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# Lakeview Tower: Case History of Foundation Failure

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**SYNOPSIS** The 14 storey Lakeview Towers apartment building in Sudbury, Ontario, Canada began with the site investigation in 1972 and ended 10 years later by demolition. During this interval settlements of over 400 mm occurred in one section of the structure. The reasons for the settlement revolved around a change in foundation design from piles to spread footings on improved ground, complicated by an unexpected layer of highly compressible clay. While the building may have been able to tolerate the settlement depending on the reader's interpretation of the results, it was demolished after legal disputes. A final structural analysis indicated a deficiency related particularly to earthquake loading.

In the summer of 1972 the geotechnical consultant was authorized to carry out the site investigation for a four storey apartment/commercial complex. The building, approximately 165 m x 25 m was proposed to be constructed without basement. The site investigation consisted of four widely spaced holes inside the proposed building area. The stratigraphy was found to be relatively simple, compact to dense silty sand over bedrock at 4.3 to 6.1 m below present ground surface. The consultant recommended spread footings for this four storey building utilizing a safe net allowable bearing pressure of 0.15 MPa for footings 1.5 m below ground surface.

The geology of the North Bay area is well known for its erratic bedrock profile and glacial sand and clay deposits. At the site location the superficial deposit is mapped as Wisconsinan aged sand. The grey to brown sand and silty sand was deposited below water plane as proglacial lake sands. These lacustrine deposits are often associated with moraines and eskers and commonly show repetitious interbedding with silt and clay and occasionally grade upward into varved clay. The sands are expected to overly red and grey noncalcareous silt till and bedrock consisting of Precambrian gneiss. Harrison, J.E. (1972).

Soon after the submission of the site investigation report the building proposed was changed to a 14 storey reinforced concrete building with brick infill, again without basement. The building shape altered considerably and extended beyond the original site investigation. Fig. 1. The original site investigation was considered to be adequate for the 14 storey building as the new foundations recommended were piles or caissons and little variation in subsoil conditions was evident from the four borings.

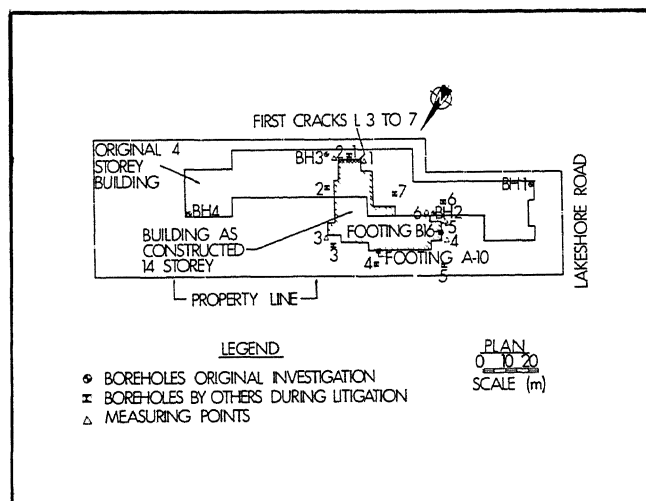


Fig. 1. Building Layout

Immediately prior to construction in 1973 the owner was approached by an international firm specializing in ground improvement using vibration to obtain either increased density and/or stone columns to support the load from conventional spread footings. This type of ground improvement had not been used in the Province of Ontario and the Canadian reference provided by the contractor was a single site in Quebec.

For this type of construction it is not uncommon for the owner to act as his own project manager and contractor. He will retain the geotechnical consultant, the architects, the engineer, and will then subcontract each facet of the construction. It was therefore not uncommon for the owner to directly engage the foundation contractor. In this case the structural consultant and the geotechnical consultant were advised of the alternative to the piling proposed and they

offered no constructive criticism. The process was, after all, virtually unknown in Canada yet the contractor specified that "Total or differential settlements will take place when the footings are loaded and will not exceed 12 mm. Any settlement after the building is erected will be negligible". This contractor further stated "Our work is covered by 100% performance bond issued upon request and by comprehensive general liability including completed operation". The process was accepted in Europe. Greenwood, D.A. (1970). The contractor explained that he would use vibroflotation down to bedrock at an average depth of 5.2 m.

Since no adverse comments were forthcoming, the

owner retained the contractor to complete the ground improvement and to utilize spread footings designed with a safe net bearing value of 0.5 MPa resting on the improved subsoil. The soil consultant was also retained to check the results to ensure that they were satisfactory.

In retrospect, no one is certain what the foundation contractor intended to do. His contract called for 254 compaction points, each with a safe working load of 150 kps. This would indicate a vibro-replacement system yet this contractor stated "We are positive that a densification of the subsoil by means of the vibroflotation method is feasible", possibly inferring a vibro compaction system.

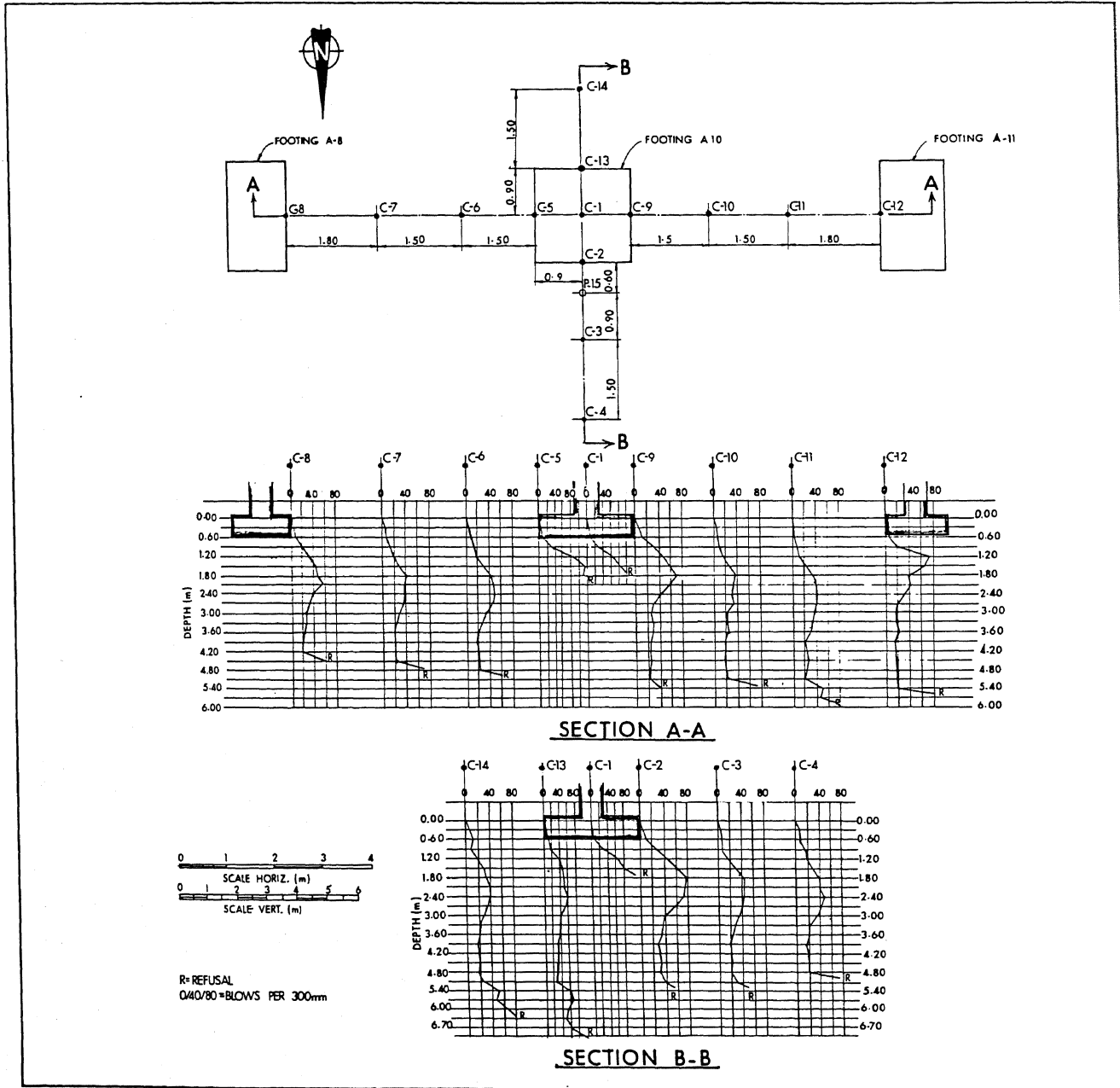


fig. 2. Subsoil Improvement Footing A 10

The site work to improve the subsoil began without delay. The contractor began work in the north central section of the site - an area that had been covered by the initial site investigation.

To prove that the vibratory subsoil improvement system was adequate, the soil consultant elected to use a modified S.P.T. test. This test, while empirical, is extensively used in the area for estimating safe net bearing values, and as the before and after results were available for comparison, this was felt to be the most rapid and economic approach. The procedure consists of driving a 50 mm diameter 60° solid cone using 'A' rods. The

energy used was equivalent to the S.P.T. test, i.e. 63.6 kg hammer dropping 0.76 m. This procedure, defined locally as a dynamic cone test, has the disadvantage of friction build-up along the rods. Fig. 2 summarizes the results for footing A 10.

The foundation contractor elected to improve the subsoil in the area of the larger footings (approx. 2.1 m x 2.1 m) using five "compaction points", one at each corner and one in the centre. For smaller footings (approx. 1.2 m x 1.2 m) two compaction points were to be used. The compaction points were in fact stone columns formed using 15 mm clear stone.

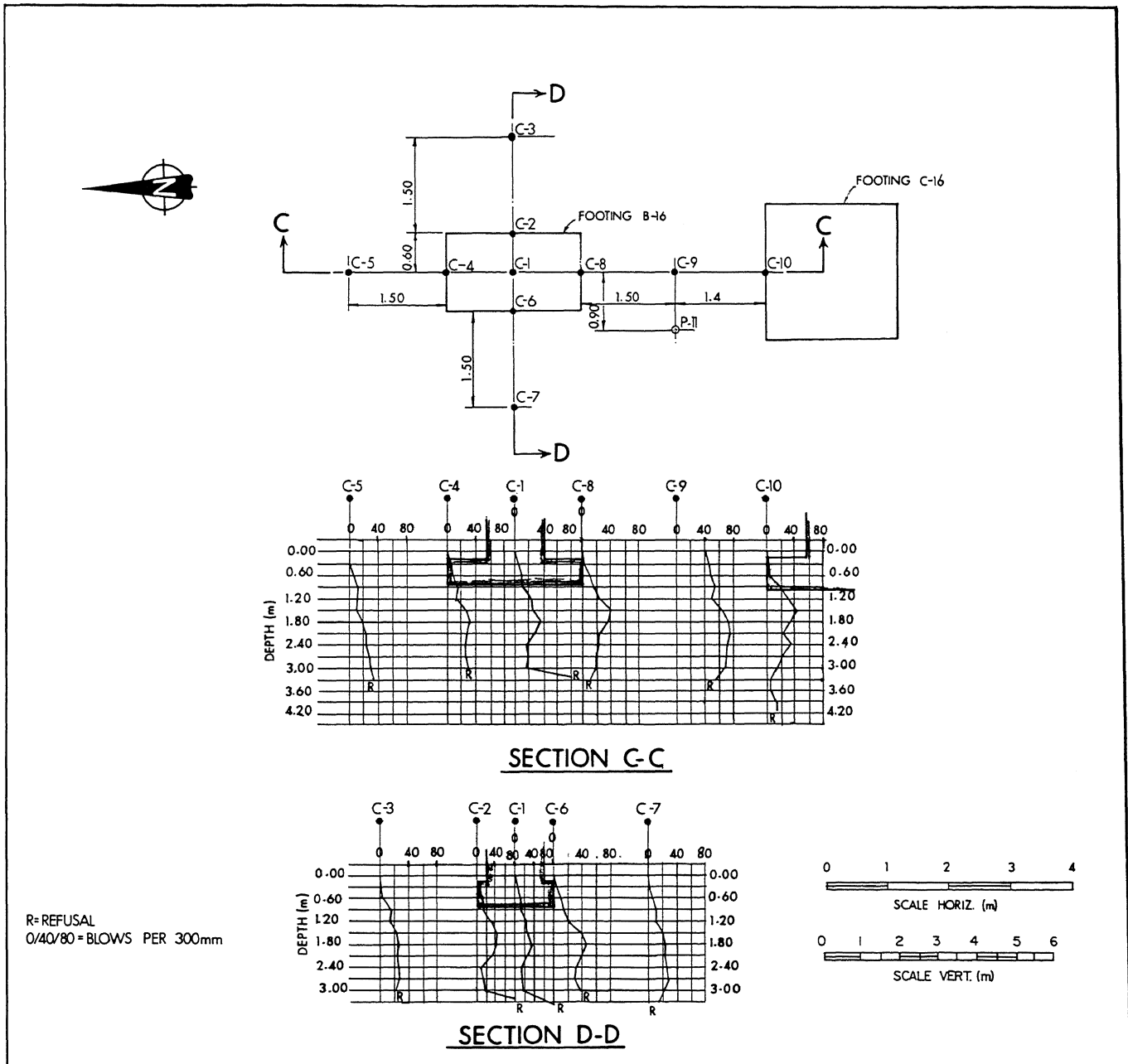


Fig. 3. Subsoil Improvement Footing B 16.

On evaluation of the test results, the soil consultant accepted the procedure of five compaction points for the larger footings despite the problem of compaction in the upper region of low confining pressure, but rejected the two compaction point procedure for the small footings. Fig. 3 shows the original results of inadequate compaction for footing B 16. The contractor agreed to increase to four compaction points for these footings and the soil consultant withdrew from the site.

No accurate records are available for the work carried out by the foundation contractor. Whether electrical input measurements controlled the vibration; whether the wet or dry procedure was used; what quantities of stone were used, or in fact the actual depth of the soil improvement, is not known.

The first public indication of a settlement problem was documented by the site meeting on August 21, 1973, some three months after completion of the foundation and site improvement and when the building was at approximately the 10 storey construction level. At this time, settlement measurements taken as a routine procedure, indicated movements of 70 mm over a one month period (measuring point No. 1). The foundation contractor, soils consultant, structural designer and builder (owner) were advised of hairline cracks occurring in the area along line L between lines 7 and 3. A program of measurements on columns was set out as well as additional soil investigation in the problem area. "The masonry on the remaining floors is to proceed on schedule in order to achieve maximum loading".

At this time in the building's history, i.e.

July, 1973, it can be asked "Why did construction continue if significant settlements were observed and a change in soil conditions was suspected?". This question cannot be answered but some of the factors leading to the decision on July 20, 1973 to continue were: that the building was nearing completion, i.e. at approximately the 10 storey level with nearly 70% of the dead load in place; no significant cracking could be found in the building although minor cracking as reported was observed.

Economic pressures dictate as short as possible construction cycle; publicity would not be beneficial. The condition of the building and the settlement observations taken over the following few weeks indicated to the geotechnical engineer that despite the increased loads the settlement was reducing. Certainly all parties at the meeting agreed construction should proceed.

The additional soil investigation indicated a layer of soft clay reaching a maximum thickness of 2.8 m near measuring point No. 2, with zero thickness near point No. 4 (Fig. 1). This clay layer was described as soft grey silty clay having increased silt content with depth. Laboratory test results for the clay layer gave values of approximately 71 to 76% for moisture content; 71 to 72% for liquid limit; 22 to 25% for plastic limit; 15.6 kN/m<sup>3</sup> unit weight. Undrained triaxial compression tests placed the shear strength in the 16 to 19 kPa range. Consolidation tests carried out in the oedometer indicated the preconsolidation pressure at the overburden pressure.

The subsoil stratigraphy for the site was now accurately known and is shown as Fig. 4.

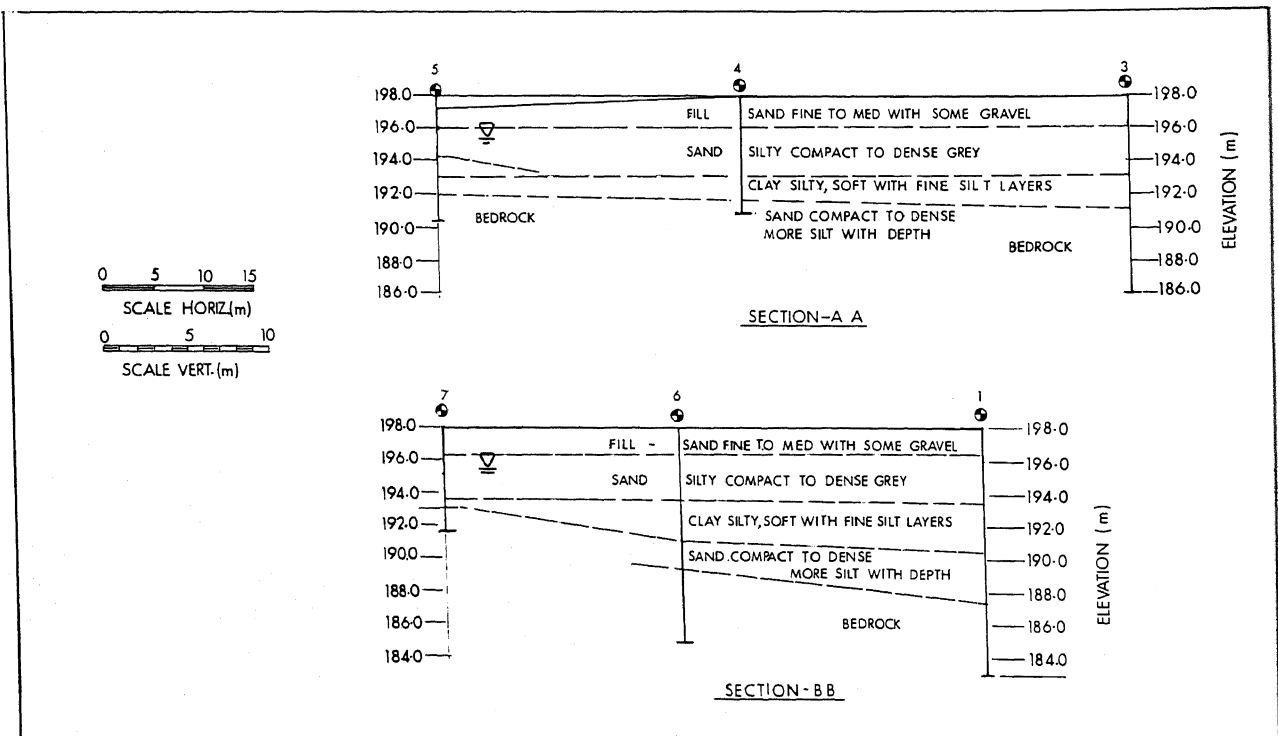


Fig. 4. Subsoil Stratigraphy

Settlement measurements were undertaken by a registered land surveyor using 6 points established on the building. The readings had begun July 20, 1973 and were continued until May 1977. During this period the points were moved from inside to outside the building explaining some of the curves, and several points were lost for a few months. The bench marks used for the survey were the original intended only for construction control. A brief study of one of these reference points indicated movements of 1.5 mm. The movements of the bench mark are not uncommon in this area and can be attributed to the movement of frost into and out of the ground.

The settlement of the building as recorded over the 3 year period is shown in detail on Fig. 5.

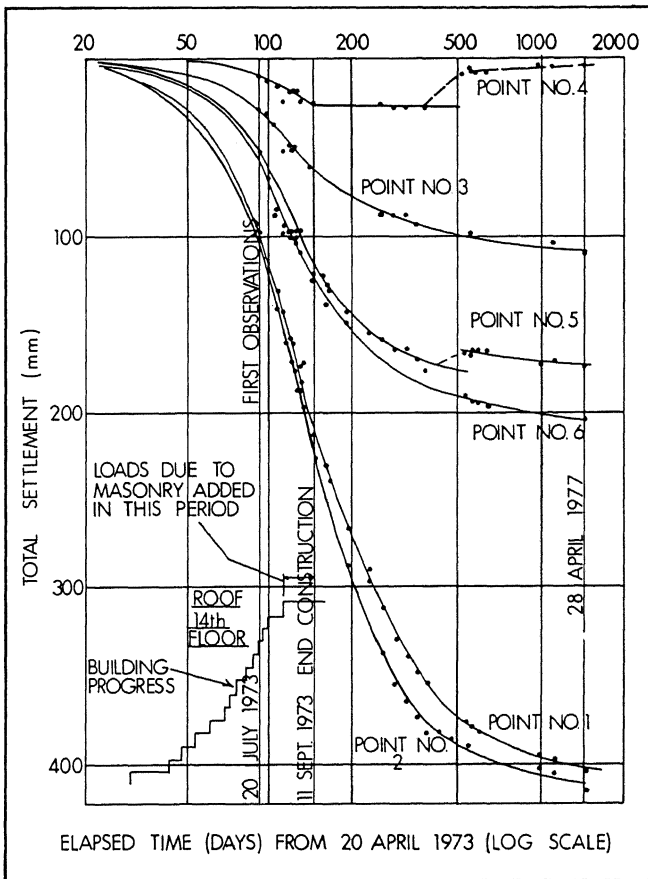


Fig. 5. Settlement Records

If it is possible to construct a settlement contour from six points, Fig. 6 provides a reasonable proposal. It is of interest to note that the settlement began virtually as construction commenced, hence the lower portion of the building settled significantly more than the roof. Fig. 7 provides an indication of the settlement for the roof and second floor. These three figures, 5, 6 and 7, summarize the performance of the building. Table I summarizes the measurements at each point.

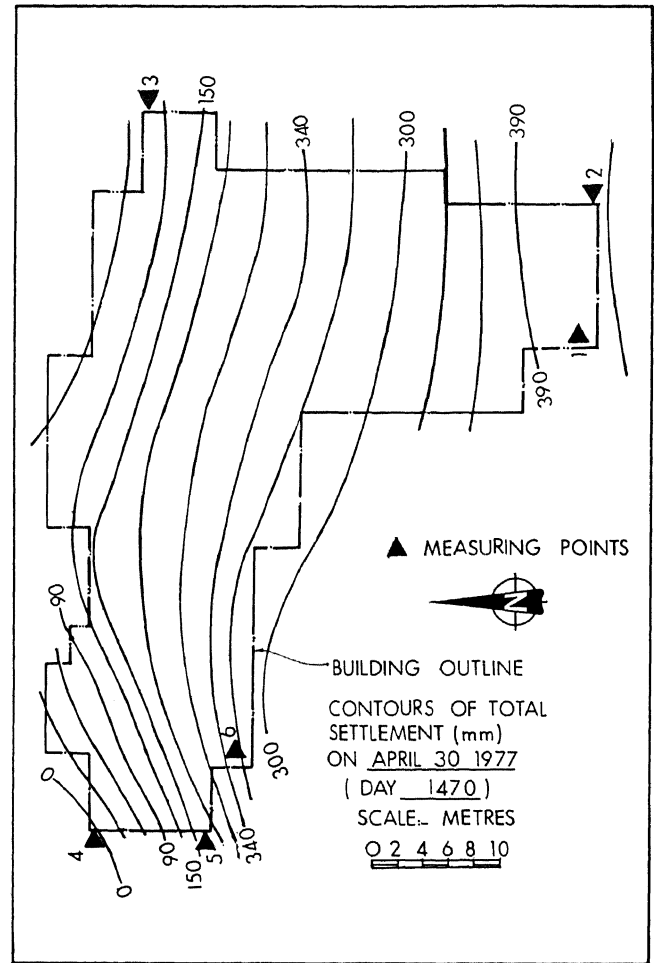


Fig. 6. Contour Drawing of Building Settlement

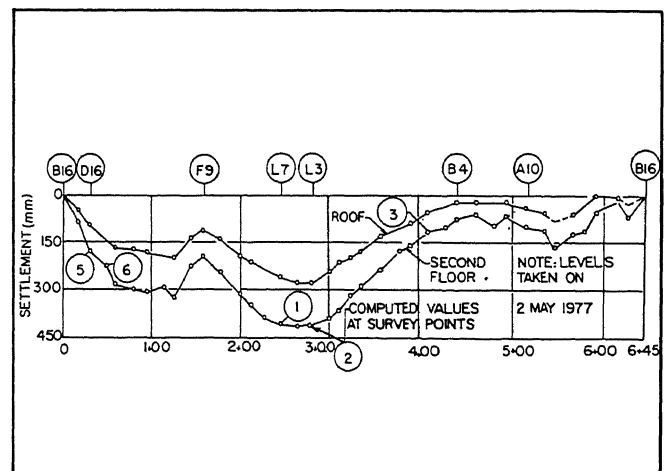


Fig. 7. Settlement of Second Floor and Roof Around Building Perimeter

TABLE I. Maximum Settlement vs Depth of Clay

Point	Settlement (mm)		(m)
	Maximum Settlement	Roof Settlement	Depth of Clay
1	412	244	2.8
2	421	274	2.8
3	113	61	2.2
4	30	0	0.0
5	180	91	0.8
6	204	152	0.8

It is worth noting that for all settlement measurements the actual total settlement is unknown. The curves shown on Fig. 5 have been projected/calculated in reverse from the date of the first measurement, i.e. July 20, 1973 to the beginning of construction of spread footings on April 20, 1973. The settlement picture is complicated by the fact that 1.2 to 1.8 m of sand fill was placed in the building area in May or June of 1973. This fill accelerated the settlement in the early stages.

The building was occupied by early 1974 and the owner/builder stopped taking settlement readings by May 1, 1974. The soil consultant continued having the readings taken until May, 1977.

Until the completion of the building and its occupation, the relationship between owner/builder, architect, engineer, and geotechnical consultant, was excellent. Close co-operation with all parties had enabled the building to be completed on time and without major problems related to the settlement. Careful observation of all portions of the building, plus the continued monitoring of the settlement, provided confidence to all the responsible parties that the building was and would continue to be satisfactory.

During February 1975 the owner proceeded with a court order against "all" parties claiming damages. From this point until the end of the building's life, scores of experts were involved in analysis, investigation, appraisal, etc. The final order to evacuate the building came on January 5, 1978 when the North Bay building inspector ordered evacuation. A dramatic ending took just 7 seconds on July 4, 1982 when Greenspoon Bros. Limited utilized the implosion principal to level the building. Fig. 8 and Fig. 9.

Before reviewing the actual verdict that caused the demise, some brief comments related to the settlement and the causes are in order.

Fig. 6 indicates a best guess at the settlement contours for the building. As drawn they show a uniform change in settlement across the building. Based on crude estimates, this project just over 300 mm of settlement across 43 m of structure.



Fig. 8. Building Just Prior to Demolition



Fig. 9. Building Just After Demolition

When compared with values of Bjerrum, L. (1963) this does not seem alarming. This interpretation of the settlement helps to clarify the condition of the building - which remained excellent with little sign of major distress. Those with a keen eye reportedly could detect a lean to the south wall, this visual perception when related to the work of Skempton, A.W. and MacDonald, D.H. (1956) tends to confirm their findings if we assume a building approximately 45 m in height.

What was the effect of the site improvement? The foundation contractor had specified "points" taken to bedrock. If these points were in fact stone columns (and later review shows this contractor used 200 tonnes of stone in the vibratory process), it is possible to estimate that the settlement would not have been as great as measured. In fact, had the spread footings been placed in the sand without the improvement, the settlement would have

been of less magnitude. It can be postulated that the foundation contractor utilized a mixture of vibro compaction and vibro replacement to increase the density of the soil. In doing so, little regard was placed on what was happening to the ground. It is probable that the vibratory probe caused considerable disturbance to the silty clay without constructing a suitable stone column. Why the contractor failed to advise anyone that the probe depths had increased well beyond the average of 5.2 m given in his contract is a mystery. One of the retained experts stated "It is surely not too much to expect that this subcontractor should have noticed the wide divergence from the anticipated conditions during his performance of the work".

The clay layer was without doubt the culprit in the case. The thickness of this layer, as indicated in Table I, is significant so why was it not encountered in the original site investigation? In the area of the original boreholes the depth to bedrock (refusal) was 4.3 m for two of the holes and in these areas no clay of significance is noted. The other two of the original boreholes went to 5.8 and 6.1 m depth but no samples were recovered from the lowest level - they were lost. Since the original design called for lightly loaded spread footings near ground surface and the modified 14 storey foundation was specified as piles, presumably the consultant felt this sample loss was not of a major importance.

If it is assumed that some form of stone column of inadequate construction and depth was left by the foundation contractor, it is easy to envisage a load transfer to these columns and overstressing of the lower clay layer. The overstressed clay could be expected to react in a manner similar to the load settlement curves in Fig. 5. This should not be assumed to be the simple solution. Many experts spent many dollars calculating, adjusting, factoring and plotting to get conventional elastic consolidation theory to fit these curves.

In the end the claim against all parties involved in the construction triggered not only geotechnical investigations but also structural re-analysis. This structural re-analysis showed that no allowance had been made for earthquake forces. This consideration was specified in a 1970 building code and its absence, when combined with the settlement problem and other problems that arose during close scrutiny, resulted in the end of the building.

From a geotechnical consideration had the building been saved? People had occupied the building for 3 years. Maintenance was not a significant problem. Fig. 5 indicates the performance. Could it be said that the operation was a success but the patient died?

To individually acknowledge all of the personnel involved would be difficult and possibly not desirable. At least let it be known that the work was essentially done by many contributors from several countries.

## CONCLUSION

A settlement problem resulted from an inadequate site investigation combined with the vibroflotation process. Careful site measurement of the settlement provided confidence that the building was safe. Litigation revealed other problems that combined with the settlement problem to cause the demolition of a 14 storey building.

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