. Concern has been expressed in the past (Teng 1949; Meyerhof 1965) and investigators are currently studying the problem.

Progress has been hampered by the lack of analytical methods sufficiently simple to be generally usable, yet based on realistic assumptions regarding soil behavior. Developments in analysis techniques have been reported by Barden (1962) and by Sommer

(1965), who considered the effect of superstructure stiffness, and by Ranganatham (1967), Rabinovici (1970), and Lin *et al.* (1971).

A major step forward in the development of procedures acceptable to practising designers occurred in 1966 when the American Concrete Institute, Committee No. 436 (A.C.I.-1966) published its report "Suggested design procedures for combined footings and mats". These analysis methods recognized the contribution of the superstructure to total building stiffness and the significance of total building stiffness in comparison with the stiffness of the soil mass. Greatly simplified soil reactions are, however, assumed in the A.C.I.-1966 procedure.

Current analysis methods using computer solutions such as ICES-STRUDL II are capable of including the assumption of a soil with variable linear contact pressure *versus* settlement characteristics. The contribution of the stiffness of the superstructure

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columns and slabs and of the foundation mat can also be considered in the analysis. Calculated or measured vertical deformations at each column location are used as input information.

Recent work (DeJong and Morgenstern 1971; Heil 1969, 1971) has recognized the changes that take place in building stiffness as storeys are added during construction. The need to include the effects of both short term settlements related to load additions and long term consolidation settlements occurring at constant load has also been recognized. The effect of creep in structural materials in reducing stresses caused by consolidation settlements can also be considered.

Not many case histories exist in the literature that give the magnitude and distribution of contact pressures at the bottom of a raft, but the more useful include the following: Lazebnik 1970; Gerrard *et al.* 1971; Focht *et al.* 1971; De Simone and Gould 1972.

Each new record provides useful background information for geotechnical and structural engineers involved in the structural design of foundations. The building now reported is not typical of past construction

such as framed, flat slab structures. It is, instead, typical of an increasing number of high rise residential buildings that have reinforced concrete shear walls and hence are much stiffer in at least one direction than conventional framed structures. A different distribution of contact pressures might be expected beneath such stiff buildings.

Site

The building is located at the north-east corner of Rideau and Wurttemberg Streets in Ottawa, close to the crest of the slope to the Rideau River. Soils include 6 ft (1.8 m) of random fill over 6.5 ft (2 m) of fine sand and silt on a Marine clay formation extending to a depth of 54 ft (16 m); there is 9 ft (2.7 m) of till on the shale bedrock at 63 ft (19 m). This sequence of strata is shown on Fig. 1. With the lower surface of the foundation raft near the top of the clay deposit, the clay deposit is the significant formation affecting the performance of the building.

Two borings were made on site, and continuous 54 mm ϕ piston samples were taken from one. Results of tests and the boring log are presented on Fig. 1. The test results indicate that the middle 10 ft (3 m) of clay were quite silty and rather sensitive. Results from this zone are probably more influenced by sample disturbance than results from the upper or lower clay layers. Preconsolidation determinations from standard consolidation tests indicate that the clay is overconsolidated by nearly $2t/ft^2$ (1.95 kg/cm^2).

As contact pressure would be influenced by the relative stiffness of the structure and the clay, some evaluation

FIG. 1. Geotechnical profile.

of a modulus of deformation of the soil mass was required. Attempts were made to determine a pseudo-elastic modulus from undrained triaxial tests. To minimize seating effects the test specimens were subjected to three cycles of loading and unloading, up to a deviator stress of 0.6 kg/cm². On the fourth loading cycle the secant modulus of the stress-strain curve was measured over the above stress range. The results of these tests are presented in Table 1. The range of E_u (undrained Young's modulus) was from 135 to 910 kg/cm², with an average of 480 kg/cm² for 12 tests. Previous experience (Burn 1969) has indicated that sample disturbance seriously affects E_u , so that the empirical rule of $E_u \approx 1000 S_u$ (D'Appolonia *et al.* 1971) was applied to the field vane strength determinations, giving a value for E_u of about 1540 k.s.f. (752 kg/cm²).

The modulus was also estimated by back calculation using the settlement measurements. This method has several inherent defects and cannot be expected to give more than the order of magnitude of the modulus:

(a) settlement measurements do not distinguish between consolidation settlements and purely elastic settlements;

(b) load increment varies from 0 to 100% over the construction period;

(c) elastic rebound is not considered; and

(d) stress distribution is not uniform throughout the clay layer.

Bearing these factors in mind, the first attempt to obtain a modulus used the following assumptions:

(a) all settlement from start of measurements to April 1971 (end of loading) were elastic, and were due to a load increment of 1420 lb/ft² (0.69 kg/cm²); and

(b) stress distribution over thickness of clay stratum was uniform and equal to 0.9 times average bearing pressure; a value of 0.9 was taken for an average factor from Jumikis' tables (1971) for conditions half-way through the clay layer and under the quarter-point of the long side of the building.

From the formula

$$E = \frac{q B}{\rho} I_p$$

E = modulus of elasticity

q = applied stress = 0.69 kg/cm²

B = width of the foundation = 55 ft or 1680 cm

I_p = influence factor dependent on length and width of the building and depth of the compressible layer = 0.9.

E was calculated to be 1040 kg/cm² (2130 k.s.f.) and may be compared with a value of E_u equal to 752 kg/cm² which was previously judged to be the most appropriate single value for purposes of structural design analysis.

Instrumentation and Measurements

Instrumentation for the building included 10 Gloetzel earth pressure cells, 15 survey reference points and a deep bench mark. The locations of the test installations (with the exception of the bench mark) are shown on Fig. 2.

The Gloetzel cells consist of a sealed flat jack approximately 8 × 12 in. (20 × 30 cm) filled with hydraulic oil (Lauffer and Schober 1964). Two tubes lead to a steel membrane encapsulating the oil in the jack. Measurements are taken by raising the pressure in the inlet line until oil leaks by a special orifice next to the membrane. The minimum pressure required to cause this flow is the pressure of oil inside the cell. Each cell is connected to the reading terminal by its own inlet line of copper tubing. The circuit is completed by a common return line connected to all cells.

Before the earth pressure cells were installed, a skim slab of lean concrete had been cast directly over the clay subsoil immediately after excavation. Wooden planks left in the skim slab were lifted for the installation of the pressure cells. Each cell was then installed on a carefully prepared surface of undisturbed clay and connected to its lead system. All were installed without difficulty, with the exception of No. 5. At this location the clay seemed to have been somewhat disturbed by excavating equipment and readings were too low. Following installation, all cells and the lead lines were covered with lean concrete and all leads were brought to a common point and taken up vertically through the foundation slab in a short length of pipe. Immediately

TABLE 1. Values of E_u determined from cyclic undrained triaxial tests

Test No.	Depth		Water content %	E_u	
	ft	m		kg/cm ² *	k.s.f.
182-1-3	16.1	4.90	72.5	417	854
182-1-4	16.5	5.03	71.5	525	1075
182-3-3	22.1	6.74	40.5	135	276
182-3-4	22.5	6.86	67.2	312	639
182-5-2	28.1	8.56	61.0	267	547
182-5-3	28.5	8.69	54.7	333	682
182-7-5	35.0	10.67	37.5	232	475
182-7-6	35.3	10.76	37.1	606	1241
182-9-3	40.1	12.22	44.2	555	1137
182-9-4	40.5	12.34	44.7	625	1280
182-11-3	46.1	14.05	49.4	909	1862
182-11-4	46.5	14.17	50.7	833	1706

*Secant modulus determined on fourth loading cycle of test in range 0-0.6 kg/cm² deviator stress.

following installation, work proceeded on the placing of reinforcing steel for the foundation raft. Concrete was placed 3 days later. Readings of contact pressure were made immediately after the slab was placed and at various intervals afterward. The time dependence of the pressure measurements is shown in Fig. 3. The readings indicate that high contact pressures developed near the edge of the slab and that they have persisted.

In general, the earth pressure cells appear to have performed satisfactorily, although results for No. 5 have been omitted because of possible disturbance of the clay subsoil. The cells were quite sensitive and measured the effect of wind pressure. This was confirmed on April 17, 1972 when readings were taken when a wind averaging 15 m.p.h. (24 km/h) was blowing from the northwest; readings were repeated on the following morning when the wind speed did not exceed 3 m.p.h. (4.8 km/h) from the east. Pressure due to wind is indicated on Fig. 3.

Settlements have been measured with a precise optical levelling system accurate to about 0.004 in. (0.01 cm). Reference points were installed in the ends of the shear walls as soon as possible (at this time work was proceeding on the building frame on the fourth floor). The locations of these points (Nos. 50 to 60) are indicated on Fig. 2.

When the building was completed, four additional points (Nos. 75 to 78) were installed on the shear walls near the center of the building. Level readings are taken with reference to a deep bench mark driven to 55 ft outside the building (Bozozuk *et al.* 1962).

Time dependent settlements of the reference points are presented on Fig. 4. Settlements of typical points at the end of load addition and after one year at constant load are presented on Fig. 5. These measurements indicate that the maximum settlement (from October 1970 to June 1972) is 3 cm (1.2 in.) and that a maximum differential settlement of 1.5 cm (0.6 in.) has occurred between the maximum and minimum points located on diagonally opposite corners.

The Structure

The building is a 15-storey residential structure on a raft foundation. The framing is very stiff transverse concrete shearwalls supporting concrete slabs. The shearwalls transmit the loads down to the raft and are cast integrally with a continuous perimeter

FIG. 2. Basement plan of building showing location of test installations.

foundation wall 10 ft (3 m) high between the raft and the ground floor. Table 2 gives some of the pertinent characteristics of the structure, and Fig. 2 shows the basement plan of the building and the location of the instrumentation.

Design Considerations

Rigidity of the superstructure affects the response of a raft and supporting soil by redistributing loads from the superstructure so that differential settlements will be compatible with elastic and creep deflections. This is a complex problem and rigorous mathematical analysis is not within the scope

of the average design office. One empirical procedure that can, however, be used to determine the distribution of soil reactions is the suggested design procedure for combined footings and mats hereinafter referred to as A.C.I.-1966. In this procedure, the relative stiffness (K_r) of the combined stiffnesses of footing, superstructure, and framing members (including shearwalls) in comparison with the stiffness of the soil determines whether a straight line distribution or a distribution governed by elastic subgrade reaction should be used. As shown in Table 2, the relative stiffness (K_r) is greater than 0.5 except for the raft taken alone. A.C.I.-1966 recommends that the footings or raft

FIG. 3. Variation in contact pressure with time.

strips be considered rigid and that variation of soil pressures be determined by simple statics.

Design Contact Pressures

Loads imposed on the soils include:

- (a) negative load created by excavation to an average depth of 13 ft 0 in. (3.96 m);
- (b) dead weight of the foundation raft;
- (c) dead weight of total building structure; and
- (d) long term sustained live load.

The excavation resulted in a temporary removal of 1.30 k.s.f. (0.64 kg/cm²) in the raft area. The combined average gross area load of the building, consisting of raft, dead load, and sustained live load, is 2.72 k.s.f. (1.33 kg/cm²). Loads (c) and (d) are transmitted to the raft along the transverse shearwalls. The net average area load of the building is therefore 1.42 k.s.f. (0.69 kg/cm²). Under earthquake or hurricane conditions the maximum gross bearing pressure at the edge of the raft, as computed by P/A plus

FIG. 4. Settlement-time records, transverse sections.

FIG. 5. Settlements along east and west sides of building.

Mc/I analysis, would be 3.25 k.s.f. (1.59 kg/cm²).

Distribution of soil pressure on the basis of simple statics, in accordance with the suggested procedure of A.C.I.-1966, resulted in a uniform pressure across the building raft of 2.37 k.s.f. (1.16 kg/cm²) (Fig. 7). Because of the compactness of the structure and its considerable stiffness in both directions it was the opinion of the designers that edge contact pressures higher than average should be assumed, in agreement with the mathematical concept of a rigid plate placed on an elastic medium.

To include the effects of soil structure interaction in pressure distribution that would reflect higher edge pressures, a pattern of raft reaction was determined and an iterative procedure set up for calculating settlement and corresponding deflections of the structure below selected reference points. The final raft reaction was varied until the settlements and deflections at the various reference points reached close agreement. As the K_r value for the raft alone is less than 0.5, the raft strip can be considered flexible so that pressure will vary according to foundation modulus theories. A series of rectangular pressure strips was chosen to represent the

actual pressure distribution for easier computation.

Figure 7 shows the calculated pressure profiles for a transverse strip along the center shearwalls, for longitudinal strips on the center line of the building, and for a strip midway between the center and the edge. These strips were also chosen as the locations for the observation pressure cells. The profiles compare pressure distribution (according to A.C.I.-1966) with design pressures and with observed readings for April 1971, when full dead load was applied to the foundation, and for the latest readings taken in June 1972. April 1971 can be considered as the end of any elastic movements dependent on increased loads. June 1972 represents conditions after about 1 year of movement under constant load.

Deflection Profiles

Figure 6 shows the calculated deflection profile for the structure along the center line of the longitudinal axis of the building. As may be seen in Table 2, the only important stiffness in the end bay longitudinal direction is that from the lowest part of the building. When the building was completed to the ground floor, the stiffness (I) of an end bay

was 2670 ft⁴ (23.0 m⁴). The addition of 15 additional levels added only about 12 ft⁴ (0.103 m⁴) to the calculated stiffness.

Evaluation of creep effects was based upon the recommendations of the National Building Code of Canada 1965, section 4.5.4.9(5), which was in effect when the building was designed. The ratio of compression steel to tension steel is small, and a creep factor of 2.0 was used.

Comparison of the observed deflections with the calculated ones was attempted but

was not satisfactory. The variation in observed deflections from the north end bay with that of the south end bay could not be explained. The amount of observed deflection at the center line on the longitudinal axis also disagreed with the calculated deflections as performed using the stiffnesses as given in Table 2. Deflections in the transverse direction are not plotted, but are calculated to be only 0.004 in. (0.0102 cm) because of the very great stiffness in the transverse direction contributed by the shearwalls.

TABLE 2. Structural data

Characteristic	Value
Number of storeys above grade	15
Number of basements	1
Elevation of base of raft relative to ground floor	-13 ft 0 in. (3.96 m)
Structural raft thickness	3 ft 0 in. (0.91 m)
Dead weight of superstructure (incl. partitions)	10 933 kips (4953 tonnes)
Dead weight of total building (incl. partitions)	14 219 kips (6441 tonnes)
Total load of building	14 672 kips (6645 tonnes)
Area of raft	5390 sq ft (501 m ²)
Average gross load of total building	2.72 k.s.f. (1.33 kg/cm ²)
<i>Typical I values, ft⁴ transverse</i>	
Foundation raft	per 21 ft 0 in. bay (6.4 m) 47.0 ft ⁴ (0.404 m ⁴)
Slabs (total 15 levels)	per 21 ft 0 in. bay (6.4 m) 5.1 ft ⁴ (0.044 m ⁴)
Shearwall	per 21 ft 0 in. bay (6.4 m) 87 381.0 ft ⁴ (754.1 m ⁴)
<i>Longitudinal</i>	
<i>(a) End bay of building</i>	
Foundation raft	per 51 ft 0 in. width (15.5 m) 115.0 ft ⁴ (0.992 m ⁴)
Ground floor slab, foundation wall, and raft	per 51 ft 0 in. width (15.5 m) 2670.0 ft ⁴ (23.04 m ⁴)
Ground floor slab, foundation wall, and raft (A.C.I.-1966)	per 51 ft 0 in. width (15.5 m) 415.0 ft ⁴ (3.57 m ⁴)
Slabs (total 15 levels)	per 51 ft 0 in. width (15.5 m) 12.0 ft ⁴ (0.103 m ⁴)
<i>(b) Center bay of building</i>	
Foundation raft	per 65 ft 0 in. width (19.8 m) 146.0 ft ⁴ (1.255 m ⁴)
Ground floor slab, foundation wall, and raft	per 65 ft 0 in. width (19.8 m) 2791.0 ft ⁴ (24.0 m ⁴)
Ground floor slab, foundation wall, and raft (A.C.I.-1966)	per 65 ft 0 in. width (19.8 m) 446.0 ft ⁴ (3.84 m ⁴)
Slabs (total 15 levels)	per 65 ft 0 in. width (19.8 m) 15.0 ft ⁴ (0.129 m ⁴)
Elevator shaft and stairwells	per 65 ft 0 in. width (19.8 m) 174 762.0 ft ⁴ (1508.2 m ⁴)
Relative stiffness, $Kr = \frac{EcI_B}{Eu b^3}$ (A.C.I.-1966)	
<i>Longitudinal, Kr</i>	
End bay	0.94
Center Bay	176.0
Raft only, end bay	0.24
<i>Transverse, Kr</i>	
	3535.0
Ec = modulus of elasticity of the materials used in the structure (432 000 k.s.f.) (210900 kg/cm ²)	
Eu = modulus of elasticity of the soil (1540 k.s.f.) (752 kg/cm ²)	
I_B = moment of inertia of structure per unit length, ft ³	
b = width of footing or raft strip, ft

FIG. 6. Calculated deflection of structure, longitudinal axis.

FIG. 7. Pressure diagrams.

Bending Moment Diagrams

Figure 8 shows the bending moment for the reinforced concrete raft, calculated for the same strips as for the previous diagrams. The bending moment diagrams assume no differential displacement between shearwalls and assume that the downward reaction of the shearwalls on the raft is equal to the design loads.

Figure 9 gives the bending moment and shear for the total structure in the longitudinal direction. This includes raft, foundation walls, all floor slabs, elevator shafts, and stairwells. The values plotted for April 1971 and June 1972 have been adjusted so that the pressure cell readings equal the actual load on the structure at that time. The product of pressure cell readings times

FIG. 8. Bending moment diagrams for reinforced concrete raft only.

the raft area did not equal the building weight, owing, it was deduced, to the fact that the contact pressures were highest at the extreme edges of the raft where no pressure cells had been installed. The product of pressure cell readings times raft area changed with time indicated and a decrease in probable extreme edge pressures. This agreed with the expected changes resulting from redistribution of contact pressures that would reduce the highest values and increase the lowest.

There are significant differences between observed values and values obtained using the A.C.I. method for the design of the superstructure. The results are actually of

more significance in the design of perimeter foundation walls than for the raft itself.

Raft Reinforcement

Reinforcement used in the raft of this particular building is illustrated in Fig. 8. Because the building is relatively narrow, areas of reinforcement were calculated at critical strips and maximum values were used for the total width. For larger buildings the areas could be adjusted for each strip, as dictated by stress requirements.

Conclusions

(1) Observed contact pressures between a raft on clay and a stiff building varied

FIG. 9. Bending moment and shear diagrams for total structure, longitudinal axis.

greatly from a uniform condition, were up to three times greater at the edge than at the center, and did not redistribute or equalize themselves greatly over the first 2 years of the life of the structure.

(2) The observed pattern of contact pressure distribution did not change significantly during the construction period. The total structure increased in stiffness as each storey was added, but the foundation walls alone, acting in conjunction with the raft, had already created a relation where the structure was very stiff in comparison with the soil mass. It may be noted that foundation wall stiffness would not have such an important effect on buildings of greater length to width ratio.

(3) A raft designed to A.C.I.-1966 recommendations would have been adequate and slightly conservative for this particular, very stiff, building. Foundation walls immediately above the raft would, however, need to be reinforced to resist the large forces that must be redistributed; these would not be anticipated by the contact pressure distributions suggested in the A.C.I.-1966 method.

(4) Judgement must still play a large part in raft design. There remain significant unknown site factors such as laterally varying soil conditions and irregular soil shrinkage or swelling conditions, even when design methods are available that recognize the relative stiffnesses of building and soil, the increase in building stiffness during construction, and the effects of differing creep rates of concrete and soil in reducing stresses due to long term differential settlements.

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