GEOTECHNICAL

ENGINEERING

GEOTECHNICAL ENGINEERING

Papers: **Page**

SOME LAND SUBSIDENCE EXPERIENCES IN JAPAN AND THEIR RELEVANCE TO SUBSIDENCE IN BANGKOK, THAILAND†

Toshinobu Akagi* ·

SYNOPSIS

The development and conditions of land subsidence in three Japanese cities have been reviewed in an attempt to extract some useful means to cope with land subsidence problems arising in Bangkok, Thailand. The geological environment of Bangkok and the present pumping rate and decline of groundwater levels strikingly resemble the conditions of some Japanese cities when they were experiencing disastrous land subsidence. Although the current rate of subsidence and the total subsidence appear to be much less in Bangkok, it is certain that subsidence will continue and the flat, low-lying area will eventually lose its scant freeboard above sea level unless some groundwater control measure is taken urgently.

INTRODUCTION

This paper reviews and summarizes some of the land subsidence experiences in Japan accumulated in the past half a century. It is an attempt to share these experiences with those concerned with problems arising from heavy depletion of ground water resources in areas where the geological conditions are alike, e.g., the Greater Bangkok area in Thailand. It took many years for Japanese investigators to pinpoint the major cause of land subsidence and more importantly, it took more costly years before the cause was generally accepted as such and accordingly effective counter-measures were taken.

Even after the mechanism of land subsidence was elucidated in Tokyo and Osaka by the 1940's, heated arguments were repeated about the same problem in other areas in Japan. This is at least in part because the local authorities responsible for vital problems, such as providing the public with adequate water supply, are always involved in the use of groundwater resources. In addition, some industrial and commercial concerns have their own vested interests. These groups raised strong opposition to the view that their activities constituted the principal cause of land subsidence on the grounds that land subsidence was a complex problem caused by many

---

factors and that the geological conditions as well as others were so unique the experiences obtained elsewhere were not applicable to their particular locality.

During the ensuing lengthy discussions the ground continued to sink at alarming rates. Consequent loss of elevations of as much as a few meters occurred before any positive measures could be taken, and resulted in substantial lowering of low-lying areas permanently below sea level. It should be noted that once lost, ground never comes back to its original elevation and only several centimeters of the lost ground may heave up even though the groundwater level regains a few tens of meters.

Thus the same mistakes were repeated time and again in many localities in Japan. Extensive instrumentation and long-term observational networks were required to show convincing evidence in each area that there was an undebatable distinct relationship between the subsidence and the groundwater levels which in turn depended almost solely on the rate of pumpage in the area. That is, subsidence was caused by human activity, not natural causes.

Based on the experience that gradual subsidence due to natural causes such as tectonic movements never exceeds an annual rate of a few millimeters, land subsidence is defined generally in Japan as the subsidence of the ground surface which takes place at an annual rate having an order of magnitude 10 to 100 times as great as that created by nature, i.e., a few to a few tens of centimeters a year, occurring in a relatively limited area up to several hundred square kilometers, and which is therefore attiributable to man-made causes.

Fig. 1. Areas of land subsidence in Japan, 1975 (after Yamamoto, 1976).

YAMAMOTO (1976) reported that some 50 areas

2

in Japan totalling 7,380 km² have experienced noticeable subsidence at a rate exceeding 2 cm/year with a total of 1,200 km² of the areas being already below sea level (Fig. 1). The following describes some of the experiences obtained in three representative areas in Japan; Tokyo and Osaka where the oldest records of land subsidence and the results of extensive investigations are available and the problem now appears to be fairly well under control, and Niigata, where acute subsidence took place relatively recently in spite of the availability of bitter experiences which had been accumulated elsewhere in Japan.
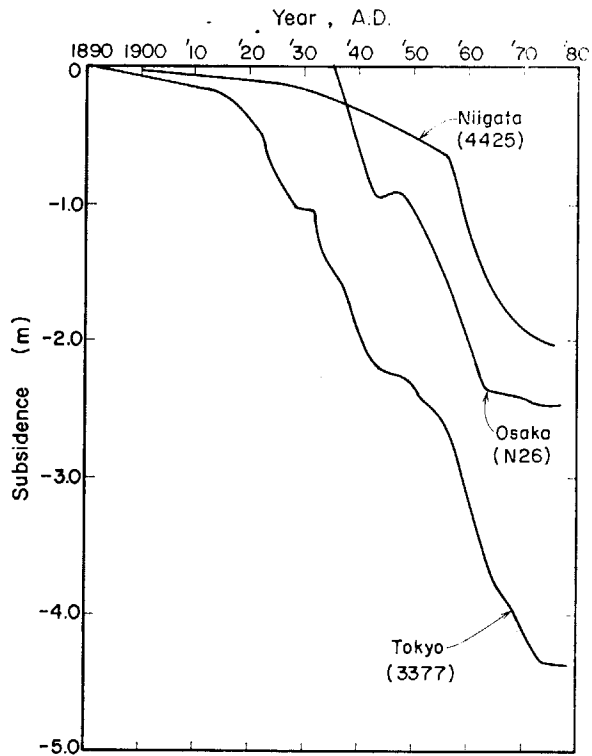
Figure 2 shows how land subsidence has progressed and

Fig. 2. Land subsidence: Records of bench mark settlements in Tokyo, Osaka and Niigata.

has eventually been brought under control in the three cities. It demonstrates that what has been caused by man can at least be controlled by him.

GEOLOGICAL CONDITIONS IN TOKYO AND OSAKA

The Tokyo Metropolis is situated in the southern part of the Kanto Plain. Its eastern part, facing Tokyo Bay, lies in an alluvial plain (the Alluvial Lowland in Figure 3) through which three major drainage channels: Sumida, Ara and Edo Rivers, flow forming a large delta known as Kohto Area at the mouths of Sumida and Ara Rivers. The western part, about 20 m higher in elevation, is located in Musashino Upland which is mantled by Kanto loam strata principally of volcanic origin, and is bounded by Tama River on the south.

Figure 4 gives a general geologic section along line A-A' as indicated in Figure 3. Table 1 is a summary of the stratigraphy and the major soil groups defined in the area. While a great deal of information has been accumulated on the soil strata of the Recent age (e.g., NAKANO et al, 1969

3

METROPOLITAN TOKYO

| TAMA AREA | 23 WARDS , TOKYO |
| MUSASHINO UPLAND | ALLUVIAL LOWLAND |

SAITAMA PREFECTURE

KOHTO AREA

CHIBA
PREFECTURE

TOKYO BAY

S = SUMIDA RIVER
A = ARA RIVER
E = EDO RIVER
T = TAMA RIVER

KANAGAWA
PREFECTURE

0   5 km.

N
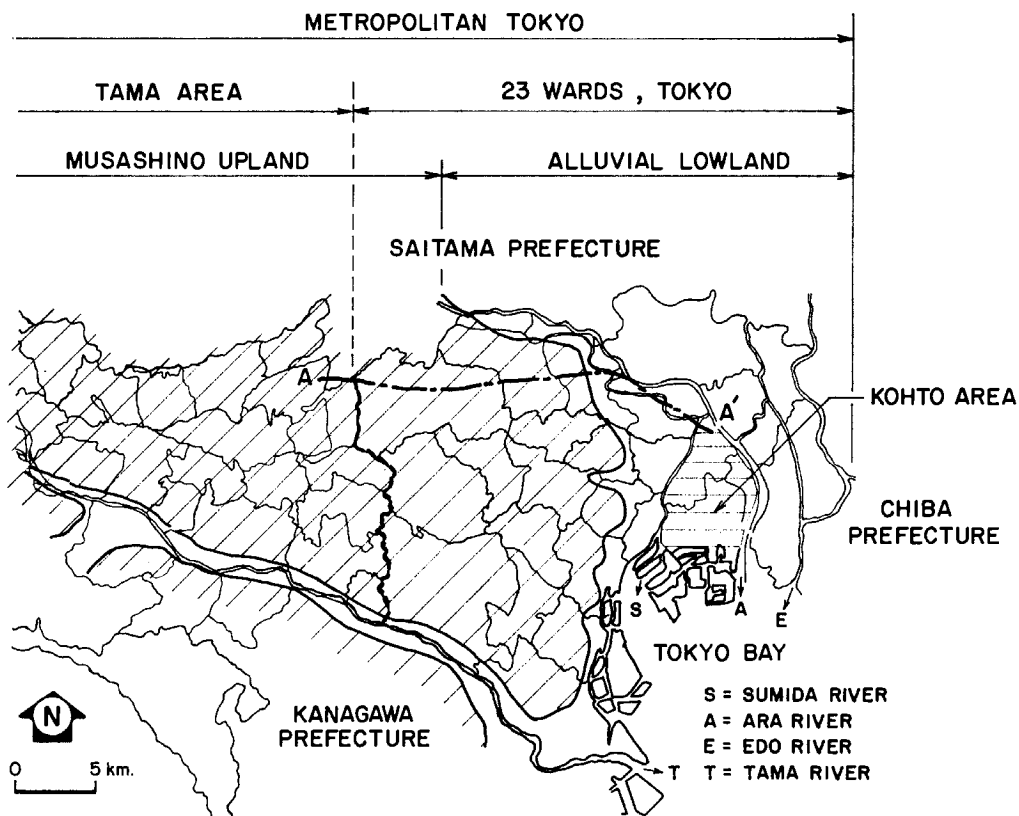
Fig. 3. Topographical and areal divisions of Metropolitan Tokyo (after Tokyo Metropolitan
Government, 1976).

Table 1. Stratigraphy in the Tokyo area (after Tokyo Metropolitan Government, 1976).

	Musashino Upland	Alluvial Lowland	Thickness (m)	Soil Types
Holocene		Yūrakucho Formation	10-30	Soft clay
		Nanagōchi Formation	10-30	Sand & clay
Pleis-tocene	Tachikawa loam Tachikawa gravel Musashino loam Musashino gravel		10-15	Volcanic ash Terrace gravel Volcanic ash Terrace gravel
	Tokyo Group		50-450	Sand, clay and gravel
Pliocene	Kazusa Group		>1000	Upper: Sand Lower: Sandy Mudstone

4

TPWRI, 1969) relatively few data are available on the soil strata deeper than a depth of approximately 300 m. The Tertiary deposits consist primarily of mudstones and extend to more than 2,000 m below the ground level in the Kohto delta, and to more than 1,000 m in the central part of the Musashino Upland. The upper boundary of the Kazusa Group is well established, but its lower boundary is yet to be determined.

Fig. 4. Geologic section across southern Kanto Plain (after M. Ishii, 1977).

Several artesian aquifers are present in the Tokyo Group and in the upper Kazusa Group. The Tokyo Group contains sand and gravel aquifers yielding abundant groundwater of good quality in general, which has in fact been extensively exploited. In the southern part of the Alluvial Lowland, however, groundwater of the Kazusa Group has high ion contents of chlorine and other soluble elements. It is also in this southern area where the groundwater contains soluble natural gas in a thick sand layer of the upper Kazusa Group at depths ranging from 450 to 650 m along Tokyo Bay. The gas-bearing water has a high chlorine ion content which, however, decreases rapidly with increasing distance from the shore. In the northern part of the Upland area, a water-bearing sand layer as thick as 250 m forms the top of the Kazusa Group and serves as a rich groundwater reservoir.

□ ALLUVIAL PLAIN

▨ DILUVIAL UPLAND

Fig. 5. Topography of Osaka Plain (after Kaido, 1975).

Osaka, the second largest city in Japan, is also situated on an alluvial plain (Fig. 5) facing Osaka Bay on the west and bounded on the east by the Ikoma mountain range. The Yodo River runs through the plain and empties into Osaka Bay. The geologic condition is roughly illustrated by an eastwest section (Fig. 6), which was constructed on the basis of the results of the deep borings made in connection with the land subsidence studies.

5

Fig. 6. Geologic section across Osaka Plain (after Kaido, 1975).

Geologically the city of Osaka is divided into the eastern and the western parts by Uemachi Terrace. This terrace is about 20 m above sea level and trends in a north-south direction. The plain is generally covered by thick alluvial dep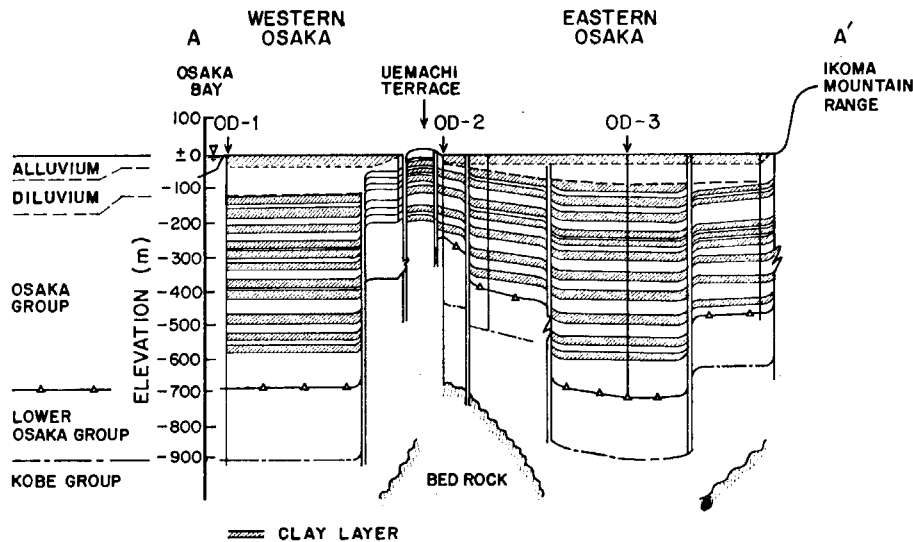osits consisting of soft clay and some sand which thicken to 30 to 35 m near the bay and thin out toward the terrace area. The diluvial* deposits of sand and gravel and some clay layers underlie the soft surface layer, and in turn overlie the Quaternary deposits termed as the Osaka Group which consist of alternating layers of sands and clays. The Lower Osaka Group is considered as a Tertiary deposit, and is underlain by an older deposit locally known as the Kobe Group (MURAYAMA, 1969). The bedrock of granite was encountered at the depth of 667 m in Uemachi Terrace, OD-2, but a test hole near the bay 907 m deep, OD-1, did not reach the bedrock which, however, was estimated to lie at a depth of approximately 1,500 m (Fig. 6).

In summary the two largest cities in Japan are both located near a bay and on an alluvial plain consisting of thick strata of unconsolidated soils of Quaternary and Tertiary ages. The strata include several aquifers of high yielding capacity. All the soils have been found to have compressibilities of varying degrees and in fact can be compressed due to the effective stress increases which may be caused by gradual lowering of groundwater

---

* In Japan diluvial refers to compacted clay, silt, sand, gravel or their mixtures that have a characteristic stiffness.

levels, leading to significant subsidence of the ground surface. Similar geological conditions prevail in other major subsidence areas in Japan, generally situated on alluvial plains, e.g., Chiba, Saitama, Nagoya, Niigata, Saga, etc.

HISTORICAL REVIEW OF LAND SUBSIDENCE IN JAPAN

Although it was not until the 1930's that any research-oriented studies directly related to land subsidence were initiated in Japan, it had earlier been pointed out on the basis of the survey data taken periodically that the ground was indeed sinking at a noticeable pace. In 1915 T. TERADA, a well-known physicist, warned that some bench marks in the Kohto delta area, Tokyo, had settled a maximum of 0.75 cm a year (SHIBAZAKI, 1971). In connection with his study on the Great Kanto earthquake in 1923, A. IMAMURA, a seismologist, reviewed the survey results and concluded that the maximum settlement must have been a staggering amount of 63 cm for a period between 1923 and 1926 and another 56 cm between 1927 and 1930 in the Kohto area. He attributed the extraordinary ground sinking to movements of active faults associated with the destructive 1923 earthquake and another strong quake in 1928 (M. ISHII, 1977).

In 1931 an explosion resulting from methane gas erupting out of an old well in the Kohto area attracted the attention of the City of Tokyo officials. Suspecting that the gas eruption might have something to do with the ground movements, they conducted a series of level surveys and found out that the area had settled 10 to 17 cm a year. In fact the maximum subsidence rate in Tokyo, 41.9 cm per year, was recorded in this area for a period of approximately a year between 1932 and 1933 (M. ISHII, 1977).

Figure 7 shows the settlements of some principal bench marks in Tokyo. This, together with groundwater level records and Table 2, gives the pertinent data. It is clearly demonstrated that land subsidence was already in progress in the 1910's at an alarming rate in the Kohto area (Bench Marks 3377 and 9832). Subsidence gradually crept upstream in the area sandwiched by Ara and Edo Rivers where it became evident by the 1920's that the area was subsiding at an unusual pace (Bench Marks 9836, 3367 and 3365). In the postwar years the land subsidence penetrated further into the upstream areas along Ara River and into the Upland area, Kita and Itabashi Wards (Table 2).

Soon after the Great Kanto Earthquake in 1923, G. KITAZAWA, an architect, noticed that the buildings supported by deep foundations were being

7
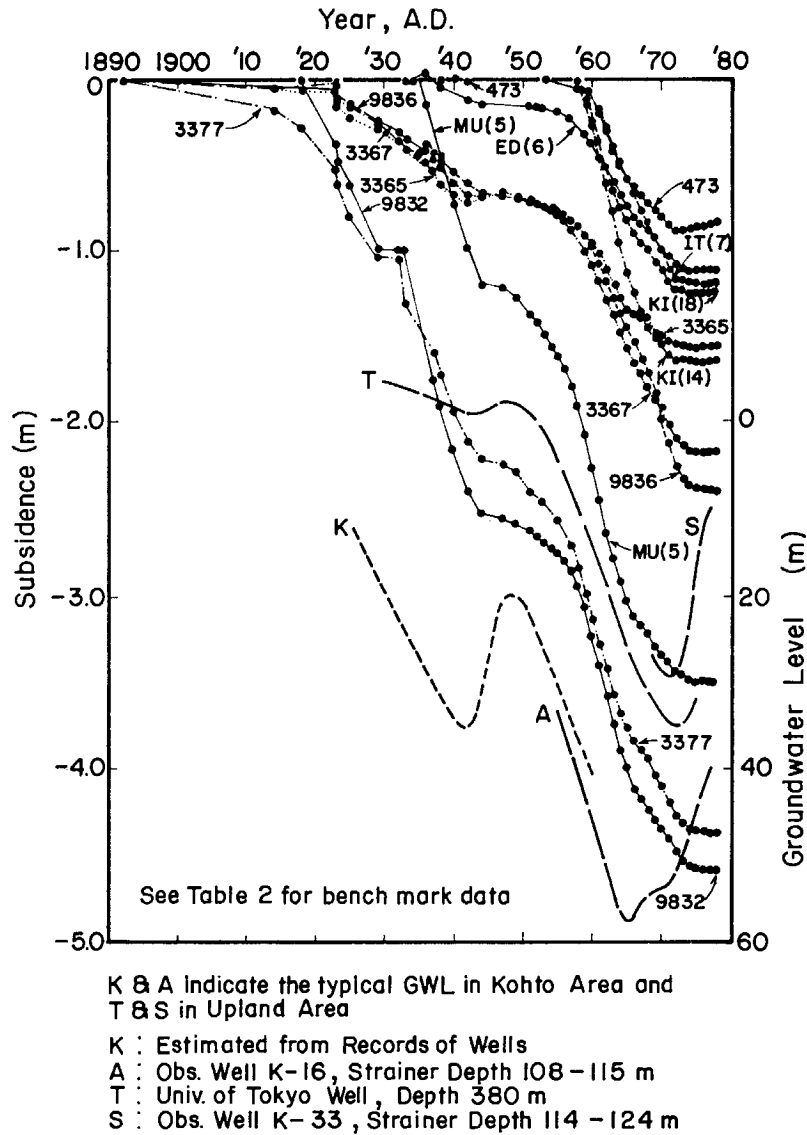
Fig. 7. Settlements of principal bench marks and groundwater levels in Tokyo, 1892-1978 (after TPWRI, 1978),

lifted up with respect to the ground surface in the Marunouchi district, Tokyo and considered that this phenomenon was due to the compression of soft clays at shallow depths. Based on this idea N. MIYABE developed the first subsidence indicator and installed two of them in the Kohto area in 1933. His instrument consisted essentially of a steel pipe, 10 cm in diameter and 35 m long, embedded about 5 m into the firm soil. The protrusion

8

Table 2. Some principal bench marks in Tokyo (after TPWRI, 1978).

Bench Mark No.	Location (Ward)	Total Settlement(m) Since Installation. Measured in 1978.	Installation Date
9832	Kohto	4.57	1918
3377	Kohto	4.36	1892
MU(5)	Sumida	3.50	1935
9836	Edogawa	2.40	1918
ED(6)	Edogawa	1.10	1934
3367	Katsushika	2.17	1892
3365	Adachi	1.54	1892
KI(14)	Kita	1.63	1958
KI(18)	Kita	1.24	1958
IT(7)	Itabashi	1.19	1958
473	Itabashi	0.84	1933

Bench Mark No. corresponds to the number indicated in Fig. 7.

of the pipe above the ground surface was magnified 40 times by a lever system connected to an automatic recorder. Although he made it clear that the subsidence was not the result of tectonic movements but that of the compression taking place in the surface layer, he was more concerned with other factors rather than the groundwater level, which however, was measured at a depth of several meters.

B.M. No.	Location
0	Minato Ward
231	Nishi Ward
231.1	Minato Ward
235.1	Naniwa Ward
237	Kita Ward

GWL based on data from Kujo Obs. Well up to 1970 & also from Minato C up to 1976 N-5, E-7, W-45, W-13 & K-26 indicate bench marks in Osaka

Fig. 8a. Settlements of principal bench marks in Osaka, 1885-1947 (after Nakamachi, 1977).

Fig. 8b. Settlements of principal bench marks and groundwater levels in Osaka, 1935-1976 (after Nakamachi, 1977).

In Osaka also, it was an earthquake in 1928 that first brought land subsidence to the attention of the specialists. A. IMAMURA, suspecting that the unusual ground sinking was associated with another strong quake in the future, prompted the city officials to review the past survey data and to initiate an intensified survey network throughout the city. It was then revealed that the bench marks in the western Osaka had started settling as early as the 1890's and the rate of settlement had been accelerated drastically since around 1930 (Fig. 8a). For the following decade or so till around 1942 the annual rate of subsidence ranged from 10 to 20 cm in the area near Osaka Bay.

In 1934 a powerful typhoon hit the Osaka area, inundating one third of the city. Coupled with the high tide this resulted in muddy sea water invading the western part, which coincided with the area where subsidence had been notably in progress. This disaster led the government to establish the Disaster Prevention Research Institute in Osaka in 1936 with land subsidence as one of the priority themes.
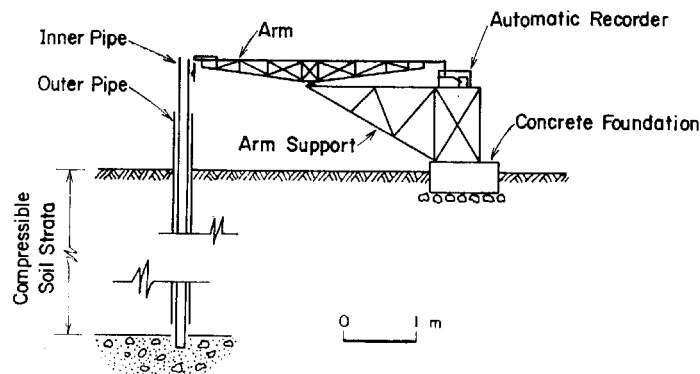
Fig. 9. Double-tube subsidence indicator installed in Osaka, 1938 (after Wadachi and Hirono, 1939).

It was there in 1938 that K. WADACHI installed a subsidence indicator using a double-tube system with the inner pipe reaching the depth of 36 m (Fig. 9). Also the City of Osaka installed three steel pipes to the depths of 33, 62 and 176 m in Kujo Park, and in 1939 started measuring continuously compressions in the soil strata of different thicknesses and the groundwater levels at different depths. Based on the continuous records thus obtained, WADACHI & HIRONO (1939, 1942) found an intimate relationship between soil compressions and groundwater levels and concluded that land subsidence was essentially the consolidation of surface layers induced by drop of groundwater pressures for which excessive pumping was responsible. WADACHI

10

(1940) found a good correlation between the groundwater level and the subsidence rate calculated over a 5-day period (Fig. 10) and presented the first quantitative expression for land subsidence:

$$-\frac{dH}{dt} = k \ (p_0 - p) \ \dots\dots\dots\dots\dots\dots(1)$$

in which dH/dt is the rate of land subsidence, k a constant related to soil conditions, p the pressure of groundwater, p_0 the standard pressure of groundwater which normally corresponds to the pressure at a depth of ten plus several meters. HIRONO (1972) later pointed out Equation 1 did not contradict TERZAGHI's consolidation theory.
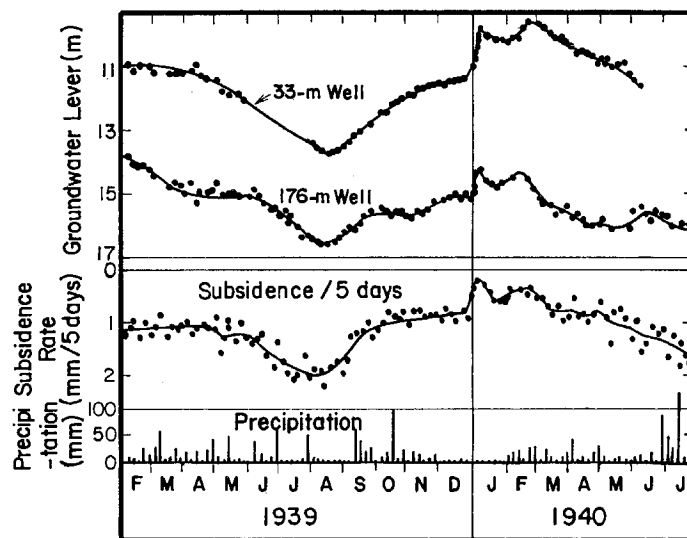
Fig. 10. Relationship between groundwater levels and subsidence rates obtained in Kujo Park, Osaka (after Wadachi, 1940).

OKA (1945) made a study on the mechanism of land subsidence extending TERZAGHI's theory of consolidation and the data obtained in Tokyo. In Osaka a group led by Y. ISHII (1949) initiated in 1946 a comprehensive study which included a soil mechanics analysis, a systematic subsurface and hydro-geologic investigation, extensive installation of instruments monitoring subsidence and groundwater levels and recommendation of countermeasures.

While Tokyo and Osaka were practically reduced to dust and ashes toward the end of World War II, 1943 to 1945, and all the production activities virtually brought to a standstill, immediately thereafter, 1945 to 1947, the groundwater level showed a dramatic rise and in actuality land subsidence came a halt (Figs. 7 and 8b). In fact the ground heaved a few

11

centimeters a year at many locations in the both cities with the maximum heave of 5.0 cm recorded in Osaka in 1946.

Ironically, this remarkable phenomenon proved to be an unexpected grand-scale field test which brought a convincing evidence to support the theory that the excessive depletion of groundwater was the principal cause of land subsidence. This brief pause, however, turned out to be merely a prelude to another disastrous period of land subsidence in Tokyo and Osaka as the post-war economy boomed into full swing (INABA et al, 1969).

ARGUMENTS ON THE CAUSE OF LAND SUBSIDENCE

In the preceding brief run-down on the early stage of the land subsidence history in Japan, one may get a notion that all the problems had been solved by the late 1940's. Historically it may be stated that the mechanism of land subsidence had been elucidated by then. It is also true that considerable controversy about the cause of land subsidence still existed even among the specialists at that time and consequently delayed enforcement of any countermeasure to prevent further subsidence.

According to SHIBAZAKI (1971), some 80 theories, most with no positive proof, were presented by various investigators in the 1930's. WADACHI & HIRONO (1939) listed the following as the major theories then being proposed to account for the phenomenon of land subsidence:

    (1) tectonic movements,

    (2) compression of soil strata under their own weight,

    (3) compression of soil strata due to the weight of heavy structures and fills: i) elastic deformation, ii) plastic deformation, iii) flow deformation and iv) consolidation settlement,

    (4) compression of reclaimed soil,

    (5) decrease in gas pressure in soil,

    (6) decrease in percolation of rainfall into the ground because of the increase of covered areas and provision of drainage systems,

    (7) pumping of sand out of aquifers and disturbance given to soil structures in aquifers,

    (8) loss of clay from clay layers eroded by the groundwater flow in aquifers,

    (9) compaction of soil strata due to traffic and earthquakes, and

    (10) compression of soil strata due to lowering of groundwater level.

Even the last item (10) was discussed in different terms such as (i) increase in electrolytes, (ii) decrease in buoyancy, (iii) shrinkage of soil due to drying and (iv) consolidation due to increase in effective stresses. In addition to the above, some researchers spent years studying the possible effects of tidal fluctuations and even changes in atmospheric pressures.

With the dramatic evidence demonstrating a definite relationship between the subsidence and the groundwater level obtained in Tokyo and Osaka during and immediately after the Second World War, 1943 to 1947, an increasing number of specialists came to realize that excessive pumping which drastically lowered the groundwater level was responsible primarily for the land subsidence. As the post-war economic and industrial activities recovered and boomed, however, the consumption of groundwater soared, the groundwater level fell sharply and subsidence accelerated its pace for about the next 15 years (Figs. 7 and 8b). Land subsidence continued in spite of the fact that many specialists were fully aware of what should have been done particularly in connection with damaging floods frequently attacking the rapidly sinking, low-lying areas in Tokyo and Osaka which had resulted from high tides and typhoons.

As a matter of fact, in the late 1950's when subsidence was dramatically accelerated in Niigata (Fig. 2), lengthy heated arguments were again initiated about the cause of subsidence there. Aoki (1975) summarized various causes debated by the parties involved during the investigation conducted between 1957 and 1960 as follows:

(1) rise of the sea level due to melting of the polar ice in accordance with the recent world-wide trend of temperature rise,

(2) general subsidence trend along the Sea of Japan,

(3) unique tectonic movement in the Niigata area,

(4) natural consolidation process of the relatively new Niigata alluvial deposits,

(5) erosion of beaches due to wave actions,

(6) seaward movement of soft soil due to the dredging in the harbour,

(7) shrinkage of ground because many rice paddies had been converted into dry fields, and

(8) excessive depletion of groundwater for natural gas production.

During much debate and an extensive investigation for the 3 year period that followed, however, the downtown Niigata lost well over 1 m of elevation with a substantial portion of it brought below sea level. It was there during

13

Even the last item (10) was discussed in different terms such as (i) increase in electrolytes, (ii) decrease in buoyancy, (iii) shrinkage of soil due to drying and (iv) consolidation due to increase in effective stresses. In addition to the above, some researchers spent years studying the possible effects of tidal fluctuations and even changes in atmospheric pressures.

With the dramatic evidence demonstrating a definite relationship between the subsidence and the groundwater level obtained in Tokyo and Osaka during and immediately after the Second World War, 1943 to 1947, an increasing number of specialists came to realize that excessive pumping which drastically lowered the groundwater level was responsible primarily for the land subsidence. As the post-war economic and industrial activities recovered and boomed, however, the consumption of groundwater soared, the groundwater level fell sharply and subsidence accelerated its pace for about the next 15 years (Figs. 7 and 8b). Land subsidence continued in spite of the fact that many specialists were fully aware of what should have been done particularly in connection with damaging floods frequently attacking the rapidly sinking, low-lying areas in Tokyo and Osaka which had resulted from high tides and typhoons.

As a matter of fact, in the late 1950's when subsidence was dramatically accelerated in Niigata (Fig. 2), lengthy heated arguments were again initiated about the cause of subsidence there. Aoki (1975) summarized various causes debated by the parties involved during the investigation conducted between 1957 and 1960 as follows:

(1) rise of the sea level due to melting of the polar ice in accordance with the recent world-wide trend of temperature rise,

(2) general subsidence trend along the Sea of Japan,

(3) unique tectonic movement in the Niigata area,

(4) natural consolidation process of the relatively new Niigata alluvial deposits,

(5) erosion of beaches due to wave actions,

(6) seaward movement of soft soil due to the dredging in the harbour,

(7) shrinkage of ground because many rice paddies had been converted into dry fields, and

(8) excessive depletion of groundwater for natural gas production.

During much debate and an extensive investigation for the 3 year period that followed, however, the downtown Niigata lost well over 1 m of elevation with a substantial portion of it brought below sea level. It was there during

13

pre-war momentum which in turn lowered the groundwater level at an annual rate exceeding 1.5 m (Fig. 8b). Osaka continued to sink until the early 1960's and Tokyo until the early 1970's.

In fact the low-lying areas near the bay in the both cities were frequently plagued by prolonged floods particularly when they were hit by violent typhoons which often accompanied unusual high tides. This was in spite of the fact that the extensive dike systems had been constructed to prevent such floods from causing serious damages under the normally expected typhoon conditions. Meanwhile, notable subsidence crept up rapidly into the Musashino Upland in Tokyo and into the eastern part of Osaka. At the same time a number of newly installed observational wells indicated that remarkable compressions were taking place in deep soil strata where previously only the compressions of shallow strata had been predominent.

The present ground surface elevations in Tokyo are shown by a contour map, Figure 11. Figure 12 shows the total subsidence measured during the period between 1938 and 1975, which bears a certain resemblance to
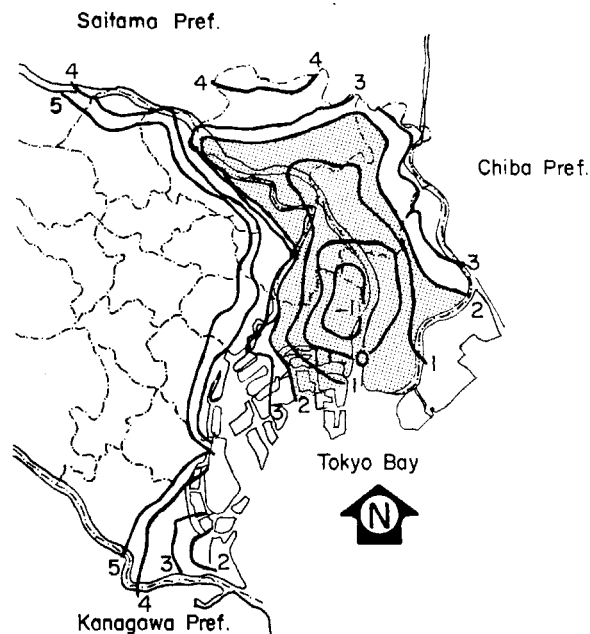
Numbers indicate elevation contours in meter, A.P. A.P. 0 m corresponds to the mean low tide level in Tokyo Bay.

Fig. 11. Elevation contours of ground surface in Tokyo, 1974 (after Tokyo Metropolitan Government, 1976).

15

the present topography. As indicated by the shaded areas in Figures 11 and 12, the area where the total subsidence has exceeded 1 meter for nearly 40 years corresponds roughly to the low-lying area which is presently below the high tide level, A.P. + 2 m.

Saitama Pref.

Chiba
Pref.

Tokyo Bay

Kanagawa Pref.   0   5 km.
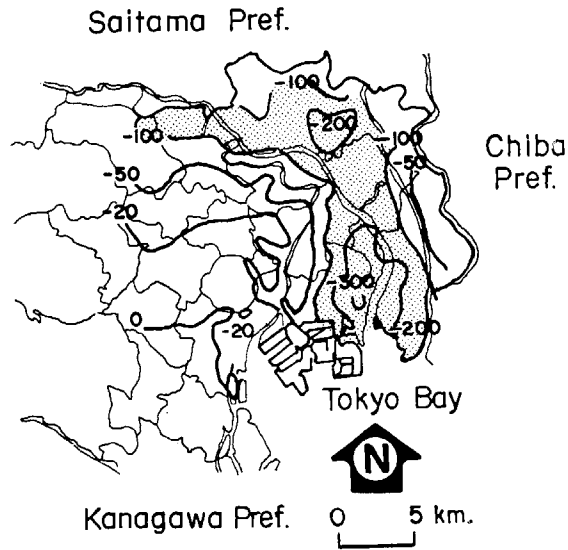
Numbers indicate contours of
subsidence in centimeter.

Fig. 12. Subsidence measured in Tokyo, 1938-1975
(after M. Ishii et al, 1976).

Area where subsidence was
greater than 5 cm./year

Area where subsidence was
greater than 10 cm./year

Fig. 13. Areas in Tokyo where annual subsidence exceeded 10 cm
and 5 cm (based on SKLSIC, 1974 and TPWRI, 1978).

16

In fact the areas where the annual subsidence rate exceeded 10 cm and 5 cm in 1961 totalled, respectively, 74 km² and 173 km² (Fig. 13) showing that, serious subsidence persisted over wide areas in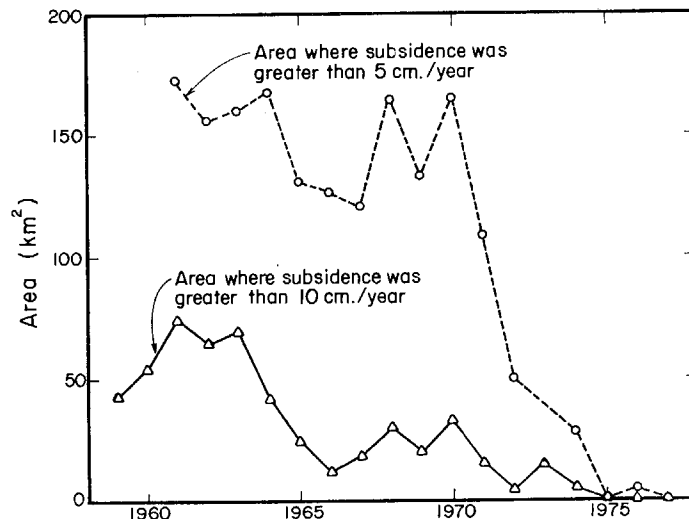 Tokyo in the years that followed, although at a diminishing rate. By 1962 the downtown Tokyo area below the low tide level (A.P. 0 m), the mean sea level (A.P. + 1 m) and the high tide level (A.P. + 2 m) reached, respectively, 11.8 km², 37.1 km²
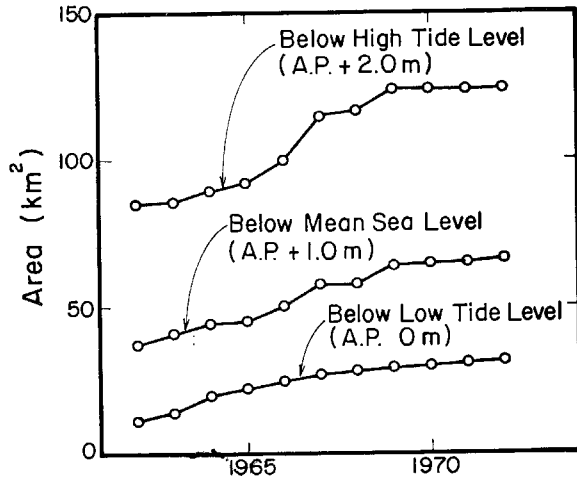
Fig. 14. Areas below sea levels in Tokyo, 1962-1972 (based on SKLSIC, 1974).

Obs.Well	Depth of Strainer	Obs. Well	Depth of Strainer
KO - 2	75 - 80 m	K - 16	108 - 115 m
K - 12	56 - 61 m	K - 22	139 - 144 m
K - 13	42 - 47 m	K - 23	125 - 130 m
K - 14	62 - 67 m	K - 65	35 - 37 m
K - 15	65 - 70 m	K - 66	76 - 87 m

Fig. 15. Subsidence and groundwater levels in Kohto Ward, Tokyo (after TPWRI, 1978).

17

and 85.4 km², and in 1972 amounted, respectively, to 31.5 km², 66.7 km² and 124.2 km², as shown in Figure 14. Since the area of the 23 wards of Tokyo, shown in Figure 3, covers an area of only 572 km², which corresponds roughly to the city proper, it may be stated that more than 10% of the city of Tokyo is located below mean sea level and more than 20% below the mean high tide level. The existence of the sub-zero areas is made possible only by the presence of the extensive dike and drainage systems.

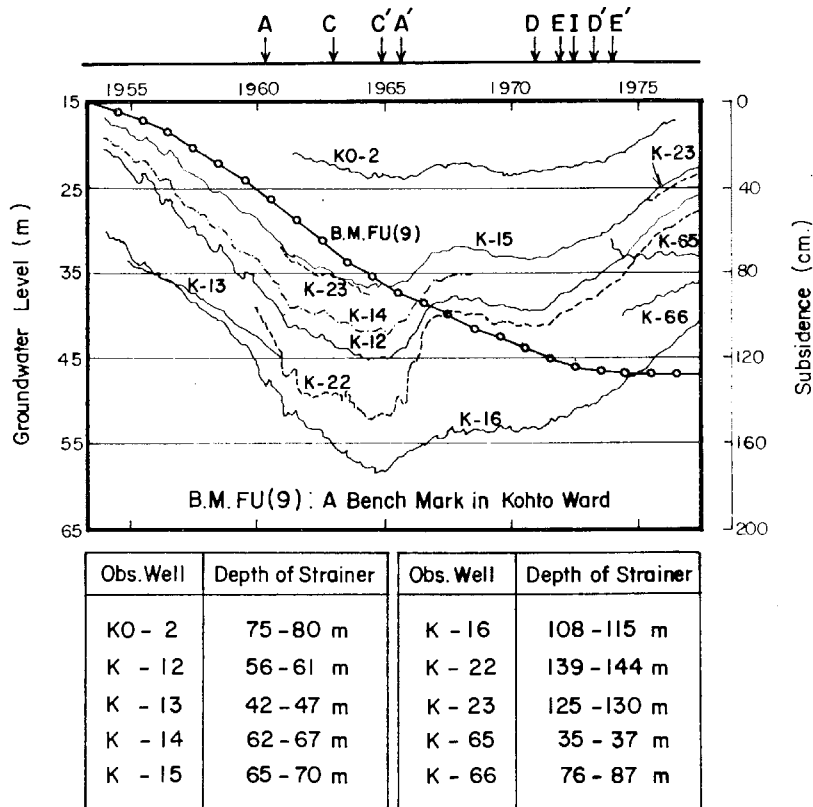Although it was in 1961 when Tokyo first enforced the groundwater control in the Kohto area, it was not until around 1965 that powerful controls were established which were then followed by a series of tightening measures to cover the entire Tokyo area. In response to the restricted use of groundwater, the rise of groundwater levels in aquifers was impressive, accompanied by rapidly diminishing rates of subsidence as earlier shown in Figure 7. Such trends are illustrated in detail by Figure 15 that summarizes some of the typical data obtained in Kohto Ward and also by Figure 16 obtained in Itabashi Ward, some 20 km upstream along the Ara River near the eastern boundary of the Musashino Upland.

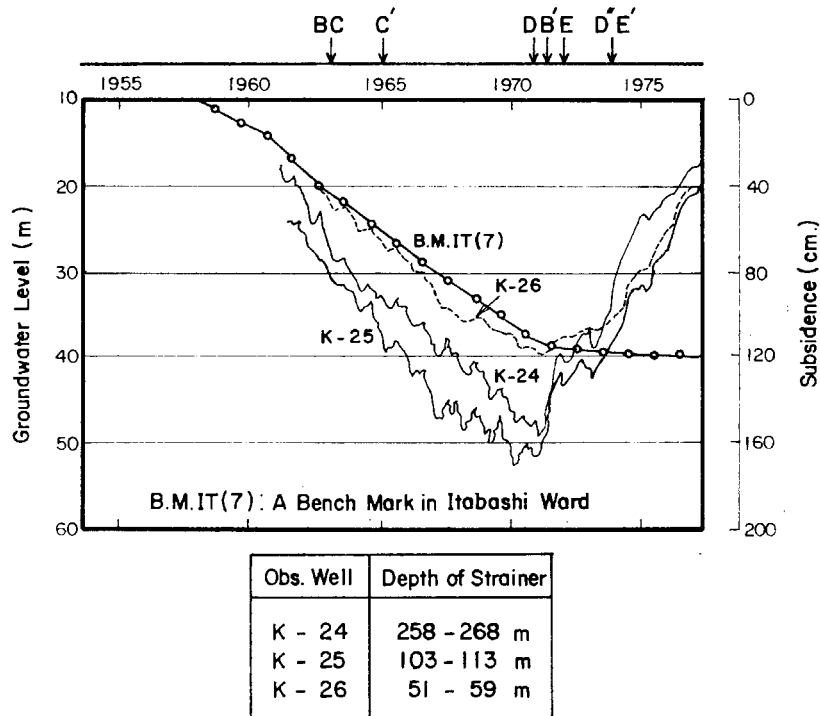| Obs. Well	Depth of Strainer
K - 24	258 - 268 m
K - 25	103 - 113 m
K - 26	51 - 59 m

Fig. 16. Subsidence and groundwater levels in Itabashi Ward, Tokyo (after TPWRI, 1978).

Now that the 1970 ordinance requires every operating well be equipped with a pumpage recorder, it has become possible to maintain a fairly accurate record of the pumped groundwater in areas of Tokyo. Figure 17 gives one such example showing a close relationship between the average daily pumping rate Q and the groundwater level H measured at an observational well, K-24, which has a strainer at the depths between 258 m and 268 m. The straight line relationship between H and Q (Fig. 17) indicates a coefficient of correlation as high as 0.96. Figure 17 also clearly points out the effectiveness of the measures enforced in this area, which will be discussed later.

The data from the observational wells generally indicates that compressions still continue at shallow depths, although at a considerably reduced rate, but that the deep soil strata have ceased compression and have been expanding
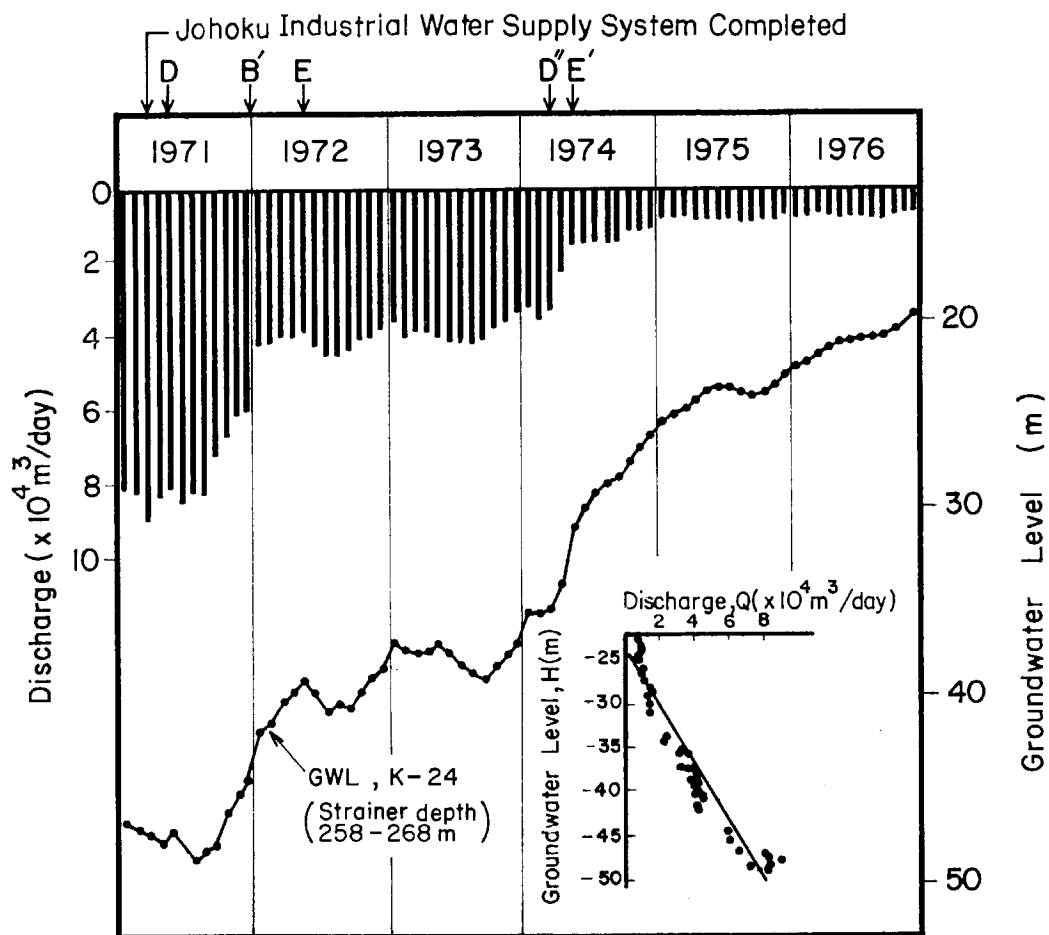

Fig. 17. Discharge by pumping and groundwater level in Itabashi Ward, Tokyo, (after TPWRI, 1778).

19

for the last several years. Figure 18 demonstrates an example obtained in the Kohto area, giving the compressions measured between the ground surface and the depth of 70 m (A), those occurring at the depths greater than 70 m (B), and the total compressions or the subsidence measured at the ground surface (A + B). Photos 1 and 2 are views of ground surface subsidence measuring devices in Japan. Figure 18 also shows the variations of the groundwater level and the average daily discharge by pumping as well as the changes in groundwater level as compared with that recorded a year before (C).
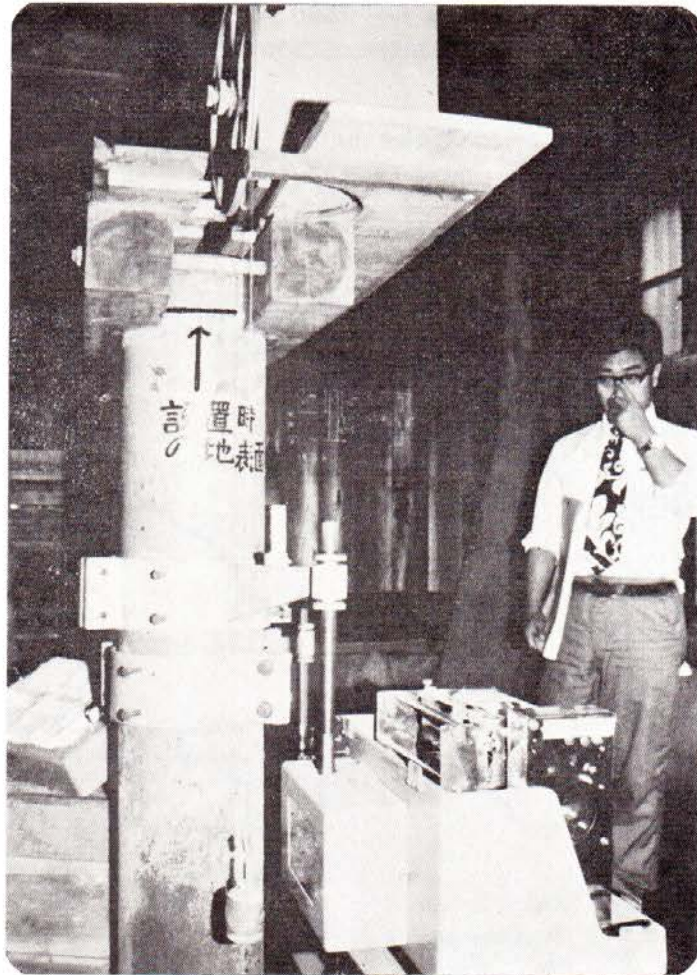

Photo 1. A single-tube subsidence indicator, K-15, at Minami Sunamachi Observation Station, Kohto Ward, Tokyo (70 m deep, installed in 1954). The horizontal line above the arrow on the pipe indicates the ground level at the time of installation. A man is standing on the present ground level, exhibiting a subsidence on the order of 1.5 m. Photo taken in September, 1978.
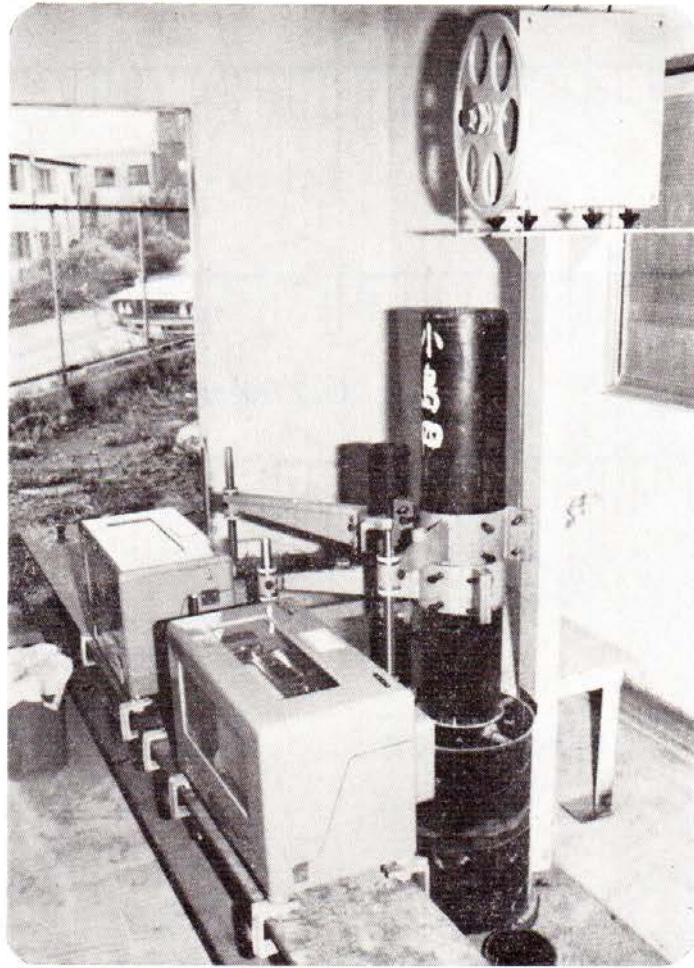
Photo 2. A double-tube subsidence indicator, K-51, at Kojima Observation Station, Edogawa Ward, Tokyo (270 m deep, installed in 1972). The outer tube shows a protrusion of approximately 30 cm. Photo taken in September, 1978.

At this location the subsurface soils, to a depth of 70 m, consist of alluvial deposits and the topmost layer of the diluvium. The shallow layers (A) consolidated as much as 13 cm in 1961 during which time the deeper strata, consisting mainly of sand, were compressed approximately 10 cm. As the rate of drop in groundwater level (C) decreased, although the groundwater level was still dropping, the rate of subsidence (A + B) correspondingly decreased. Such a trend is also remarkably reflected by compression and expansion measured at the depth greater than 70 m (B). A fairly close correlation between (A + B) and GWL in Figure 18 is essentially the same as the one given in Figure 10 and appears to support the validity of the WADACHI's formula (Eqn. 1).

21

Fig. 18. Compressions and expansions in soil strata, groundwater level and pumpage in Kohto Ward, Tokyo (after TPWRI, 1978).

As shown in Figure 19, where a few observational wells were installed to different depths at the same station, it is possible to evaluate 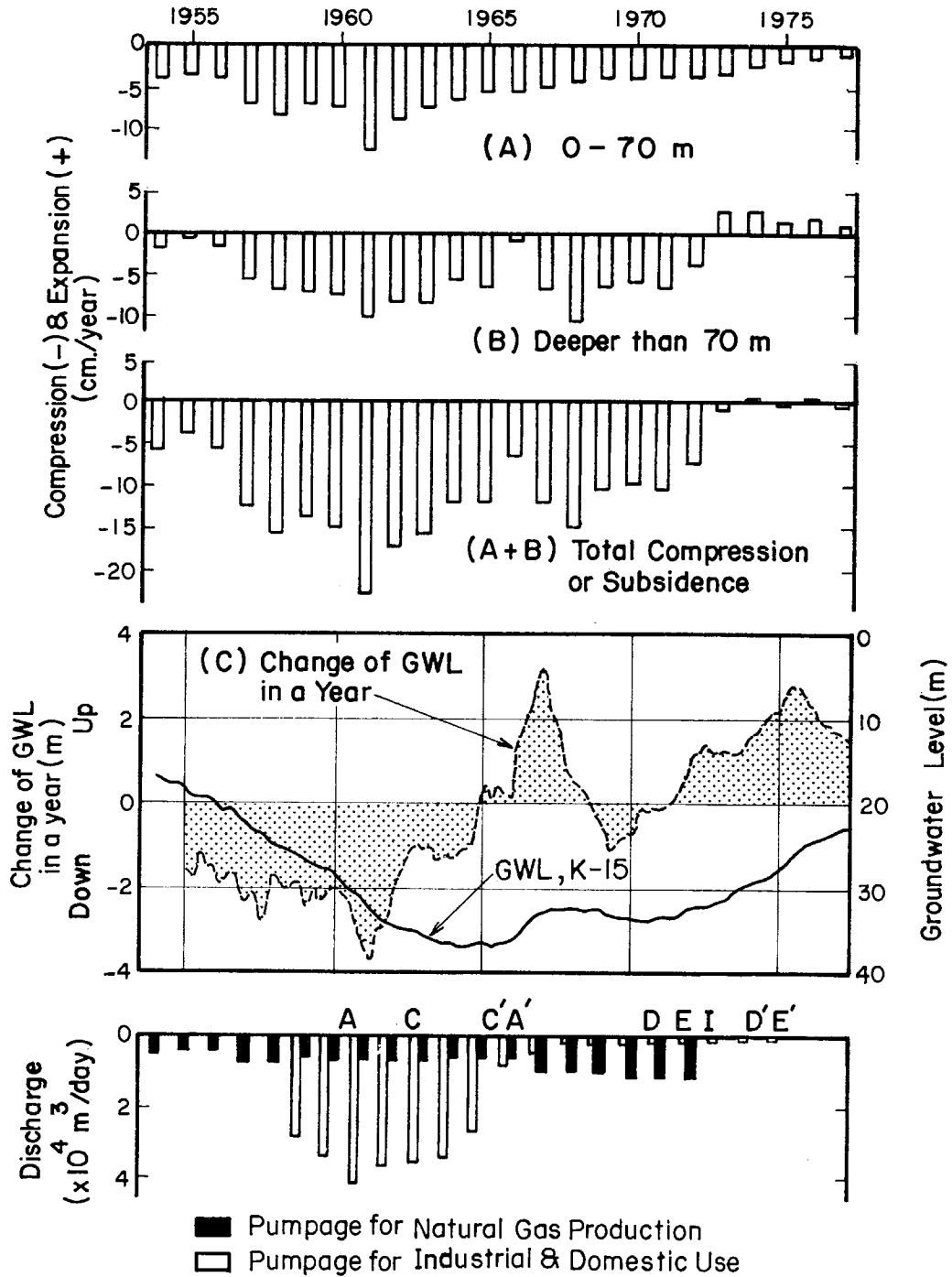the compressions occurring between these depths (A, B, and C) and at the depth greater than the deepest well (D). It may be seen clearly that the substantial part of the subsidence measured at the ground surface and indicated by black columns was the compression taking place at great depths (D) as indicated by white columns in Figure 19. It also shows that more expansion or heave is occurring at great depths as the groundwater level rises and the total compression diminishes.

It was once believed that the salt water encroachment was due to the sea water invasion into the fresh water aquifers as their piezometric levels dropped drastically. Some of the recent studies in Tokyo indicate otherwise, however.

Particularly as a result of a series of the experimental H-bomb explosions in the Pacific in the 1950's the tritium isotope of hydrogen has increased its density in the water to such an extent that the water containing a high tritium content can be differentiated from the old water that has never been exposed to the contaminated atmosphere. The analyses of groundwater from the deep aquifers in the Tokyo area show a general tendency that when the groundwater level lowers, the chlorine ion content increases but the tritium ion content decreases, whereas when the groundwater rises, just the opposite is true. This trend can be seen not only in the Kohto area close to Tokyo Bay, but in the Musashino Upland area further away from the bay and any major rivers.

When the groundwater level drops the tritium content should increase if the salt water is being introduced from the sea, because a much higher density of tritium ions still prevails in Tokyo Bay. As far as the Tokyo area is concerned, therefore, M. ISHII (1977) concluded that the porewater in the compressible soils, consisting primarily of marine deposits having a high salt content, is being squeezed out and fed into the aquifers as the piezometric level declines.

The general increase in the tritium ion content and decrease in the chlorine ion content as the groundwater level rises can be attributed to some fresh water supply from the other sources. It was once conceived that huge underground flows from the north were replenishing the deep aquifers in Tokyo at a daily rate of one to one and a half million cubic meters. The more recent hydrogeological study (SHINDO, 1968) and the tritium analysis (KAYANE, 1971) conclude that it is merely an illusion and such a natural supply of artesian groundwater is limited to the Tokyo's underground reser-
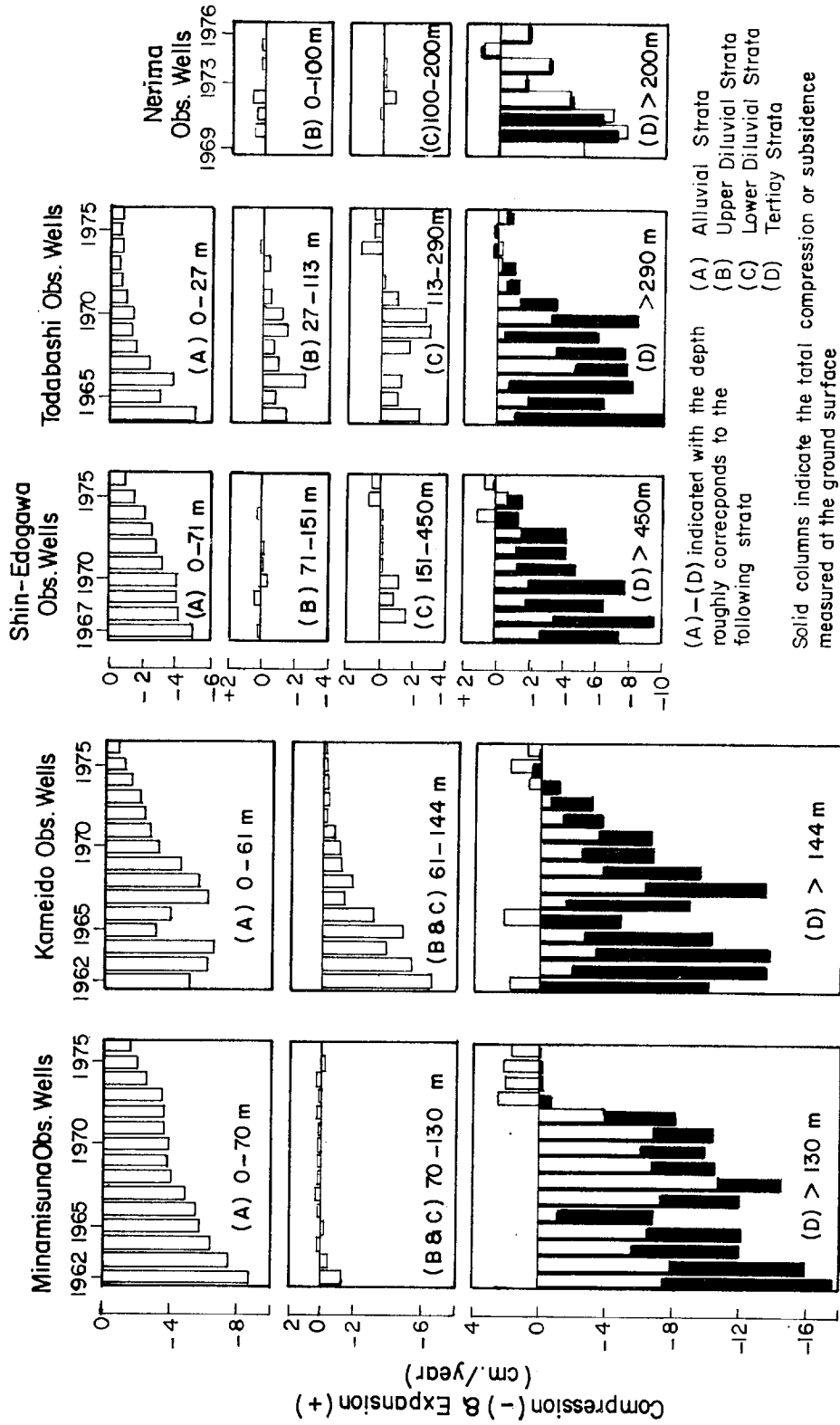
23

Fig. 19. Compressions and expansions at different depths measured at some observation stations in Tokyo (after TPWRI, 1977).

24

voir, perhaps being far less than 1 mm/day. In fact a significant portion of the dramatic rise in groundwater levels was attributable to the squeezing out of the porewater in the consolidating clays sandwiched by aquifers.

The investigations conducted for the past decade or so all point out the difficulty of assessing such controversial quantities as the safe yield of aquifers or the allowable withdrawal rate from wells that supposedly causes no harm in the area concerned. This raises arguments as to whether or not there is any such definable limit at all.

Losses and damages caused by land subsidence are also difficult to assess correctly but no doubt amount to an enormous figure. They range from such direct losses as loss of properties due to permanent inundation or frequent flooding, significant decline of the property values in subsiding areas, structural damages of public utilities, buildings and other facilities, renovation of canals, bridges, roads, etc., to necessary social-political expenditures to keep alive the areas below sea level, by providing new water supply systems, constructing and maintaining extensive dike and drainage systems including, locks, flood gates and pumping stations, etc. (UKENA et al, 1969; TERANAKA et al, 1969).

While the direct cost of groundwater in Japan usually ranges from 1 to 7 yen or US $ 0.005 to 0.035 per cubic meter, the estimated unit cost soars up to a staggering figure varying from 230 yen or US $ 1.15 per cubic meter if only the clearly identifiable public expenditures are included (SKLSIC, 1974), and to 1,100 yen or US $ 5.50 per cubic meter if the public and private expenditures are included to account for the losses and to remedy the consequences resulted from land subsidence (YAMAMOTO, 1977).

In 1977, although a maximum subsidence of 4.30 cm was recorded in Kohto Ward, less than 1 cm was detected in most of the 23 ward area. In fact, a considerable area of Tokyo experienced noticeable heave. A maximum heave of 1.69 cm was measured in Adachi Ward. As of 1978, Tokyo has 34 subsidence-monitoring stations with a total of 68 observational wells ranging in depth from 34 m to 450 m. A first-order survey is conducted annually of the 653 bench marks located in the 953 km^2 area in Tokyo. In collaboration with the neighbouring prefectures of Chiba, Saitama and Kanagawa, the southern Kanto Plain, of which Tokyo is the center, has an extensive observational network to monitor continuously the conditions of groundwater levels and land subsidence over a wide area.

Similar data are available also in the Osaka area as well as in the other subsidence afflicted areas in Japan. This data indicates that the pumpage, the groundwater level, and the land subsidence have an intimate relationship.

25

CONTROL OVER THE USE OF GROUNDWATER

General

The mechanism of land subsidence was elucidated with the convincing evidence demonstrating a close correlation between subsidence, groundwater level and discharge by pumping. Control measures were then taken to restrict the use of groundwater in the form of laws and regulations (AIHARA et al, 1969).

The "Industrial Water Law" (IWL) was enacted in 1956 and empowered the government to ban pumping and drilling new wells in any designated area where the groundwater level is dropping at an unusual rate and subsidence is in progress. It stipulates, however, that no restriction shall be imposed unless the area is provided or to be provided with an alternative water supply within a period of a year.

In 1962 another law entitled "Law Controlling Pumping of Groundwater for Use in Buildings" (LCB) was enacted to control the use of groundwater specifically for buildings which had been major users of groundwater in Japanese cities. This law is more stringent than the IWL in the sense that the restriction may be imposed even if no alternative water supply is available in the area where, in the opinion of the government, serious subsidence is taking place.

It is often commented in Japan that though they seem alike, the IWL aims primarily at preservation of groundwater resources, while the primary purpose of the LCB is prevention of land subsidence. The two laws generally forbid new installation of wells and in the areas they designate, allow the use of an existing well only when it satisfies certain requirements they impose, i.e., typically only when it has a strainer deeper than the specified depth and a pump outlet smaller than the specified cross-sectional area.

In addition to these laws, local governments such as prefectures, cities, and towns, have their own ordinances that may require well owners to observe restrictions similar to the two laws, to equip their wells with pumpage recorders, and to report the pumping records. These governments are able to order the restricted use or to ban pumping in the areas specified.

The restrictions set forth by the two laws, IWL and LCB, are by no means flawless and they have often been a target of criticism for not being stringent enough. The maximum allowable cross-sectional area of the pump outlet, for instance, is an obvious compromise to substitute the more rational but difficult to control criterion to limit the discharge by pumping from each

26

well. During the drafting of the IWL, the minimum spacing for wells was also considered but not included in the law. The minimum depth of a strainer was based on the belief at the time of the enactment that the withdrawal of groundwater from great depths had little bearing on land subsidence.

Nevertheless these laws supplemented by local regulations made it possible to establish a firm control over the use of groundwater quite effectively throughout the nation. It is true that such control merely made some of the wells deeper, but the law enforcement certainly cut down the total number of wells to such an extent that the total discharge was substantially reduced in the subsidence plagued areas.

Regulated by :

Central Government
Prefectural Government
Local Government
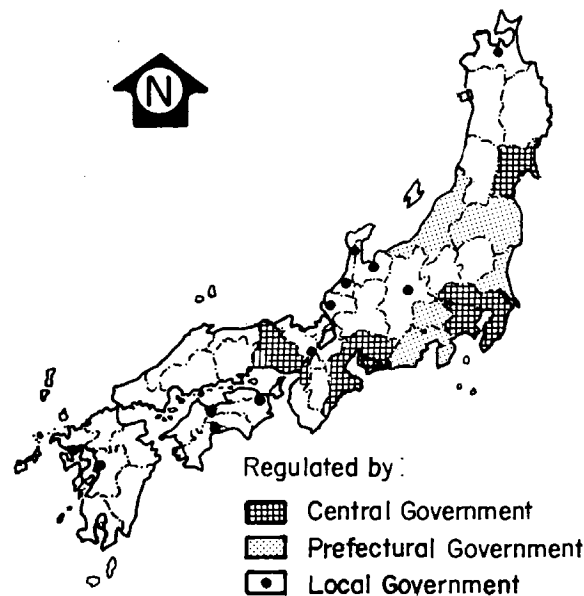
Fig. 20. Control over the use of groundwater resources (Yamamoto, 1977).

Figure 20 shows the areas of Japan presently regulated by the laws and ordinaces. Generally the areas regulated by the central government are also subjected to the regulations imposed by the local governments.

Control in Osaka

It was the city of Osaka that launched the first step toward the control of the excessive withdrawal of groundwater, determined to stop the land subsidence which increasingly had been deteriorating the conditions in the western Osaka. Figure 21 shows the lowering and rising of the groundwater levels in the City of Osaka during the 1939-1976 period. The data

27
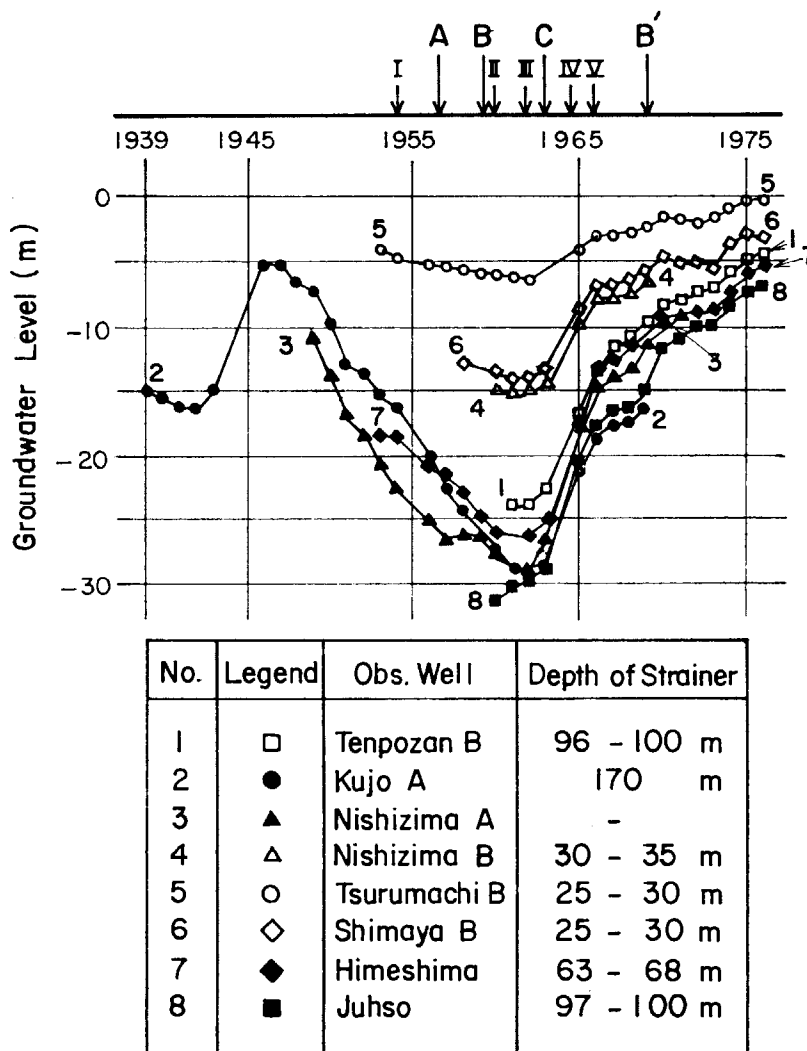
Fig. 21. Groundwater levels in Osaka, 1939-1976 (Nakamachi, 1977).

from the Kujo well is also shown in Figure 8b. The letters A, B, C, and I through V on Figure 21 indicate the events related to the control which took place in Osaka as briefly summarized in the following.

In 1954 (I) Osaka completed the first phase of its industrial water supply system even before the IWL was enacted in 1956 (A). The city ordinance to prevent land subsidence was carried into effect in 1958 (B) and the two laws IWL and LCB were proclaimed and enforced in 1962 (C). In the meantime the construction of its extensive industrial water supply system was under way in five stages to replace the groundwater resources with the

28

new surface water supply, being completed one after another (I through V). The last phase of the project was completed in 1965 (V). The laws and the city ordinance were fully enforced by the end of 1968 (B').

The response of the groundwater levels to the controls thus established is astonishing in the 1960's, particularly in the first few years after the restrictions were imposed. The recovery of the groundwater level was as much as 25 meters in 15 years.

Control in Tokyo

Figures 15 through 18 demonstrate the equally impressive recovery of the groundwater levels in Tokyo in response to the laws and regulations brought into effect. The letters A through I on these figures as well as in Figure 22 represent the following events.

In January 1961 (A), the Kohto area was designated by the IWL as a restricted area where no new wells were to be installed and no pumping would be allowed from the existing wells unless they would conform the
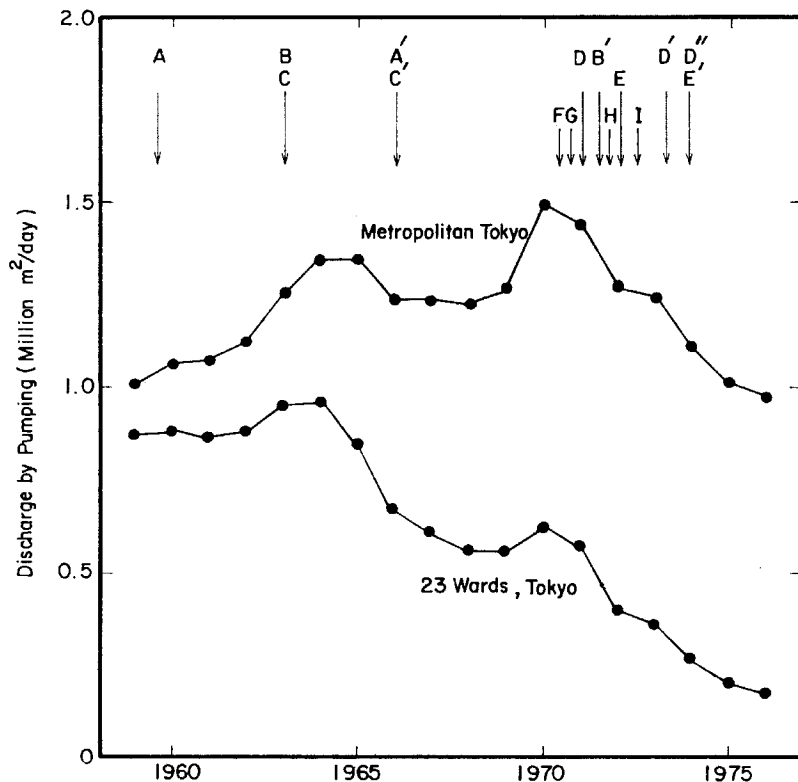

Fig. 22. Discharge by pumping in Tokyo, 1959-1976 (based on TPWRI, 1978).

29

requirements that strainers must be deeper than the depth of 100 to 250 m depending on the area so specified, and that the cross-sectional area of the outlet of a pump must be smaller than 46 cm^2 (about 7.7 cm in diameter). The same IWL restriction was imposed in the Johoku area, located just upstream of the Kohto area along Ara River, in July 1963 (B). Upon completion of the new Kohto industrial water supply system in 1965, the above restriction was fully enforced in the Kohto area by June 1966 (A'). In the Johoku area, however, it was not until December 1971 (B') when the above restriction was brought into force upon completion of the new Johoku industrial water supply system in March 1971.

For the buildings located in the 14 downtown wards with the elevations lower than A.P. 5 m, the same restriction was applied by the LCB in July 1963 (C) and was carried into effect by July 1966 (C').

In spite of such law enforcement, land subsidence continued at an alarming rate throughout the 1960's. It was therefore decided, starting in May 1971 (D) in the Kohto and Johoku areas, to tighten up the control measure by virtue of the IWL as follows. No existing wells were to be pumped unless they had strainers at a depth greater than 400 m to 650 m depending upon the specified areas, and unless the pump outlet's cross-sectional area did not exceed 21 cm^2 (about 5.2 cm in diameter). In the areas subjected to the tighter IWL restrictions, numerous wells had to be abandoned and the remaining wells were converted in conformity to the more stringent requirements by September 1973 (D') in the Kohto area, and by April 1974 (D'') in the Johoku area. In fact the remaining areas in Tokyo also conformed to this law by April 1977.

Likewise the same stringent control was enforced by the LCB in May 1972 (E) and was put force by May 1974 (E') for all the buildings located in the entire Tokyo area. In the meantime, the Metropolitan Tokyo government set forth a revised ordinance on its own in November 1970 (F). This ordinance empowers the governor to impose restrictions on the users of wells, to order to improve or modify the facilities so that the water may be recycled for repeated use or an alternative water source may be used. It also stipulated that every well having a pump outlet with the cross-sectional area larger than 21 cm^2 be equipped with a pumpage recorder and that the owner should keep the record of pumping and report it to the governor. Since February 1971 (G), therefore, this ordinance has made it possible to obtain a good record of pumpage throughout the Tokyo area.

In April 1972 (H), the ordinance was strengthened by including the same requirement as stipulated by the 1971 IWL and was enforced quite

30

effectively over almost the entire Tokyo area. This forced many wells to be either converted or abandoned.

Natural gas production continued in Kohto and Edogawa Wards at a gradually diminishing rate coming to a complete halt in December 1972(I) when the Metropolitan Tokyo government agreed to purchase the mining rights of the gas-producing wells.

With the mounting public concern for serious consequences of land subsidence and also with the positive, if not always willing, cooperation from the industries, the two laws IWL and LCB as well as the ordinance have proved to be an effective means to control the use of groundwater, to help it recover and to brake the persistent subsidence virtually to a stop. The effectiveness of the control is best illustrated in Figure 22 which summarizes the discharge by pumping in the entire metropolitan Tokyo area and in the 23 ward area during the period between 1959 and 1976. The letters A through I on Figure 22 show when the various control measures came into force as described in the foregoing. While the entire Tokyo area still consumed nearly a million cubic meters of groundwater everyday as of 1976, the use of groundwater in the 23 ward area was successfully cut down to a daily average of 185,000 m³/day in 1976, only 19% of 967,000 m³/day being pumped in 1964. Figures 15 through 18 also demonstrate the rapid recovery of groundwater levels and the decelerated subsidence in response to the controls established.
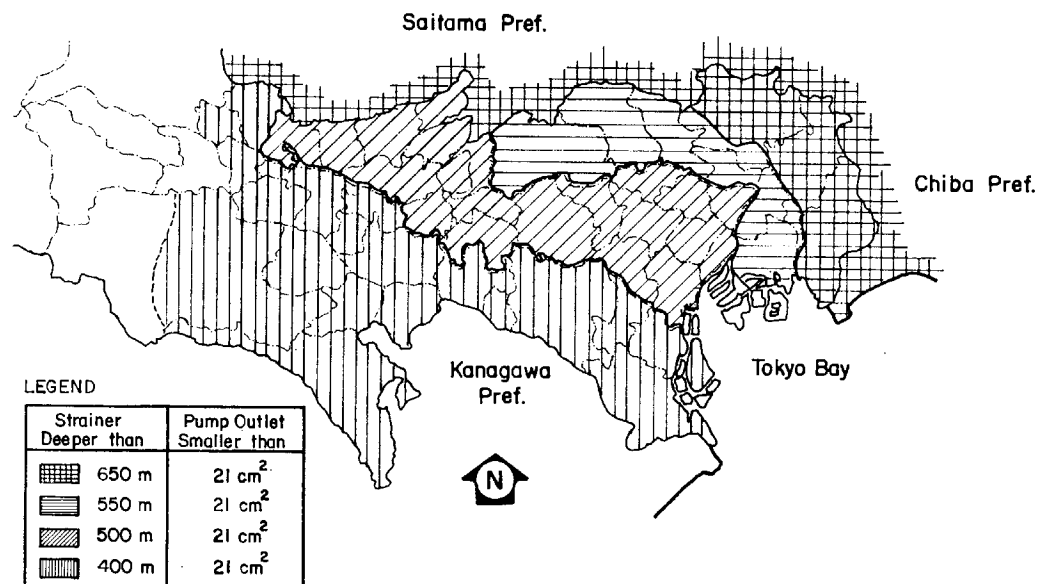

Fig. 23. Groundwater control in Tokyo (after ALSAC, 1973).

However, in the Tama area in Tokyo, located just west of the 23 wards, depletion of groundwater still continues in large quantities because of the lack of the adequate alternative water source. The metropolitan government also enforced its ordinance in this area in 1972 when the daily pumpage there reached 874,000 m³/day. In 1976 a daily average of 792,000 m³/day was being pumped and a drop in the groundwater levels is still being noted in some of the aquifers. Presently most of the metropolitan area is designated as subsidence areas which are subjected to either, or a combination, of the two laws and the ordinance as shown in Figure 23.

Control in Niigata

In and around the city of Niigata, a number of natural gas producing wells have been in existence for many years. Particularly after the Second World War, it was found that abundant natural gas was dissolved in the brackish groundwater in deep sand and gravel layers (Fig. 24), notably in the G-4 layer at the depth ranging from 250 to 490 m, the G-5 layer 380 to 780 m, and the G-6 layer 500 to more than 1,000 m in depth. Pumping of natural gas for domestic use has continued from the shallower gas-yielding layers, mainly from G-1 through G-3 at the depths varying between 50 to 100 m. The withdrawal of gas-bearing groundwater for the industrial
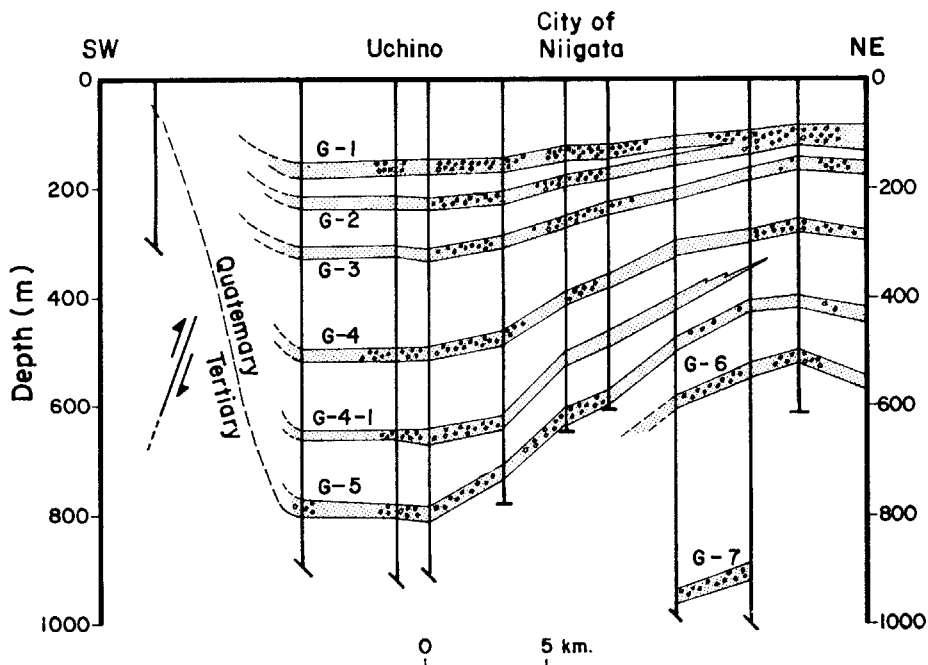

Fig. 24. Geologic section along the sea coast in Niigata (Niigata Prefecture, 1974).

purpose has been far more greater from the more productive deep aquifers, G-4 through G-6, and also from a few other layers later found at greater depths (TAKEUCHI et al, 1969; AOKI, 1976).

It was estimated that in 1953 the natural gas companies were pumping 147,000 m³ of groundwater everyday. By 1958 the discharge for natural gas production reached the average daily rate of 527,000 m³ from more than 450 industrial wells some of which were as deep as 2,000 m (Fig. 25). AOKI, (1976) gives the geographical distribution of industrial and domestic wells in the Niigata area as well as the subsidence measured during the 15-year period between 1959 and 1974 (Fig. 26). In Figure 26 each point represents a dozen to a few dozen wells.
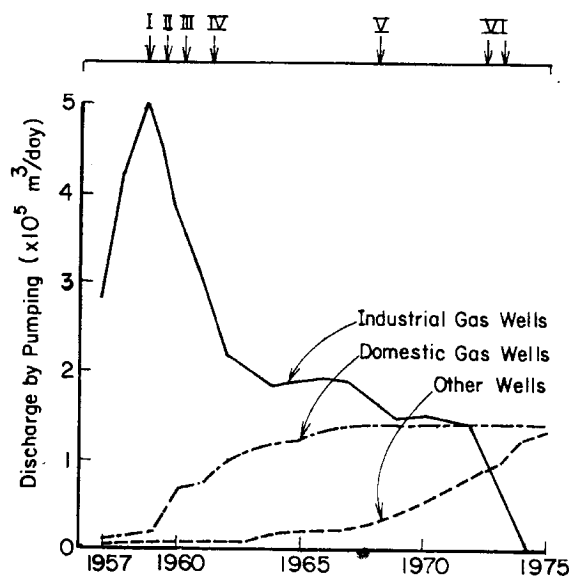
Fig. 25. Discharge by pumping in Niigata, 1957-1975 (after Aoki, 1977).

As the land subsidence investigation was initiated in 1957 and the main cause became apparent, the first control was enforced voluntarily by the gas producing companies in February 1959 (I), Figures 25 and 27, reducing the daily discharge by 60,000 m³ in the Yamanoshita area of the city where the land subsidence had been so severe almost the entire area had already dropped below sea level. It was estimated that the groundwater levels in this area had been lowered 30 to 40 m by that time (Fig. 27).

It was not until September 1959 (II), however, when effective control was initiated in the city of Niigata. The central government banned the major gas companies from pumping in the urban area, which reduced the

33
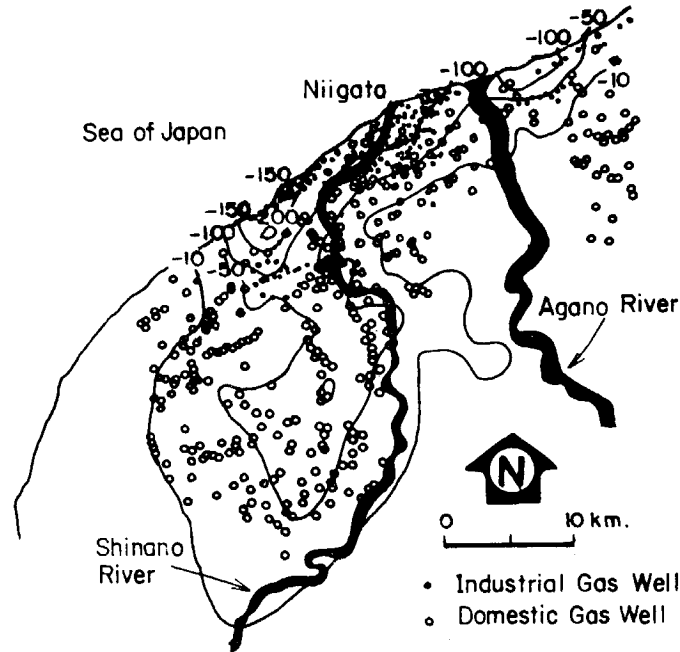
Numbers on contours indicate total subsidence in cm., 1959–1974

Fig. 26. Distribution of wells and subsidence contours in the Niigata Area (Aoki, 1976).

daily discharge by about 125,000 m³. In July 1960 (III), the proposal to cut down the pumpage to the 1956 level of 250,000 m³/day was submitted to voluntarily by the two associations of gas-producing companies in the Niigata area and was approved by the central government. This action supposedly decreased the total discharge by 142,000 m³/day.

In November 1961 (IV), the fourth control was enforced by the central government. This divided the Niigata area into three zones; approximately the city proper (Area A), a 5 km radius zone from the city center just outside the Area A (Area B), and a 10 km zone outside the Area B (Area C). Thus in Area A the pumping for natural gas production was totally banned, in Area B no pumping was permitted from the layers shallower than the G-6 layer and no well was allowed to yield more than its previous record, and in Area C continued pumping was allowed provided that it did not exceed the recorded maximum discharge. This governmental control reportedly cut down the total discharge by about 162,000 m³/day.

34

In July 1967 (V) a further voluntary control was established in Area C stopping pumping from the aquifers shallower than G-3. In response to the recommendation given by the central and prefectural governments, the association of natural gas industry finally decided to convert all of their wells in Niigata so that, upon separation of gas, all the pumped water should be recharged back into the deep aquifers, G-4 through G-6. The conversion was completed during the one-year period starting in October 1972 (VI). Since October 1973, therefore, no dis-

Numbers in meter indicate the strainer depths of Yamanoshita Obs. Wells

Fig. 27. Recovery of groundwater levels in Niigata, 1958-1975 (Aoki, 1977 and Niigata Prefecture, 1978).

charge has been permitted from the industrial wells, The daily recharging rate has since then been in the range of 130,000 to 150,000 m³/day.

With the series of stringent controls enforced in the Niigata area, the discharge from the industrial wells has been drastically cut down to nil, but pumping still continues from other wells (Fig. 25). Although domestic gas wells were also placed under control in 1963, 1966 and 1969, the wells for other uses are discharging at an increasing rate.

The groundwater levels have recovered as much as 25 m since the first control was established, responding remarkably well to the tightening measures enforced thereafter (Fig. 27). The subsidence reduced its pace considerably since around 1966 to such an extent that during the 1973-1974 period a heave as much as 2.8 cm/year was recorded in the Uchino area. Land subsidence, although only at a nominal rate, still continues in many areas in Niigata, and the 48 observational wells installed to the maximum depth of 1,200 m are detecting compressions in progress in various shallow and deep strata. With approximately 150 km² of the Niigata area having been lowered below the high tide level due to the disastrous land subsidence, Aoki (1977) warns that the effectiveness of recharging is yet to be fully determined and the land subsidence monitoring system be strengthened.

LAND SUBSIDENCE IN THE BANGKOK AREA, THAILAND

Bangkok, like Tokyo, Osaka and Niigata, is situated on a large, flat alluvial plain and also near a sea coast, approximately 25 km from the Gulf of Thailand with a major drainage channel, Chao Phraya River, meandering through the area. The metropolitan Bangkok covers an area of approximately 750 km² with a population of about 4.5 million. The area being so flat, its elevation ranges from only 1.0 to 2.0 m above mean sea level.

Bangkok is in the center of the Lower Central Plain of Thailand, which consists of a geological depression filled mainly with alluvial and deltaic deposits with minor shallow sea sediments, exceeding 2,000 m in thickness (Fig. 28). In the upper 550 m or so of the sediments there are eight principal aquifers which consist of sand and gravel with some clay inclusions. These aquifers are separated by thick, virtually impermeable, layers of clay or sandy clay. The regional geology and hydrogeology have been discussed in detail by many investigators, e.g., Cox (1968), SODSEE (1978) and AIT (1978a). The reader is referred to these works for additional references.
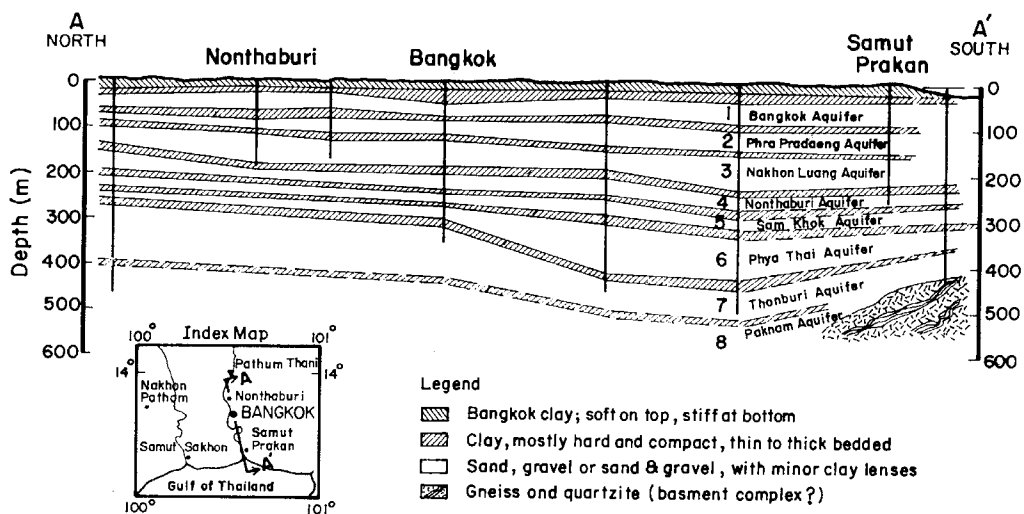
Fig. 28. Geologic section in the Bangkok area (after Piancharoen, 1976).

Figure 29 shows the estimated pumpage in the metropolitan Bangkok area. Only a nominal amount of groundwater had been pumped for domestic use before heavy utilization commenced in 1957. For the past two decades or so, about one third of the total public water supply has come from the public wells drilled and operated by the Bangkok Metropolitan Water Works Authority (MWWA Wells in Figure 29). Reportedly, a total of some 6,000 wells exist with the number of private wells increasing sharply in the recent
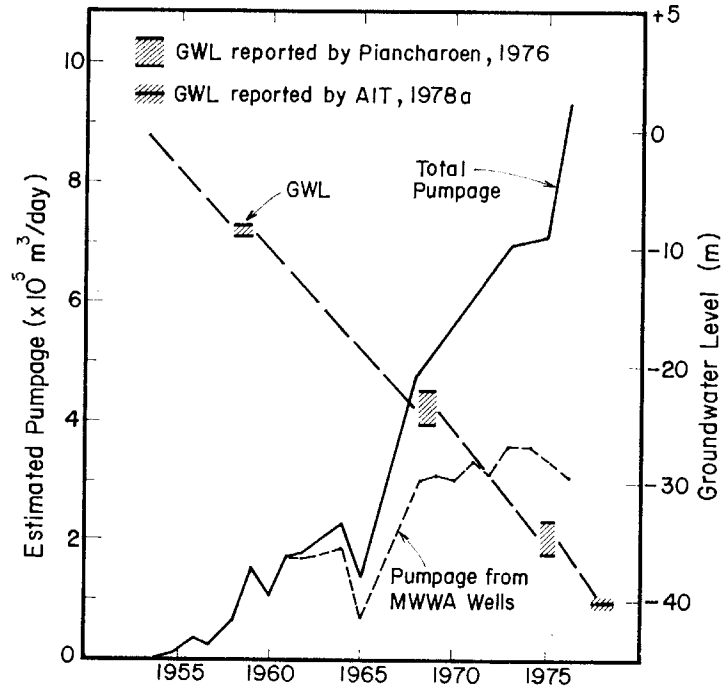
Fig. 29. Estimated pumpage and groundwater level in the Metropolitan Bangkok Area,1954-1976 (AIT, 1978a).

years. It was estimated that the total pumpage for domestic and industrial use was on the order of 700,000 m³/day around 1974 (PIANCHAROEN, 1976), but soared up to about 940,000 m³/day in 1976 (AIT, 1978b).

In view of the heavy depletion of the groundwater resources, the Groundwater Act was implemented by the Thai Government in 1978, but presently imposes no control over the pumping from the existing wells nor over installation of new wells.

Because the groundwater in the first aquifer is saline, most wells penetrate the second, the third and the fourth aquifers which are highly productive and yield water of good quality. It has repeatedly been pointed out that excessive pumping is causing drastic drawdown in the water levels in the deep aquifers. Consequently the new wells are becoming deeper and deeper and in fact the second aquifer is gradually being abandoned because of salt water encroachment. Figure 29 also demonstrates the decline in the piezometric levels in the second through the fourth aquifers. The piezometric levels used to coincide with the ground surface but are now lowered to more than 40 m below it in the cones of the groundwater level depressions which are fairly well defined in the metropolitan area.

37

Ground Surface

Pore Pressure , u , t/m^2

0   5   10   15   20   25

Weathered Clay

Assumed 1.0 m Mean Annual Water Table

Soft Clay

Hydrostatic Pressure

Pore Pressure Decline, Δu

Depth (m)

Stiff Clay

Fluctuation in 1975

Sand

Fluctuation during
Aug. 1977 to June 1978
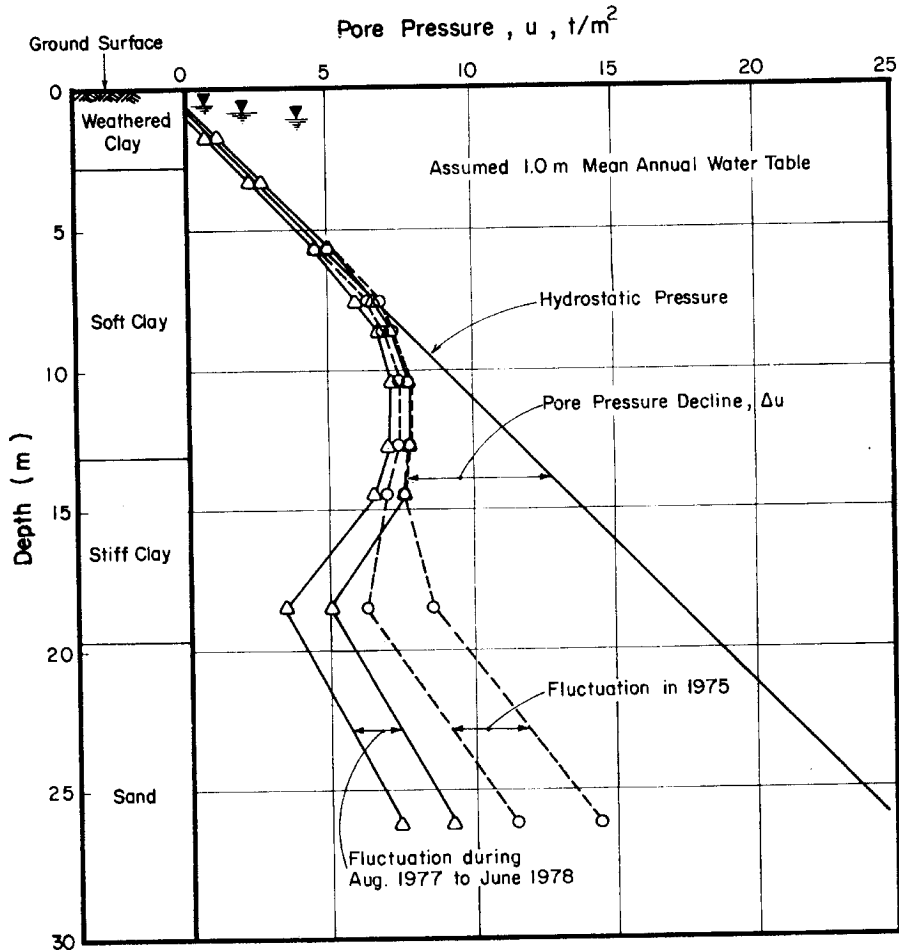
Fig. 30. Change in piezometric levels in the Bangkok Clay Layer, Chulalongkorn University Campus, Bangkok (AIT, 1978a).

Some of the piezometers installed in the shallow Bangkok clay layer also indicate that the piezometric pressures are well below the hydrostatic level and this highly compressible and relatively impermeable clay has clearly been in the process of consolidation for quite some time (Fig. 30).

Unfortunately no long term continuous measurements of subsidence and groundwater levels have been made in the Bangkok area. Based on the protrusions of deep wells (Photo 3), however, it has been estimated that the total subsidence is on the order of 50 cm in central Bangkok (AIT, 1978a). In 1978, the Royal Thai Survey Department conducted a first-order relevelling survey of the principal bench marks in the Bangkok area for the first time
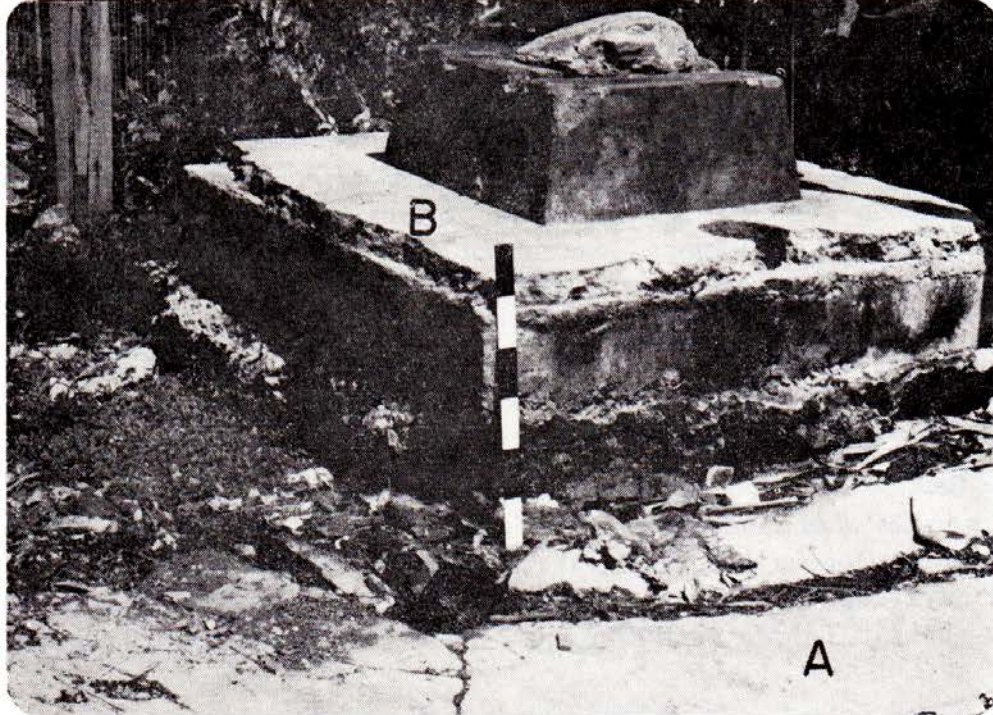
Photo 3. Protrusion of an abandoned well at Lumpini Park, Bangkok (120 m deep, installed some 20 years ago). A vertical scale Shows approximately 50 cm protrusion of the concrete cap above the present ground surface (A). Plane (B) coincided with the ground surface at the time of well installation. Photo taken in June, 1978 by Dr. Jerasak Premchitt.

since their installation, and found that their elevations had changed in a range from 20 cm to 86 cm. For example, a downward change of 49.3 cm was detected at the bench mark (P 386 installed in 1933) located at the Meteorological Department in the downtown Bangkok where a large decline of groundwater level had been noted.

Considerable analyses have been carried out at the AIT to predict the future trend of land subsidence. BRAND & BALASUBRAMANIAM (1976) forecast the probable ultimate subsidence of about 2.0 m, 2.7 m and 3.1 m for the permanent drop of groundwater level of 20 m, 30 m and 40 m, respectively, on the assumption that an equal pressure decline occurred throughout the entire 600 m of soil deposits. Of the above predicted values, the compression of the soft Bangkok clay amounted to approximately 1.0 m. On the basis of his model study, PREMCHITT (1978) estimated the present subsidence rate of the order of 5 cm/year, and also predicted an annual rate of the surface subsidence ranging from 1.2 to 7.5 cm/year with the total sub-

39

sidence from 137 cm to 212 cm taking place in the next 20 years if the current pumping rate or a greater rate continued.

Since January 1978 the AIT has been conducting an investigation of land subsidence in the greater Bangkok area in compliance with the request made by the National Environment Board, Government of Thailand. In connection with this study, subsurface investigations and instrumentation were made at 24 locations. Subsidence and groundwater levels are being measured systematically to monitor the changes in the subsurface strata to the depth of about 50 m. On the basis of the short observational period of 4 to 5 months in 1978, it was estimated that the rate of the compression in the upper 50 m zone was about 3 cm/year, suggesting that the rate of the surface subsidence was somewhat greater than this (AIT, 1978c).

Because of the low altitude of the Bangkok area, only 1.0 to 2.0 m above mean sea level, land subsidence is indeed a serious threat to the existence of the metropolis itself. In fact the maximum water level of 2.05 m in Chao Phraya River was recorded at the end of the long rainy season in 1975 and again in 1978 at Memorial Bridge near the heart of Bangkok, and a substan-
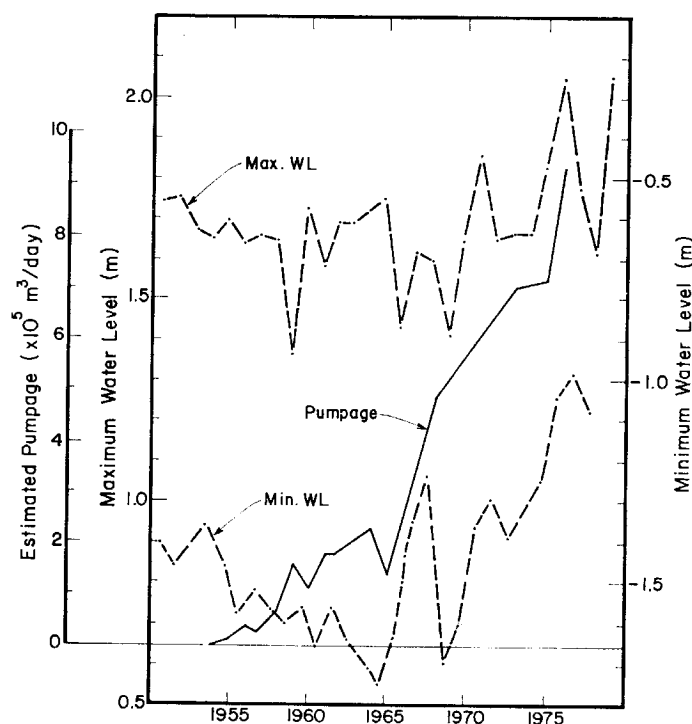
Fig. 31. Yearly maximum and minimum water levels of Chao Phraya River recorded at Memorial Bridge, Bangkok (based on the Data from Royal Irrigation Department).

40

tial part of the downtown area remained inundated for several days. Not only during such crucial periods but virtually during every high tide season, the low-lying areas in Bangkok suffer increasingly from prolonged floods in recent years.

Figure 31 shows the yearly maximum and minimum water levels recorded at Memorial Bridge, together with the estimated total pumpage given in Figure 29. It is interesting to note that there appears to be a trend of gradual increase in these extreme water levels after 1965 when the pumpage started to sharply increase. At least this upward trend supports the general feeling of the public that the degree of flooding has been increasing year after year.

The water levels are measured by the height of a gage which moves downward as the ground subsides. If the water levels fluctuate within a certain range every year, the record of the water levels should represent an indication of a continuous record of land subsidence. One such example is given in Figure 32 which shows the tide levels recorded in the Niigata harbour indicating an unusual rise starting around 1955, as compared with those taken at the harbours of Sakata and Nanao, approximately 130 km and 210 km away from Niigata, respectively. This unusual rise in the tide level coincides almost precisely with the land subsidence occurring then in the Niigata harbour area (MOMOTAKE & MIYAZAWA, 1976).

Fig. 32. Tide level records in the harbours of Niigata, Sakata and Nanao, Japan, 1947-1959 (Momotake and Miyazawa, 1976).

No definite conclusions may be drawn from the data in Figure 31, because the river level fluctuations are much more complex than the tide levels in

sea ports. A plot such as this, nevertheless, seems to suggest a possibility that the land subsidence in Bangkok has accelerated since around 1965.
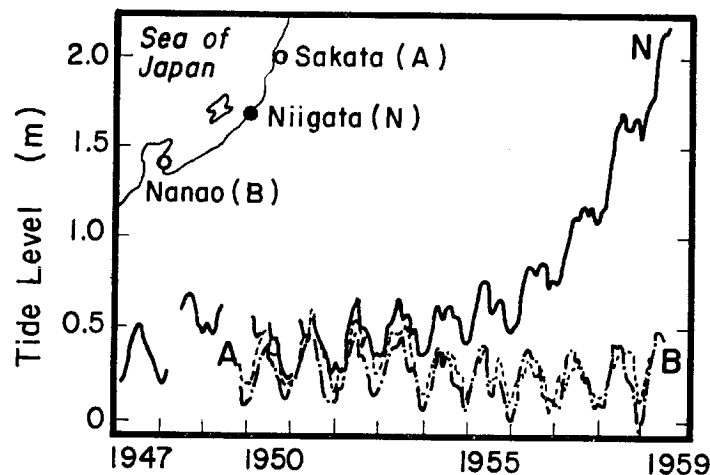
In spite of the current pumping rate, the magnitude of drops in groundwater levels, and the geological condition of thick unconsolidated sediments, all of which are comparable to the situations in Tokyo, Osaka and Niigata, the total subsidence and the rate of subsidence as estimated have so far been surprisingly small in the Bangkok area.

One possible explanation for this is the considerably slower process of consolidation taking place in the thick stratum of the relatively impervious Bangkok clay than that in the strata of clays and silts normally encountered in Japan. According to TERZAGHI's one-dimensional consolidation theory, the effect of permeability is represented by the coefficient of consolidation, c_v. The theory dictates that the subsidence of the ground surface, S, at time t due to the consolidation of the compressible strata may roughly be expressed:

$$S = \sum S_{ui} \; f \, (c_{vi} \, t / H_i^2) \dots\dots\dots\dots\dots (2)$$

in which S_{ui} is the ultimate consolidation settlement of the ith compressible layer and H_i is the maximum length of drainage path in the ith layer. H_i is therefore either one half of the layer thickness when drainage is provided at the top and bottom of the layer, or the entire thickness when drainage is allowed only at one end. In a simplified sense, therefore, S_{ui} is directly proportional to H_i.

In short, the smaller the c_v value and the greater the H_i value, the smaller is the total subsidence S, as well as the rate of subsidence at time t.

As a matter of crude comparison, the laboratory oedometer test results indicate that the alluvial and marine clays in Japan generally have a c_v value of the order of 10^{-3} cm^2/sec, whereas the soft Bangkok clay in general on the order of 10^{-4} cm^2/sec (AKAGI, 1972), although very little is known of the properties of the clays at great depths.

It is often postulated on the basis of boring logs that the value of H_i may be clearly defined, but in many cases strata of soft clays and silts in Japan contain thin seams of sand which often remain undetected and in actuality serve as drainage layers, thus reducing the actual value of H_i considerably and accelerating the consolidation process drastically. In fact, experiences indicate that the correct values of H_i are often as difficult to determine as the appropriate values of c_v. In this regard, the more impervious Bangkok clay in general appears relatively homogeneous having fewer subdrainage layers in it, as compared with the more pervious soft sediments generally

encountered in the subsidence plagued localities in Japan. In short, the consolidation parameter, c_v/H^2, for the Bangkok clay stratum is likely to be at least one order of magnitude smaller than those for the cases in Japan.

Another possible reason for the smaller magnitude of subsidence and its rate in the Bangkok area may lie in the difference in the compressibilities of the soils at great depths. This, however, is merely a speculation at the present time and is yet to be investigated.

The above comments pertain only to the total subsidence and the rate of subsidence at time t. It is to be noted that land subsidence definitely continues, although at a relatively slow pace in the Bangkok area, as the groundwater level goes down or even if it is held at the present level. Should the piezometric level rise in the aquifers as a result of some control over the use of groundwater or recharging, it is highly likely, by the same token, that the response of the ground surface would be much slower than those experienced in the Japanese cities, and the subsidence would continue for a prolonged period of time before it ever stops or shows any sign of heaving.

Because of the slow rate of subsidence, varied opinions still persist whether the problem of land subsidence indeed exists in the Bangkok area. With the bitter experiences in Japan as summarized in the foregoing, however, it comes as no surprise when one hears even some specialists in Bangkok say that 'the present deep well pumpage, mostly below 150m, has no effect on land subsidence,' that 'if there is any subsidence, external loads are to blame,' and that 'local flooding is due to poor drainage,' as has been reported in the Bangkok area.

CONCLUSIONS

(1) The development and conditions of land subsidence in Tokyo, Osaka and Niigata in Japan as well as Bangkok, Thailand, have been reviewed and some of the relevant data in each case are summarized in Table 3.

(2) The three major cities in Japan as well as Bangkok, Thailand, are situated near a sea coast and on an alluvial plain, consisting of thick strata of unconsolidated sediments, all of which are significantly compressible under the effective stress changes caused by the lowering of the groundwater level as much as a few tens of meters. Generally the upper several hundred meters contain several aquifers which constitute the highly productive water resources of good quality for domestic and industrial use. In Tokyo and Niigata, some of the deep aquifers form the source of natural gas production.

Table 3. Some land subsidence data in three Japanese cities and in Bangkok.

	Tokyo (23 Wards)	Osaka	Niigata	Bangkok
Maximum Pumpage (m³/day)	967,000	470,000	658,000	940,000
Lowest Groundwater Level Below Mean Sea Level (m)	64	30	42	>40
Drop in Groundwater Level:				
Average Rate (m/year)	2.7	1.7	?	1.7
Maximum Rate (m/year)	4-5	4-5	6-7	3-4
Depths of Aquifers Heavily Exploited (m)	100-500	100-500	200-1,000	100-300
Recorded Maximum Subsidence (cm)	458	286	228	>50
Recorded Maximum Rate of Subsidence (cm/year)	42	20	54	13.4*
Area Below Mean Sea Level (km²)	70	30	60	?
Population in the Above Area	600,000	340,000	75,000	?
Area Below High Tide Level (km²)	120	50	150	?
Population in the Above Area	2,400,000	630,000	130,000	?

* According to the latest data released by the Royal Thai Survey Department (March, 1979), the bench mark at the Meteorological Department in Bangkok settled 4.04 cm for the period of 110 days between July and November, 1978. This indicates an estimated maximum subsidence rate of 13.4 cm per year in the Bangkok area.

(3) The heavy depletion of the groundwater in Japan over a period of a few decades has resulted in a drastic decline of groundwater level totalling a few tens of meters at an annual rate of a few meters, and in land subsidence as much as a few meters at a yearly rate reaching a few tens of centimeters.

(4) There are close correlations between the pumpage, the groundwater level and the subsidence. The land subsidence in Tokyo and Osaka has a unique history which has twice repeated a drop and rise cycle of groundwater level with the corresponding subsidence phenomena. Where the groundwater level rises rapidly, some small heave of the ground surface is always noted.

(5) The deep well pumping from the depths of several hundred to more than one thousand meters causes land subsidence. In response to the decline and rise of the groundwater level, the deep soil strata indicate noticeable compressions and heaves.

(6) A loss of elevation as much as a few meters eliminated a scant freeboard above the sea level of the low-lying areas in some of the major cities located near the sea coast. Consequently substantial portions of the downtown areas in these cities are permanently below sea level, the existence of which has been made possible only by the provision of the extensive dike and drainage systems. The losses and damages due to land subsidence are so enormous that if they are taken into account the cost of groundwater would be prohibitive.

(7) The control over the use of the groundwater resources in the form of laws and regulations successfully helped the groundwater level recover rapidly and in general arrested the subsidence in Japan. This means, however, that land subsidence, still continues over a wide area although at a drastically decelerated rate, and at best nominal heaves up to several centimeters have occurred in places. Land subsidence is a non-reversible phenomenon and the once lost ground never bounces back to its original elevation.

(8) Ample evidence of land subsidence in progress has been accumulated in the Bangkok area, Thailand, although the rate of subsidence has so far been relatively small as compared with the Japanese experiences. The pumpage, the drop in groundwater levels and the geological setup are all strikingly similar. The indications are that subsidence will definitely continue at a rate of several centimeters a year unless some control over the use of the groundwater resources is established at the earliest possible time.

(9) The Bangkok area being so flat and low-lying, land subsidence is a serious threat to the existence of the metropolis itself. It has been plagued by prolonged floods in localized areas increasingly in recent years and such is a typical symptom of land subsidence in progress. In view of the past experience in Japan and the recent studies conducted in the Bangkok area, it is a matter of time before a substantial area subsides 1 to 2 meters, namely, being lowered below mean sea level, should the current pumpage be allowed to continue.

(10) Presently Bangkok is not prepared for such serious consequences. A dike and drainage system to protect the Bangkok metropolis would be not only extremely costly but present difficult construction and maintenance problems because of the presence of the highly compressible, weak Bangkok clay.

(11) It is recommended that some effective control measures be enforced urgently to cut down the pumpage at least where the cones of the groundwater level depressions are noted, and that the observational systems be intensified including more new installations of instruments to monitor the changes in groundwater levels and the compressions taking place at great depths.

(12) Land subsidence is caused by man and can also be controlled by his will.

> *We learn from history that we learn nothing from history*
> (*George Bernard Shaw*).

45

ACKNOWLEDGEMENTS

REFERENCES

AIHARA, S., UGATA, H., MIYAZAWA, K. and TANAKA, Y. (1969), Problems on the Groundwater Control in Tokyo, *Proceedings of the Tokyo Symposium on Land Subsidence*, Vol. 2, pp. 635-644.

AKAGI, T. (1972), Geotechnical Problems in Connection with Highway Construction in the Bangkok Area, Thailand, *Research Report* Vol. 8, *Faculty of Engineering, Toyo University*, Saitama, pp. 71-87.

AOKI, S. (1975), Foundations and Land Subsidence (in Japanese), *Kiso-ko, Sogoh Doboku Ken*, Tokyo. Vol. 3, No. 2, pp. 2-10.

AOKI, S. (1976), Land Subsidence in Niigata, *Proceedings of the Second International Symposium on Land Subsidence*, Anaheim, California, pp. 105-112.

AOKI, S. (1977), Land Subsidence in Niigata Plain (in Japanese), *Japanese Society of Soil Mechanics and Foundation Engineering, Tsuchi-to-Kiso* (Journal), Vol. 25, No. 6, pp. 21-28.

ASIAN INSTITUTE OF TECHNOLOGY (1978a), History, Theoretical Considerations, Field Instrumentation and Laboratory Testing Program, *Investigation of Land Subsidence Caused by Deep Well Pumping in the Bangkok Area, Progress Report, Phase I*, 104 p.

ASIAN INSTITUTE OF TECHNOLOGY (1978b), Groundwater Resources in Bangkok Area: Development and Management Study, *Final Report, Phase 1*, 65 p.

ASIAN INSTITUTE OF TECHNOLOGY (1978c), Geology, Field Instrumentation, Laboratory Testing Results and Mathematical Model, *Investigation of Land Subsidence Caused by Deep Well Pumping in the Bangkok Area, Final Report, Phase I*, 214 p.

ASSOCIATION OF LAND SUBSIDENCE-AFFLICTED CITES (1973), *Land Subsidence* (in Japanese), Brochure, Tokyo, 17 p.

BRAND, E.W. and BALASUBRAMANIAM, A.S. (1976), Soil Compressibility and Land Subsidence in Bangkok, *Proceedings of the Second International Symposium on Land Subsidence*, Anaheim, California, pp. 365-374.

COX, J.B. (1968), A Review of Engineering Properties of the Recent Marine Clays in South East Asia, *Research Report* No. 6, Asian Institute of Technology, Bangkok.

HIRONO, T. (1972), Cause of Land Subsidence (in Japanese), *Seko-Gijutsu, Nikkan-Kogyo Shimbun*, Tokyo, Vol. 5, No. 2, pp. 41-51.

INABA, Y., AOKI, S., ENDO, T. and KAIDO, R. (1969) Reviews of Land Subsidence Researches in Tokyo, *Proceedings of Tokyo Symposium on Land Subsidence*, Vol. 1, pp. 87-98.

ISHII, M., KURAMOCHI, F. and ENDO, T. (1976), Recent Tendencies of the Land Subsidence in Tokyo, *Proceedings of the Second International Symposium on Land Subsidence*, Anaheim, California, pp. 25-34.

ISHII, M. (1977), Land Subsidence in Tokyo (in Japanese), *Japanese Society of Soil Mechanics and Foundation Engineering, Tsuchi-to-Kiso* (Journal), Vol. 25, No. 6, pp. 29-36.

ISHII, Y. (1949), Investigation on Land Subsidence in Osaka (in Japanese), *Report of Harbor Technical Committee, Bureau of Osaka Harbor*, City of Osaka.

KAIDO, S. (1975), Recent Trend of Land Subsidence in Osaka (in Japanese), *Kiso-ko, Sogoh Doboku Ken*, Tokyo, Vol. 3, No. 2, pp. 60-69.

KAYANE, I, (1971), Report on Tritium Analysis (in Japanese), *Southern Kanto Land Subsidence Investigation Committee*, Tokyo.

MOMOTAKE, M. and MIYAZAWA, K. (1976), Land Subsidence in Niigata (in Japanese), *University of Niigata Chikoh-ken Report* No. 4, pp. 349-355.

MURAYAMA, S. (1969), Land Subsidence in Osaka, *Proceedings of Tokyo Symposium on Land Subsidence*, Vol. 1, pp. 105-130.

NAKAMACHI, H. (1977), Land Subsidence in Osaka Plain (in Japanese), *Japanese Society of Soil Mechanics and Foundation Engineering, Tsuchi-to-Kiso* (Journal), Vol. 25, No. 6, pp. 61-67.

NAKANO, T., KADOMURA, H. and MATSUDA, I. (1969), Land Subsidence in the Tokyo Deltaic Plain, *Proceedings of Tokyo Symposium on Land Subsidence*, Vol. 1, pp. 80-87.

NIIGATA PREFECTURE (1974), *Land Subsidence in Niigata* (in Japanese), Brochure, Niigata, 32 p.

NIIGATA PREFECTURE (1978), *Land Subsidence in Agricultural Land* (in Japanese), Brochure, Niigata, 78 p.

OKA, G. (1945), Study on Mechanism of Land Subsidence (in Japanese), *Report of Tokyo Public Works Research Institute*, Tokyo.

PIANCHAROEN, C. (1976), Groundwater and Land Subsidence in Bangkok, Thailand, *Proceedings of the Second International Symposium on Land Subsidence*, Anaheim, California, pp. 355-364.

PREMCHITT, J. (1978), Analysis and Simulation of Land Subsidence with Special Reference to Bangkok, *D. Eng. Thesis*, Asian Institute of Technology, Bangkok, 295 p.

SHIBAZAKI, T. (1971), Land Subsidence (in Japanese), *Sanseido Shinsho* No. 94. Tokyo, 205 p.

SHINDO, S. (1968), Hydrogeology of the Musashino Terrace (in Japanese), *Journal of the Japanese Geological Society*, Tokyo, Vol. 77, No. 4 pp. 31-53.

SODSEE, S. (1978), Geohydrology of the Chao Phraya Basin with Respect to Subsidence, *M.Sc. Thesis*, Asian Institute of Technology, Bangkok.

SOUTHERN KANTO LAND SUBSIDENCE INVESTIGATION COMMITTEE (1974), *Report on Land Subsidence in Southern Kanto Area* (in Japanese), Tokyo, 551 p.

TAKEUCHI, S., KIMOTO, S., WADA, M., MUKAI, K., and SHIMA, H. (1969), Geological and Geohydrological Properties of Land Subsided Areas - Case of Niigata Lowland, *Proceedings of Tokyo Symposium on Land Subsidence*, Vol. 1, pp. 232-241.

TERANAKA, K., TAGAMI, H., KANNO, T. and KONO, K. (1969), High Tides Countermeasure in Land Subsidence Area, *Proceedings of Tokyo Symposium on Land Subsidence*, Vol. 2, pp. 622-628.

TOKYO METROPOLITAN GOVERNMENT, Bureau of Construction (1976), *Land Subsidence and Flood Control Measures in Tokyo*, Brochure, Tokyo, 40 p.

TOKYO PUBLIC WORKS RESEARCH INSTITUTE (1969), *Metropolitan Tokyo Map of Subsurface Geology, 23 Wards* (in Japanese), Tokyo.

TOKYO PUBLIC WORKS RESEARCH INSTITUTE (1977), Land Subsidence due to Groundwater Withdrawal 1976 (in Japanese), *Annual Report*, Tokyo, pp. 267-433.

TOKYO PUBLIC WORKS RESEARCH INSTITUTE (1978), *Report on Land Subsidence in 1977* (in Japanese), Tokyo, 39 p.

UKENA, K., KANNO, T. and TERANAKA, K. (1969), Countermeasures Against Land Subsidence Mainly with Respect to the Basic Conditions of Internal Water Displacement During Flood Tide by Typhoon, *Proceedings of Tokyo Symposium on Land Subsidence*, Vol. 2, pp. 611-621.

WADACHI, K. (1940), Land Subsidence in West Osaka, *Second Report* (in Japanese), *Disaster Prevention Research Institute, Osaka*, Bulletin No. 3, pp. 1-41.

WADACHI, K , and HIRONO, T. (1939), Land Subsidence in West Osaka, *First Report* (in Japanese),*Disaster Prevention Research Institute, Oaska*, Bulletin No. 2, pp. 1-57.

WADACHI, K. and HIRONO, T. (1942), Land Subsidence in West Osaka, *Third Report* (in Japanese), *Disaster Prevention Research Institute, Osaka*, Bulletin No. 6, pp. 1-33.

YAMAGISHI, K. and OGASAWARA, H. (1975), Recent Trend of Land Subsidence in Tokyo (in Japanese), *Kiso-ko, Sogoh Doboku Ken*, Tokyo, Vol. 3, No. 2, pp. 53-59.

YAMAMOTO, S. (1976), Recent Trend of Land Subsidence in Japan, *Proceedings of the Second International Symposium on Land Subsidence*, Anaheim, California, pp. 9-15.

YAMAMOTO, S. (1977), Land Subsidence in Japan (in Japanese), *Japanese Society of Soil Mechanics and Foundation Engineering, Tsuchi-to-Kiso* (Journal), pp. 13-19.

LAND SUBSIDENCE IN BANGKOK, THAILAND: RESULTS OF INITIAL INVESTIGATION, 1978†

JERASAK PREMCHITT*

SYNOPSIS

A comprehensive investigation program is being conducted to find conclusive answers concerning the land subsidence problems in the Bangkok area. The work program for the first year is now completed. Many field instruments were installed in the area to observe the sinking of ground surface and the decline of piezometric level. It was found that subsidence of considerable magnitude and rate is occuring in the Bangkok area. Good correspondence was observed in the relationship between soil layer compression and piezometric level decline and this clearly reveals the link between groundwater pumping and land subsidence in the area.

INTRODUCTION

The city of Bangkok is situated on the Lower Central Plain of Thailand which consists essentially of a geological depression filled with alluvial and marine sediments. These sediments are many hundreds of meters thick, and consist of alternate layers of compressible clays and permeable sands and gravels. The sand and gravel strata constitute a number of distinct aquifers some of which are presently used for water supply. Over the entire area of the Plain is a surface deposit of very soft normally consolidated clay which is particularly compressible and which has been termed 'Soft Bangkok Clay'. Previous experiences in Mexico City, Tokyo, Taipei, California and many other places have shown that this combination of geological and hydrological conditions can result in land subsidence of great magnitude over large areas.

The problem of land subsidence in the Bangkok area was brought to attention about ten years ago by Cox (1968). In his report Cox discussed the similarities of the subsurface conditions in Bangkok and Tokyo, which had experienced serious subsidence problems at that time. Since then several researchers have studied the problem, but definite conclusions have not been reached as to what the magnitude and rate of subsidence is in the Bangkok area. It has been thought for many years in some quarters that a comprehensive program of investigation should be carried out as soon as possible to find out the conclusive answers before unrecoverable damages occur.

Beginning in February 1978, the National Environment Board commissioned the Division of Geotechnical and Transportation Engineering and the Division of Water Resources Engineering of the Asian Institute of Technology and the Royal Thai Survey Department to conduct a three and a half year comprehensive investigation into the problem. The activity of these three teams of investigators are closely coordinated and the organization of the research work concerning the subsidence problem can be represented as the chart given in Figure 1. It is now about one year since the start of the project and, even though definite conclusions cannot be drawn at present, substantial results obtained during this initial stage clearly indicate the magnitude of problems that are facing us at present.

The three main questions to be answered in this land subsidence investigation are (AIT, 1977):

(1) Is the present pattern of groundwater pumping causing subsidence in the municipal area of Bangkok and, if so, what is the present magnitude and rate of subsidence?

(2) How will the future patterns of groundwater pumping affect subsidence?

(3) What measures can be taken now and in the future to prevent subsidence or damage because of subsidence?

From the analysis of the data compiled and the results obtained from the field measurements during this initial stage, partial answer to the question number 1 can now be derived. A good correspondence was found in the relationship between the decline of piezometric level in the subsurface strata (which is caused by groundwater pumping) and the compression of the strata. The results from the first order relevelling of the Royal Thai Survey Department indicate that lowering of the ground surface on the order of 30-50 cm has occurred since the benchmarks were levelled many years ago. The measurement of the field instruments yielded the rate of the compression of the upper 50 m stratum, to be about 3 cm per year corresponding to the rate of piezometric level decline in this stratum of about 2 m per year. All these results strongly suggest the affirmative answer to question number 1. The full answer for the first question can be reached when the measurement of the deep compression indicators (over 200 m deep) and the relevelling over another year or two are conducted. The questions numbered 2 and 3 can be answered after the complete mathematical model study has been carried out.

The following sections will briefly describe the work conducted by the research team of the Division of Geotechnical and Transportation Engineering of AIT during the initial one year period.

LAND SUBSIDENCE IN BANGKOK
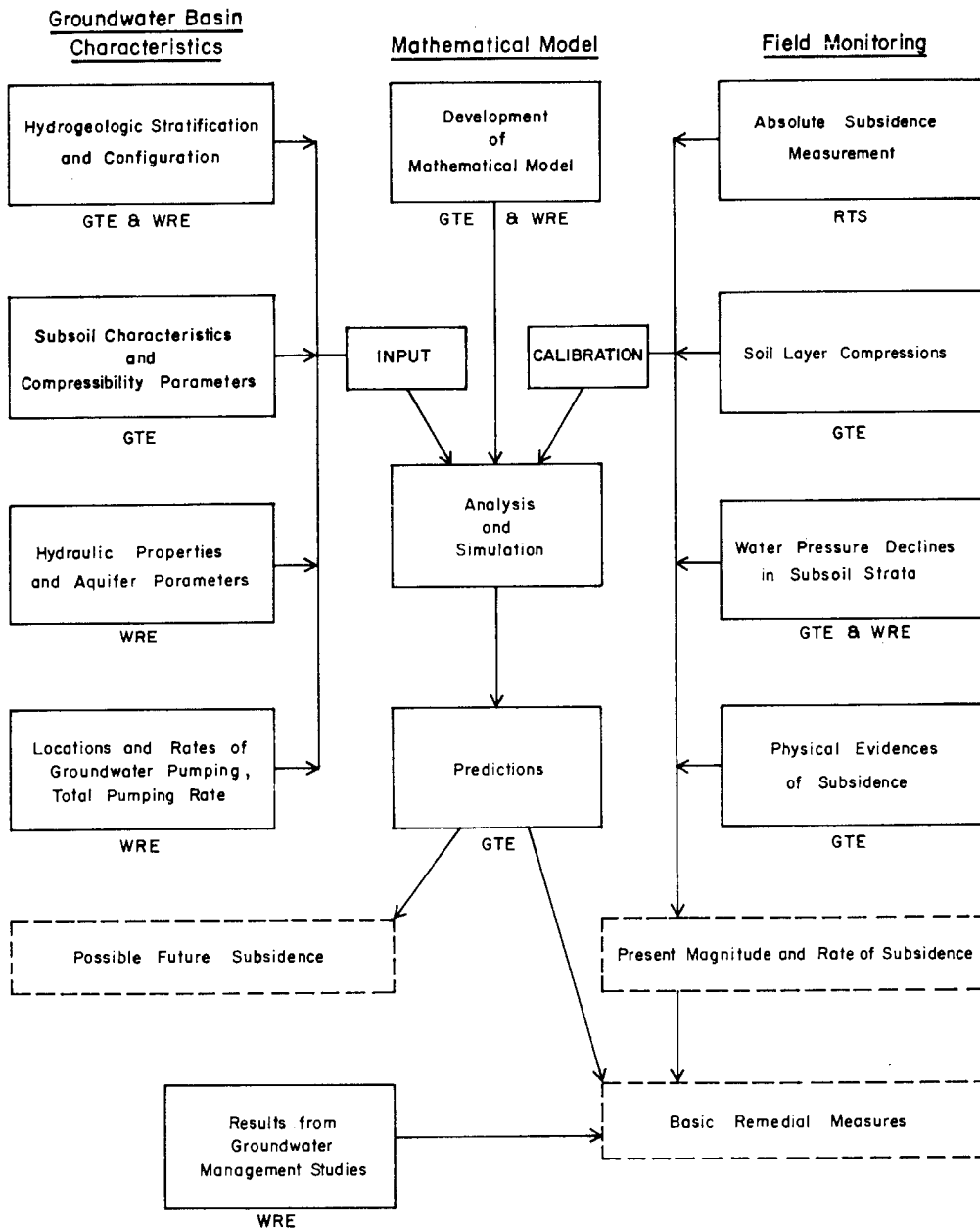
Fig. 1. Organization of research work.

SUMMARY OF PROJECT WORK

Study Area

In order to deal fully with the subsidence problem in the Bangkok area, it has been necessary to study the complete aquifer basin as a single geo-hydrological unit, since groundwater withdrawal and recharge at all parts of the basin affect the piezometric pressure changes in the many soil layers. The areal extent of this basin is delineated in Figure 2, where it can be seen to cover the whole Lower Central Plain and to extend well into the Gulf of Thailand. It is necessary to determine the geometry of the aquifers and
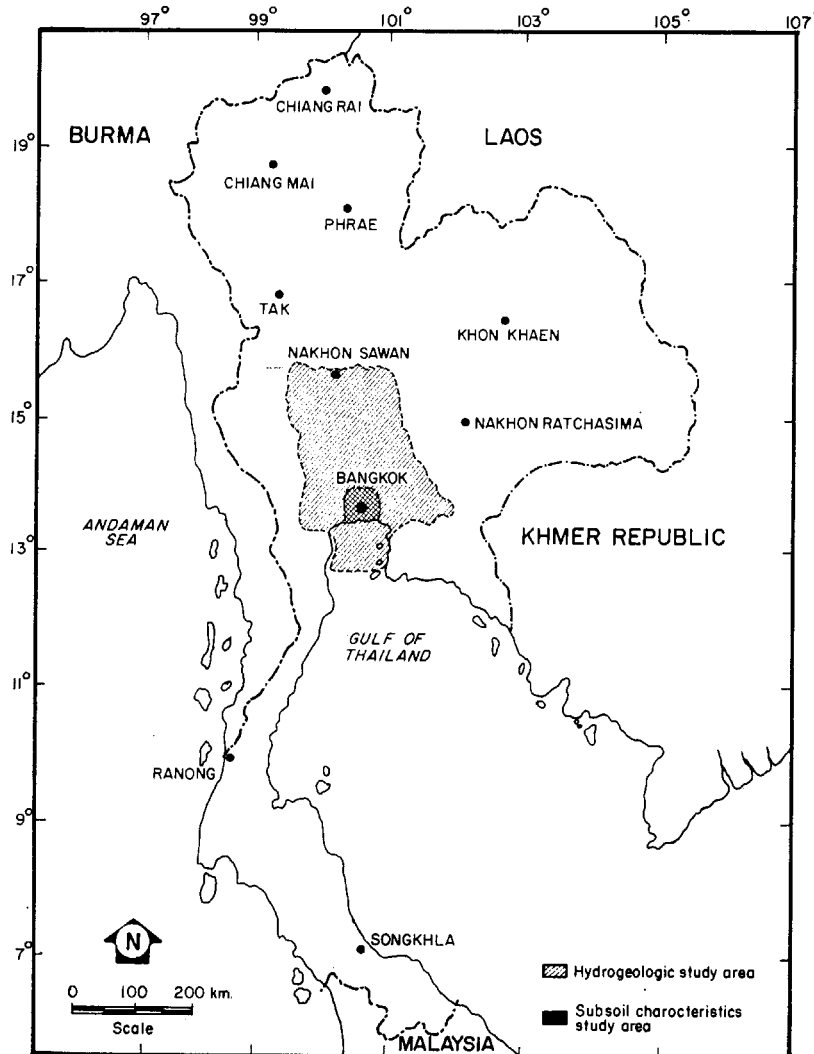

Fig. 2. Map of Thailand showing areas of the investigation of land subsidence due to deep well pumping.

their hydrogeological characteristics, together with the distribution of pumping and recharge in time and space. Considerable headway has been made towards establishing a complete hydrogeological description of the basin from records of existing well logs over the entire area of the Lower Central Plain of Thailand.

The investigation of surface subsidence has placed emphasis on the city of Bangkok and the adjacent areas. For this reason, the field instrumentation has all been installed in the smaller area shown in Figure 2. The soil sampling and associated laboratory testing has also been carried out completely in this area.

Data Compilation and Analysis

Well data. Some 2,000 well logs have been examined and transformed to a standard format to enable the subsequent establishment of the three dimensional geohydrological profile of the aquifer basin to great depth. The physical three dimensional model of the whole study area is being made to be the basis of the structure of the mathematical model for subsidence analysis.

The data obtained from the well logs have also been stored in the IBM 370-145 computer at the AIT Regional Computer Center using a modified version of the GRASP program (BOWEN & BOTBOL, 1975).

Soil data. Nearly all the existing information on the compression characteristics of the soils in the Bangkok area are restricted to the upper 30 m of the soil profile. Between 30 m and 100 m the data is sparse, and no information exists for depths greater than 100 m. The reliable existing data on compression characteristics have been compiled for more than 300 boreholes.

Field Instrumentation

In this investigation instruments have been developed to make it possible to monitor the compression of the soil layers associated with measured pore pressure changes in those layers. The instruments are:

(1) Type I benchmarks (BM I),

(2) Type II benchmark (BM II),

(3) surface compression indicator (CI-1),

(4) auger tip compression indicator (CI-2),

(5) cone tip compression indicator (CI-3),

(6) open standpipe piezometer (P), and

(7) water table observation well.

Twenty four locations were selected for the installation of instrumentation. These locations are marked on Figure 3 and listed in Table 1. All the
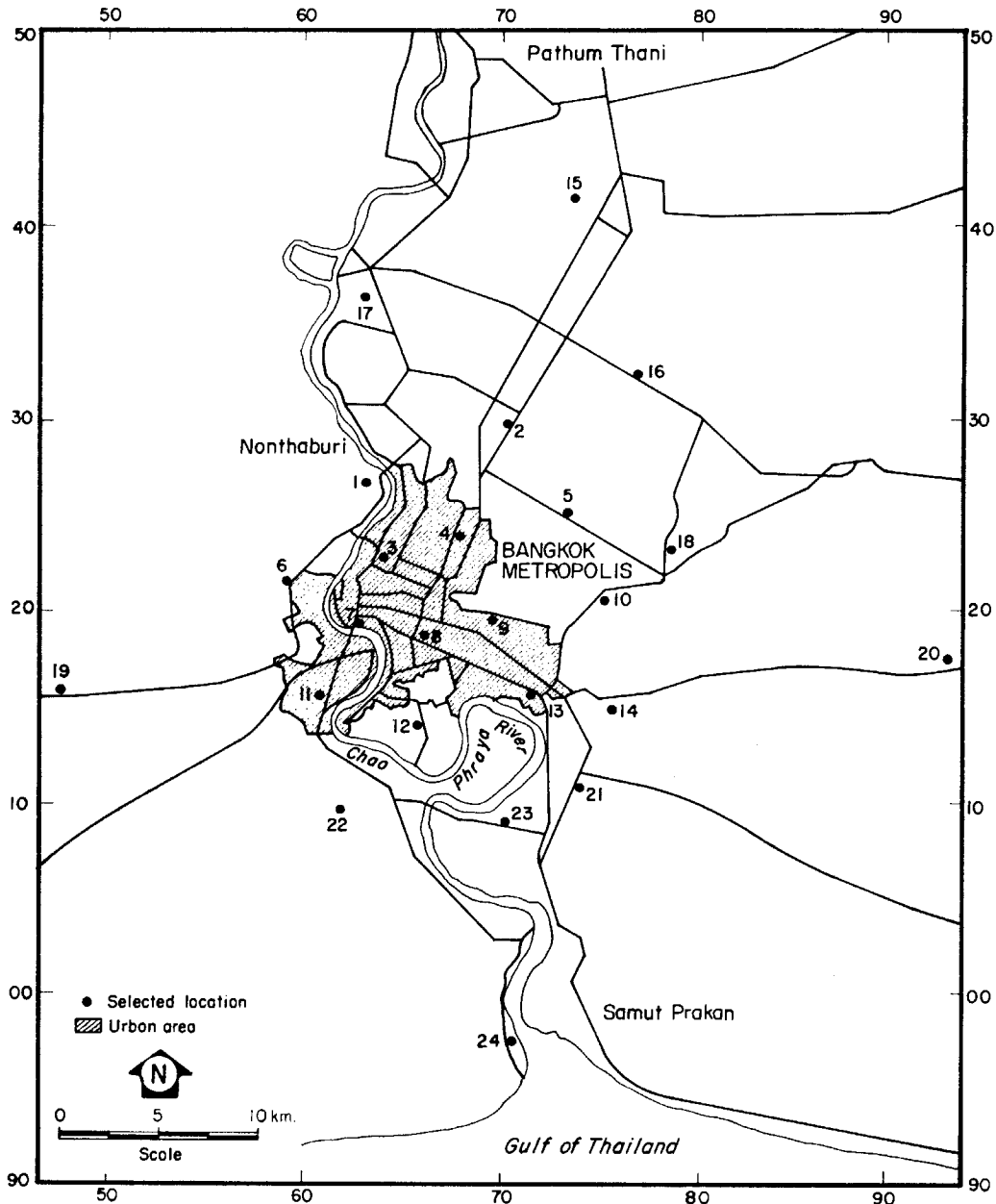
Fig. 3. Locations for soil sampling and field instrumentation.

Table 1. Locations of soil sampling and field instrumentation in Bangkok and adjacent areas.

Station Number	Location	Grid Reference
1	Electricity Generating Authority	633269
2	Lang Development Department	704299
3	National Assembly	641229
4	Post & Telegraph Department	679239
5	Khuru Sapha Printing	735250
6	Wat Plengvipassana	590215
7	Wat Rajaburana	625195
8	Chulalongkorn University	658186
9	Prasanmitr Campus	696198
10	Ramkamhaeng University	750212
11	Somdej Phra Pinklao Hospital	611158
12	Phra Mae Mary Sathupradit	661140
13	The Livestock Trading Cooperation Ltd.	716156
14	Wat Rajasathathum	758151
15	Don Muang Airport	740412
16	Aviation Division, Police Department	769320
17	Royal Irrigation Department, Pak Kret	632365
18	National Housing Authority	785231
19	Thai TV Color Channel 3	475157
20	Lad Krabang Campus, KMIT	926179
21	Meterological Department, Bang Na	740112
22	Thonburi Campus, KMIT	620095
23	Animal Export Station	704091
24	Pom Phrachul	704975

Grid reference number based on Military Grid System on the 1:50,000 map sheets.

instruments were installed in the 50 m depth range in this first year of the investigation.

It should be noted that the benchmarks (BMI & BMII) provide reference points for the Royal Thai Survey settlement measurements as well as acting as a compression indicator for the clay layer above the sand stratum. The compression indicators (CI-2 & CI-3) and the surface compression indicator (CI-1) act together to provide the compression characteristics of the stiff and the soft clays above the sand layer.

Soil Sampling and Testing

Drilling and sampling. Soil investigations have been carried out at the 24 locations shown in Figure 3. One hole was drilled at each site to depths

of up to 50 m to reach the first sand stratum (Bangkok Aquifer), continuous samples being obtained from the hole.

Laboratory testing. The samples obtained from the 24 boreholes were logged in great detail and large number of laboratory tests were carried out at AIT on the soils from the sampling locations. These tests included the following.

(1) Determination of the basic properties of the soils, including natural water content, total unit weight, particle size distribution, specific gravity, Atterberg limits and salt content of the pore water. These tests were performed at very close depth intervals and a total of over 500 tests were conducted for each station.

(2) Measurement of coefficient of permeability of the sand from the upper sand layer.

(3) Determination of the compression characteristics of the clays and the sands by means of oedometer tests. About 6 tests were conducted for each station.

Development of Mathematical Model

The development of the mathematical model to simulate the land subsidence process over an entire aquifer basin has now been completed. The finite difference technique was applied to the governing equations to produce a general digital computer model which is capable of application to any aquifer basin. The model has been tested to establish its validity and flexibility, and parametric studies have been conducted to assess the relative importance to the subsidence problems of the hydrological and consolidation properties of an aquifer basin. Successful simulations have been achieved of time-dependent water pressure declines and subsidence for idealized aquifer models.

ESTABLISHMENT OF GEOLOGIC CONFIGURATION

The geologic information of the subsurface strata in the Lower Central Plain of Thailand was derived from the existing boring records of the groundwater wells over the area. The extensive groundwater well data, which covers the whole Lower Central Plain, was compiled. Lithological correlations based on these existing well log data were employed to delineate the stratigraphy and aquifers of the Lower Central Plain.

It was found that the sedimentary deposits in the Lower Central Plain of Thailand vary a great deal in type of sediments, thickness, and lateral

and vertical distribution. In the central part of the basin, interbeds several hundred meters thick of sand, clay and gravels occur, whereas toward the edges only thin veneers of sediments lie on top of shallow bedrocks.

On the basis of the nature of stratification, the Lower Central Plain Basin sediments can be divided into eight geohydrologic regions (Fig. 4) as follows.
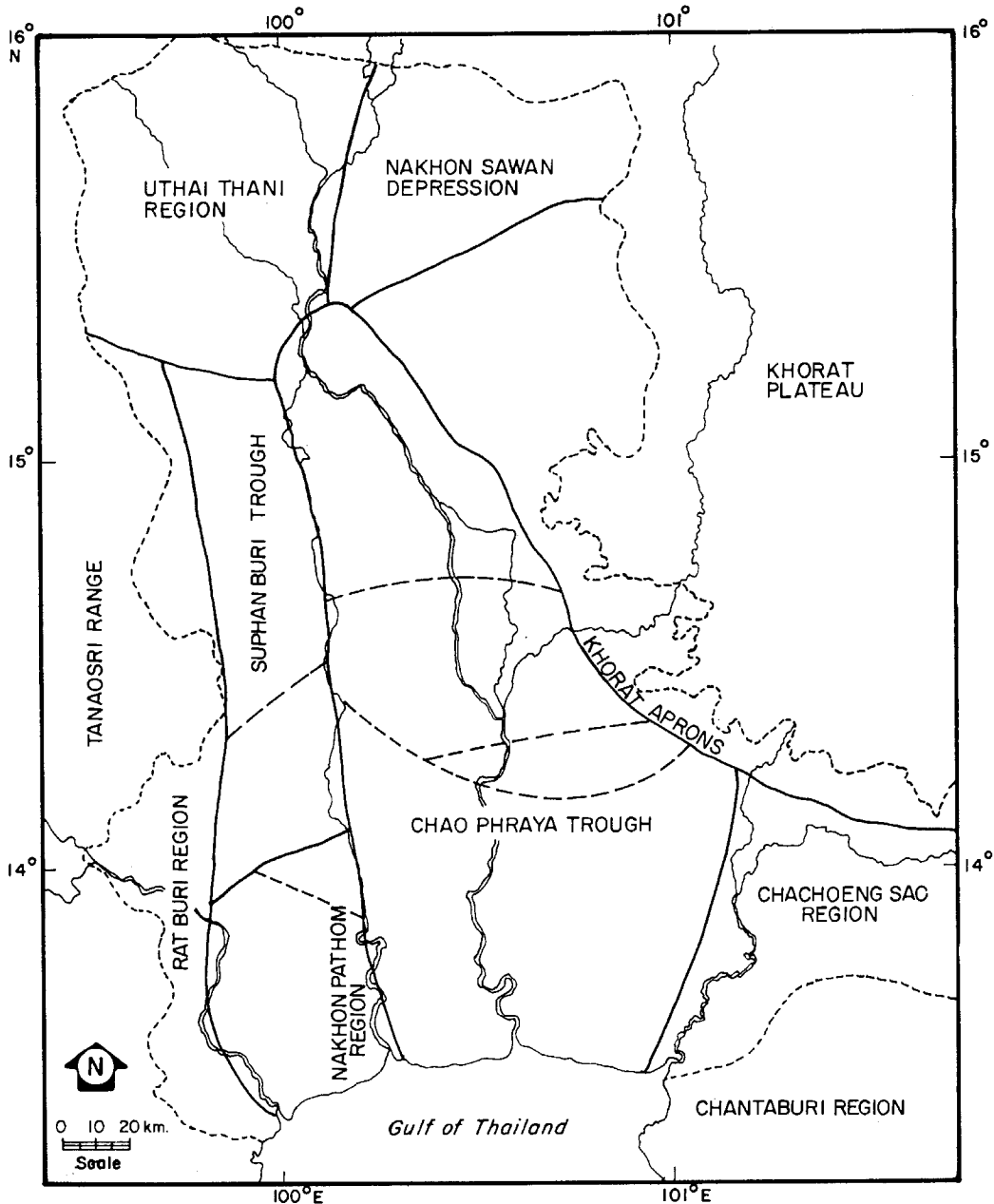

Fig. 4. Subdivision of the geohydrologic regions of the Lower Central Plain of Thailand.

The Chao Phraya Trough

The Chao Phraya Trough covers an area spanning both sides of the Chao Phraya river from the Gulf of Thailand to Uthai Thani Province. The southern boundary of this trough cannot be defined due to the lack of information in the Gulf of Thailand.

Bangkok subregion. The majority of wells data available to depths of 150 to 200 meters indicate the existence of four sand layers (aquifers), from which most of the water in Bangkok area is derived, each separated by a clay layer. The three dimensional fence diagram of the subsurface strata in the Bangkok area is given in Figure 5.

Ayutthaya-Pathum Thani subregion. This subregion is located in between Ayutthaya and Pathum Thani. The sediments in these two areas are different in sequence and thickness. Ayutthaya-Pathum Thani subregion can thus be defined as the area of discontinuation. The records of a few wells drilled in this are indicate that sediments are mostly clay mixed with sand and gravel.

Ayutthaya-Angthong subregion. The sediments found in this subregion can be distinguished from the Bangkok subregion sediments by their sequence of deposition and thickness. Numerous thin layers of sand (less than 10 meters thick) are found within the depth of 130 meters.

Singburi subregion. The trough in this subregion is narrow and limestone was encountered near the boundary on both sides at about 50 to 80 meter depth.

Uthai Thani Region

The sediments in this area overlie on top of bedrocks such as granite, limestone, quartzite etc. Two beds of gravel and sand are found. A thin clay layer, ranging from 5 to 15 meters thick, is found in between these sand beds.

Nakhon Sawan Depression

On the northern part of this alluvial fan complex lies the Nakhon Sawan Depression. The sediment is mostly clay with lateritic sediments. A sand bed extends from north of Bung Boraphet to south-east of Chum Saeng area where it encounters limestone beds there.
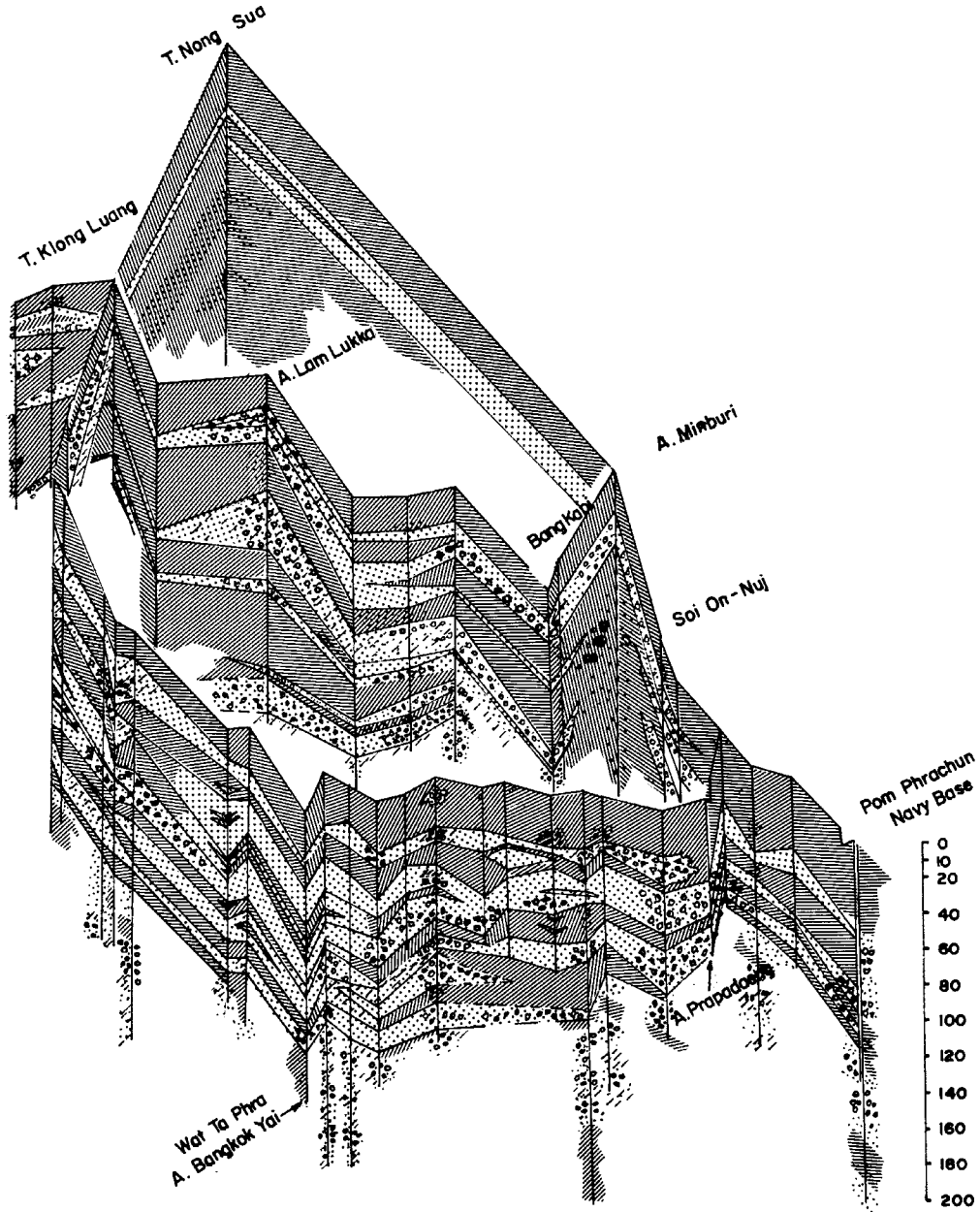
58

Fig. 5. Fence diagram of subsurface strata in the Bangkok and adjacent areas (after Sodsee, 1978).

The Khorat Aprons

As evidenced by exposed rock outcrops, the sediments in this area were deposited on top of the bedrock. No sand layer of a wide extent has been found.

59

Chachoengsao Region

A few sand layers of about 2 to 4 meters thick are found at depths of about 50 meters. These layers pinch out in the thick clay layer nearby. There is one gravel lens of about 10 to 35 meters thick in the northeast part of the province.

Suphan Buri Trough

This trough can be divided into two subregions. The first is the area south of Changwat Suphan Buri where small sand and gravel layers are found within 120 meters depth. Their thickness is about 4 to 10 meters. The second subregion is the area north of the first one where only lenses of gravel and sand occur. Some of these lenses appear to be alluvial fan type deposits.

Nakhon Pathom Region

The sands and gravels are very thick compared to the clay layer can be traced continuously to the Pathum Thani area. These sand and gravel layers can also be traced westward until they meet the rock outcrop in the area along the west bank of the Mae Klong river. This is probably the recharge area of the aquifers in this basin. The aquifers found are not thicker than 20 meters and generally are about 4 to 10 meters thick.

Ratburi Region

This region is adjacent to the mountains and its elevation is relatively high compared to the other areas. Outcrops of various types of rock are found in this area.

The majority of deposits are found overlying bed rock and they are about 20 to 30 meters thick. The type of deposits are sandy gravels, sandy clays, marls or lateritic clays.

INSTALLATION OF FIELD INSTRUMENTS

Field instruments for the observation of pore water pressures and soil layer compressions were installed at 24 stations in the Bangkok area. These stations are spread over Bangkok and adjacent areas and they constitute a monitoring network that provide sufficient coverage over the subsidence area. The field instruments installed during this Phase I work are intended for the observation of layer compressions and pore water pressures in the

upper 50 m soil stratum. The upper stratum is highly compressible and it is anticipated that more than half of the magnitude of the subsidence is due to the compression of this upper layer (PREMCHITT, 1978).

Type I Benchmarks (BMI)

Type I benchmark provide the reference point for the Royal Thai Survey settlement measurements as well as acting as a compression indicator for the upper clay stratum. Clay layer compressions can be monitored either by a precise level or by a dial gauge. The schematic diagram describing the details of Type I benchmark is given in Figure 6a.

The type I benchmark was installed in the hole that was drilled to obtain the continuous soil samples. The surface reference point was installed coaxially with the outer casing and the inner pipe (Fig. 6a). With this type of benchmark, accurate measurements (apart from the usual levelling technique) of the soil layer compressions can be made by using a dial gauge to measure the change in elevation of the inner pipe (which rests on a deep stratum) with respect to the outer pipe, which rests almost on the ground surface (surface reference point).

Type II Benchmarks (BMII)

The type II benchmark serves the same purpose as the Type I benchmark except that the observation of layer compression can be made only by levelling. The installation procedures for the two types of benchmarks are almost the same except that Type II benchmark did not require the coaxial surface reference point (refer to the schematic diagram of Type II benchmark in Fig. 6b). The observation for this type II benchmark require the surface compression indicator, CI-1 (surface reference point), in order to get the compression of the upper clay stratum.

Surface Compression Indicators (CI-1)

The surface compression indicator is designed to act as the surface reference point since it settles down by the same magnitude as the ground surface. It was installed to the depth of 1.0 m (Fig. 7). Considerable efforts have been made to ensure that the surface compression indicator acts together with the ground surface without any relative movement between the two.

Auger Tip Compression Indicators (CI-2)

The auger tip compression indicator is equipped with a 30 cm diameter auger head, which acts as a screw to install the compression indicator below

BM I
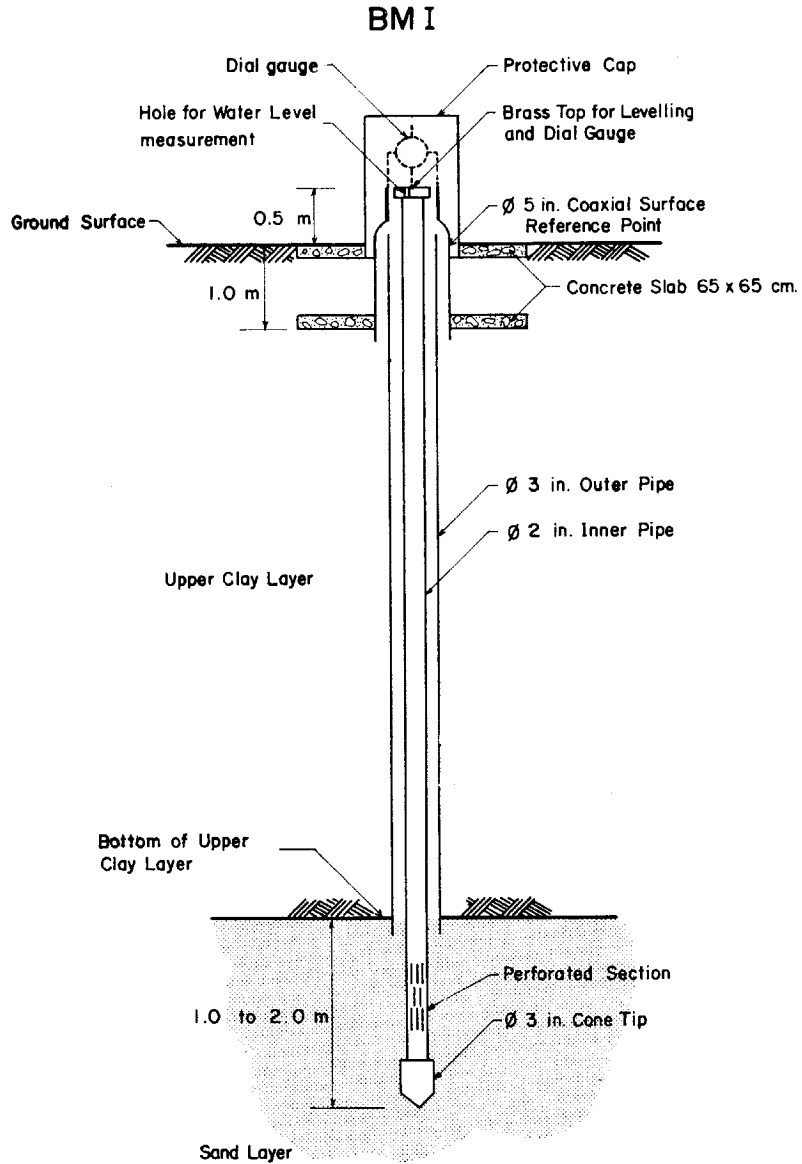
Fig. 6a. Schematic diagram of benchmark BM I.

the ground surface to the desired depth and also serves as an anchorage to prevent the movement of the indicator with respect to the soil mass at the auger head. The auger tip compression indicator was installed for the purpose of measuring the compression of the upper 10 m zone of the soft clay stratum. The movement with respect to the surface compression indicator of this auger tip compression indicator specifies the compression of the clay
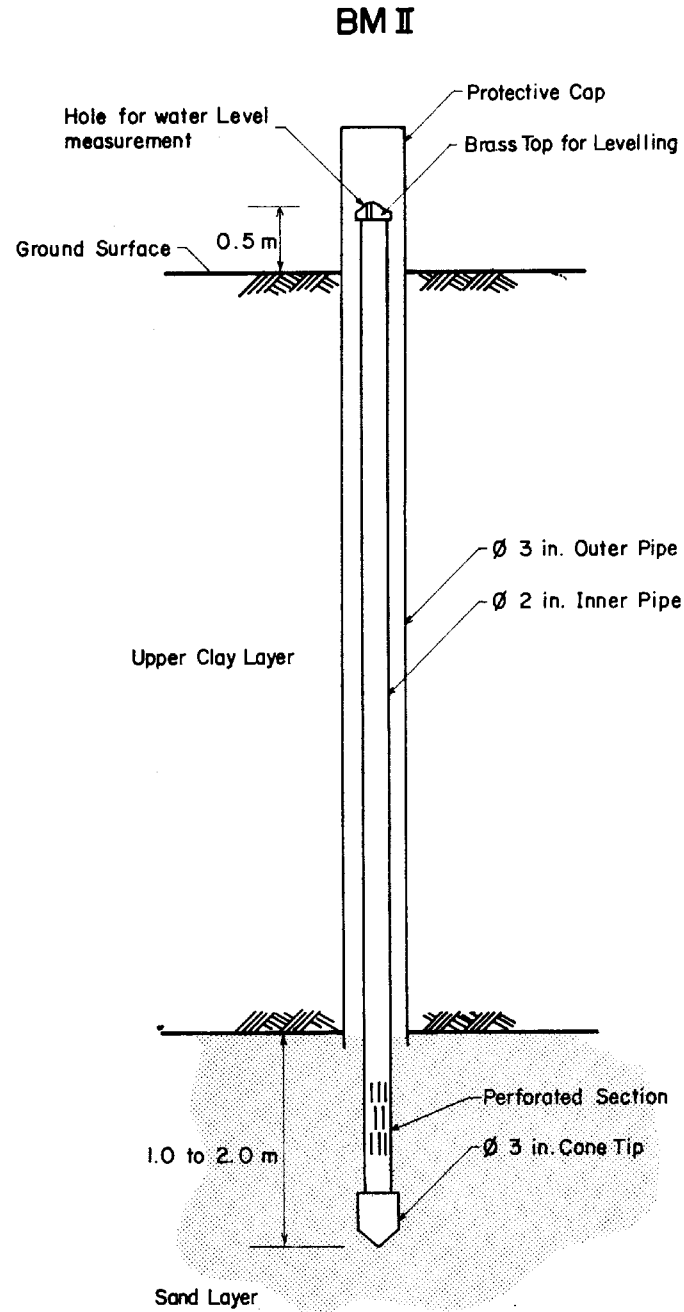
BM II

Fig. 6b. Schematic diagram of benchmark BM II.
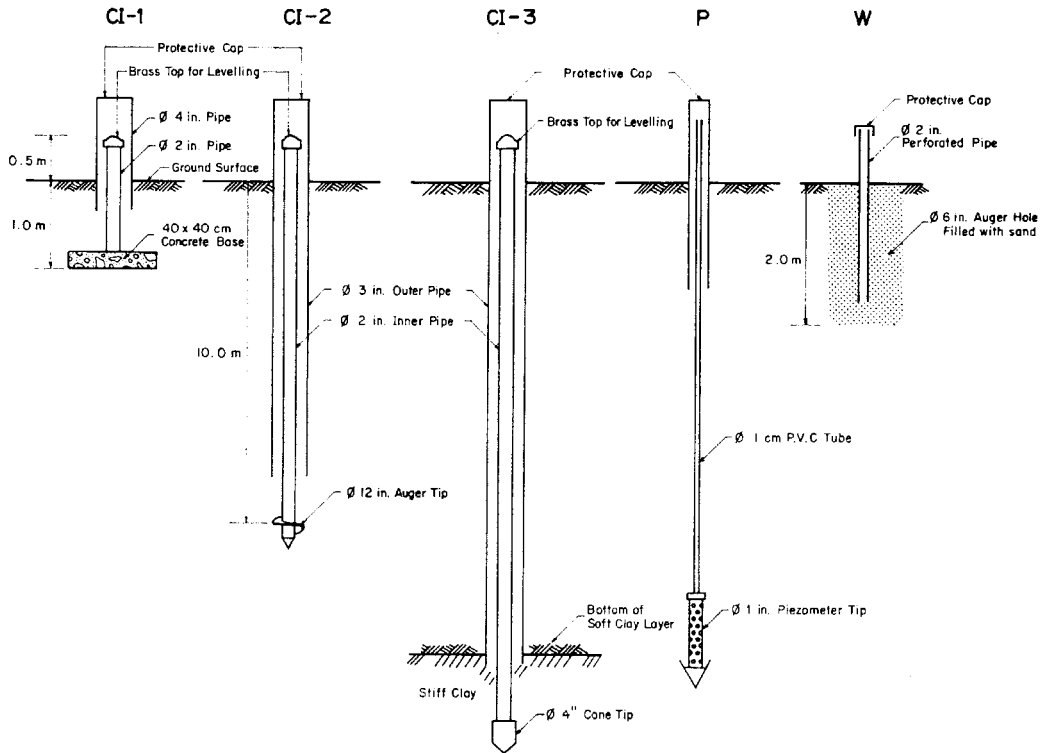
Fig. 7. Schematic diagrams of compression indicators, piezometer and water table observation well.

from ground surface down to the depth of 10.0 m. The schematic diagram of this compression indicator is given in Figure 7.

Cone Tip Compression Indicators (CI-3)

The cone tip compression indicators was designed to be hammered down through the soft clay layer and to rest on the top of the stiff clay layer (Fig. 7). Since the thickness of the soft clay layer varies from location to location, the depths of CI-3 vary accordingly. This indicator is intended to be an instrument for the measurement of the compression of the whole soft clay stratum. The relative movement of the inner tube of this cone tip compression indicator with respect to the surface compression indicator is the compression of the whole soft clay stratum.

Open Standpipe Piezometers (P)

Open standpipe piezometers were installed for the observation of pore water pressure at various depths at each station. The piezometers were in-

64

stalled at depths of about 2.5, 5 and 10 m and at every 5 to 10 m interval in the deeper zones. The schematic diagram of this piezometer system is given in Figure 7. The depth from the top of the standpipe to the ground water level in the pipe was measured by an electric wire detector, and this depth was used to calculate the water pressure at the level of the piezometer tip.

Water Table Observation Wells (W)

An observation well was used to observe the level of groundwater table at each station. A perforated pipe was installed in a pre-bored auger hole down to a depth of about 1.60 m (Fig. 7). The hole was then filled with sand to enable to flow of groundwater from surrounding soils. The water level in the observation well was also observed by means of an electric wire detector.

MEASUREMENT OF WATER PRESSURE

Pore water pressures were observed from the open standpipes of the piezometers as well as from the inner tubes of the benchmarks. The water pressures at various depths in the clay layer were measured from the piezometers while the water pressures in the upper sand stratum were measured from the benchmarks. The levels of water in the standpipes of piezometers and in the inner tubes of benchmarks were measured by means of the electric wire galvanometer (Fig. 8). These measurements were taken at monthly intervals.

The variations with time of the pore pressures at various depth were recorded at each station The decline in pore pressure from the hydrostatic condition was found to be large at all locations. The pore pressure decline induces an increase in the effective stress which in turn causes the compression of the soil layer. The typical pore pressure distributions with depth and the variation of piezometric level with time are shown in Figure 9, which shows the results of measurement made at Chulalongkorn University.

The drop in pore water pressure in the upper clay stratum is induced by the pressure decline in the adjacent sand layer (the Bangkok Aquifer) which is in turn, caused by the pumping of groundwater from the Bangkok Aquifer itself or from the deeper aquifers. It is now recognized that there exists many interconnections between various aquifers underneath the Bangkok area. The heavy pumping in deep aquifers (e.g. Phra Pradaeng and Nakhon Luang Aquifers) will undoubtedly cause water pressure decline in the Bangkok

Galvanometer

Double Wires Exposed at Lower
Ends with a Scale Graduated on

Cord Reel

h_1

Ground Surface

h_2

Open Stand Pipe

d

Water Level in the Pipe

h_3

Piezometer Tip
(or Perforated Section of BM)

$u = h_3 \gamma_w$

$(h_3 = d + h_1 - h_2)$
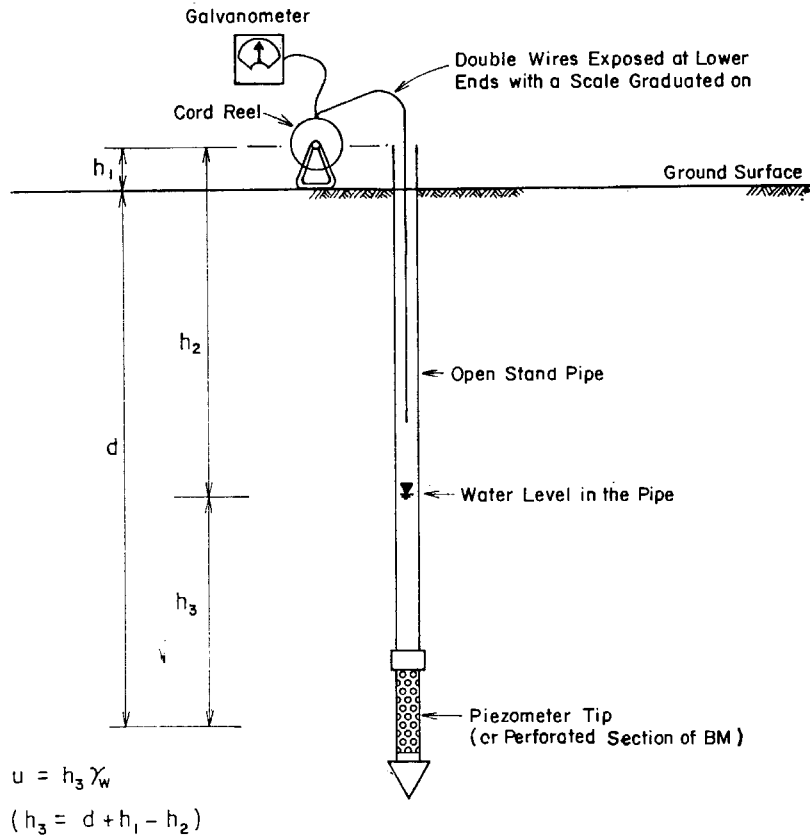
Fig. 8. Schematic diagram of the observation of pore water pressure.

Aquifer, even though the magnitude of drop may be smaller than those in the deep aquifers.

It was found from field measurements (Fig. 9) that the pressure drops in the stiff clay layer, which is adjacent to the Bangkok Aquifer, are large while the pressure drops in the soft clay layer are small. Down to a depth of about 10 meters the magnitudes of pressure drop are very small. Below this depth the pressure decline becomes larger and the greatest declines were found in the Bangkok Aquifer for all stations. The magnitude of pressure drop in the Bangkok Aquifer was about 20 t/m² (20 m of water column) in the central Bangkok area. The drop is relatively small, about 6-8 t/m² on Thonburi side of the Chao Phraya River, but it is large in the southern part of Bangkok. The contour of the piezometric level below ground surface in the Bangkok Aquifer over the Bangkok and adjacent areas is shown Figure 10.

66
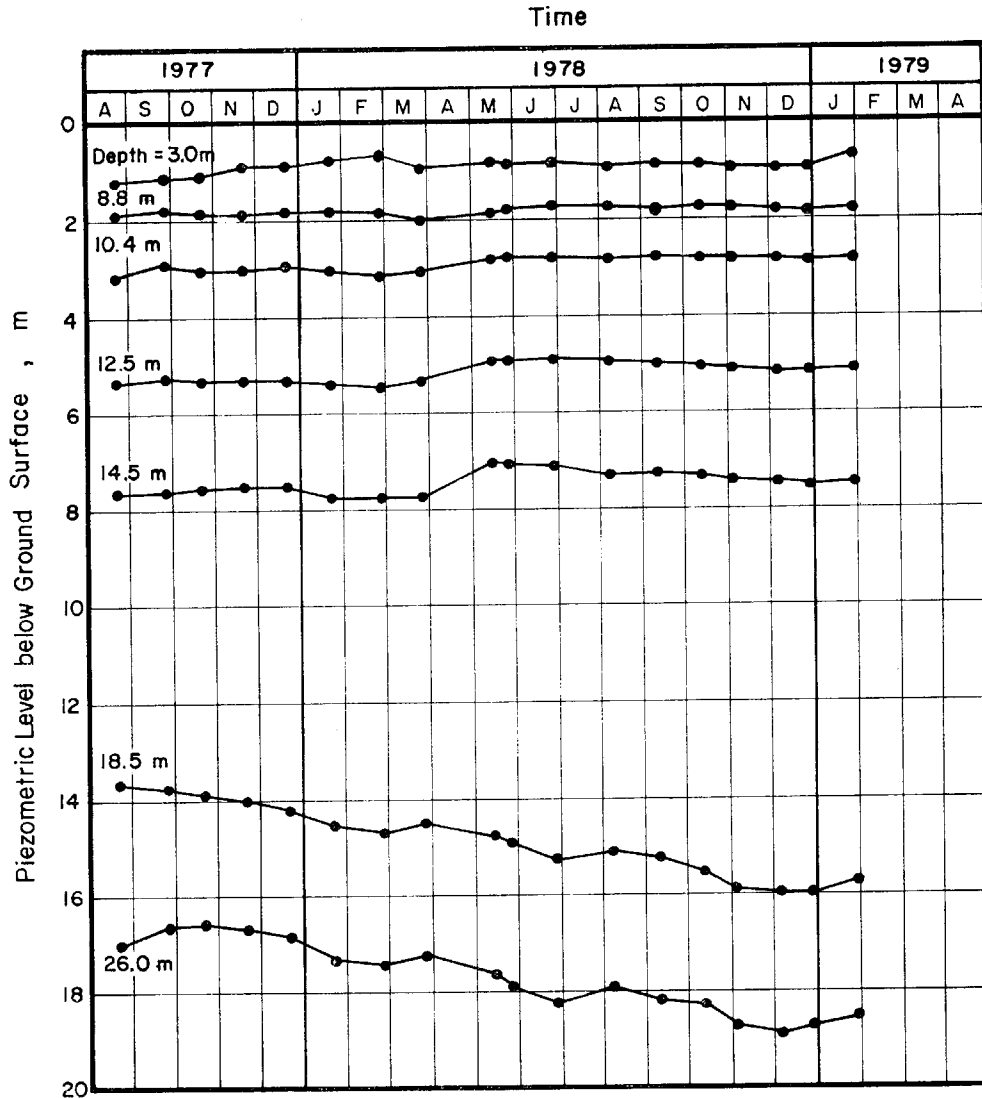
Fig. 9a. Pore pressure declines in upper clay layer at Chulalongkorn University.
Variation of piezometer level with time.

MEASUREMENT OF SOIL LAYER COMPRESSION

In this initial work, instruments were installed to observe the compression of various layers in the uppermost clay stratum. The compressions of the upper 10 m zone of soft clay layer were observed from the auger tip compression indicators (CI-2) while the compressions of the whole soft clay layer (about 12 to 20 m deep) were observed from the cone tip compression indicator (CI-3). The compression of the uppermost clay stratum (including

67

Fig. 9b. Pore presure declines in upper clay layer at Chulalonkorn University.
Variation of pore pressure with depth.

soft and stiff clay layers) were observed from the benchmarks (BM). All these compressions were taken as the relative movements between the instruments and the surface compression indicators (CI-1). The compression of the intermediate layers can also be determined from the relative movements between two of the instruments, such as the CI-2 and CI-3. A schematic diagram of the observation of soil layer compression is presented in Figure 11.

The monitoring of compression indicators was carried out simultaneously with the monitoring of piezometers. The elevations of CI-2, CI-3 and BM

68
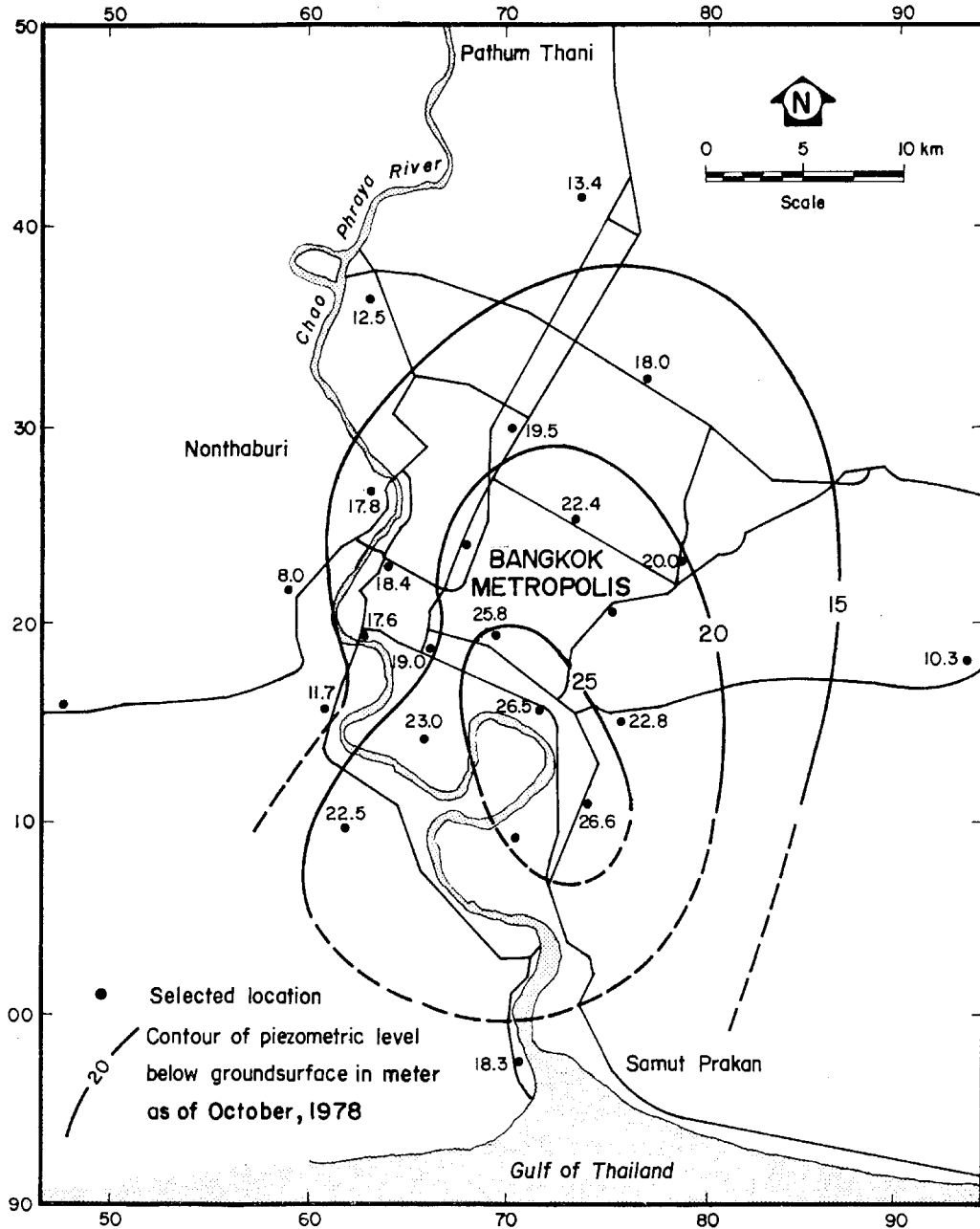
Fig. 10. Contours of the piezometric level below ground surface in the Bangkok Aquifer.

with respect to CI-1 were recorded. The initial elevation readings were taken as the first observation. The changes in the elevations of these instruments with respect to CI-1 as derived from the later observations indicate the compression (or swells) of the respective clay layers.
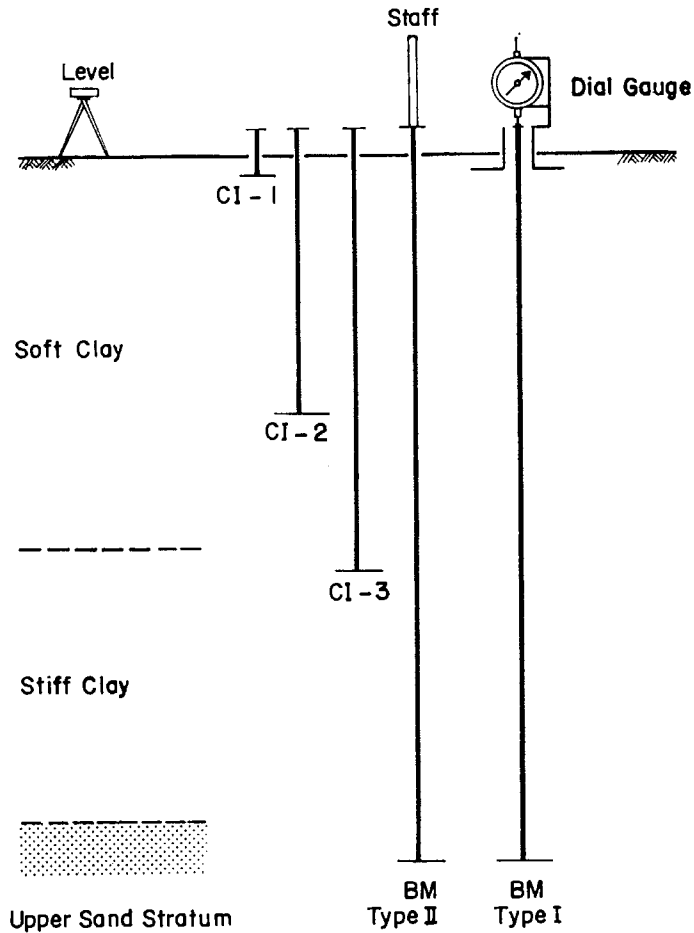
Fig. 11. Schematic diagram of the observation of the field
instruments for soil layer compressions.

For the Type I benchmarks, the observation can be conducted either
by levelling or by using a dial gauge. Both methods were used in the moni-
toring and the results obtained are in impressive agreement with one another.
The levelling method provides the results with the accuracy of 0.5 mm while
the dial gauge gives an accuracy of 0.01 mm. It seems that the dial gauge
has the advantages in short term observations while the levelling is more
suitable for the long term observation.

At various stations in which the instruments have been installed for a
sufficiently long period, considerable soil layer compressions were observed,
especially for the deep instruments. Table 2 shows the observed compressions
of the whole upper clay stratum as observed from the benchmarks at four

70

stations where the benchmarks were installed to the depth of about 50 m (the deepest ones). The observed compressions are about 15 to 20 mm during the period of 6 months at all of these stations. These results clearly indicate that subsidence of considerable magnitude is occurring in the Bangkok area.

Table 2. Monitoring results for compression of various soil layers at four stations.

Station Number	Period months	CI-1 to CI-2		CI-1 to CI-3		CI-1 to BM	
		Depth Interval, m	Compression, mm	Depth Interval, m	Compression, mm	Depth Interval, m	Compression, mm
9	6	1.0-10.0	4	1.0-14.9	8	1.0-47.0	19
13	6	1.0-10.0	6	1.0-15.4	13	1.0-54.7	17
21	7	1.0-10.0	7	1.0-16.0	18	1.0-53.0	27
24	8	1.0-10.0	1	1.0-18.2	18	1.0-47.0	27

CI-1 = Surface Compression Indicator
CI-2 = Augur Tip Compression Indicator
CI-3 = Cone Tip Compression Indicator
BM = Benchmark

Apart from the compression of the whole upper clay stratum, the compressions of the soft clay layer and the upper 10 m zone were also recorded from CI-2 and CI-3. The monitoring records for the four stations, showing the compression in various layers, are given in Table 2 and are also illustrated in Figure 12.

From Figure 12 it can be seen that there are small compressions in the upper 10 m zone of the soft clay layer, which correspond with the very small pore water pressure declines in this zone as demonstrated in the previous section. Considerable magnitude of compression occurred in the soft clay layer below 10 m depth down to the interface of the soft clay-stiff clay layer (the depth of CI-3) and about the same magnitude of compression occurred in the stiff clay layer down to the depth of the benchmark (about 50 m in this case). The piezometric level below ground surface at these stations are also plotted in Figure 12, in which the correspondence between soil layer compression and piezometric level is clearly seen.

Based on the observation during this initial stage, it may be deduced that the rate of the compression of the upper 50 m zone of the subsurface strata underneath the Bangkok area in this short period is about 3 cm per year. It is expected that the rate of surface subsidence, which includes the compression of the whole deeper soil strata, is higher than this.
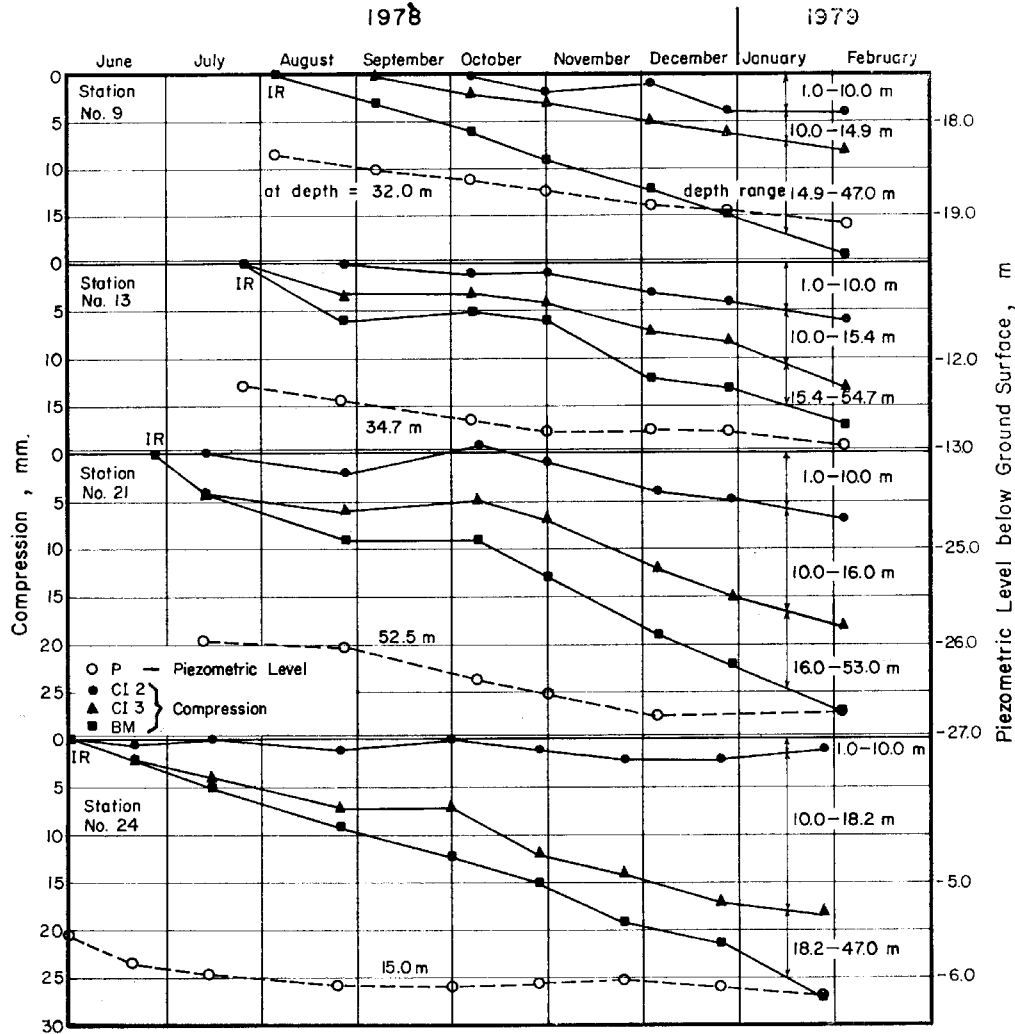
Fig. 12. Monitoring results of the instruments at 4 stations in the initial period.

RELATIONSHIP BETWEEN PIEZOMETRIC LEVEL DECLINE
AND SOIL LAYER COMPRESSION

The subsidence of a ground surface can result from the consolidation of soil deposits due to the lowering of the piezometric level or the piezometric pressure. When water is removed from a water bearing subsoil stratum (aquifer), the piezometric level in the stratum will be lowered, unless there is sufficient recharge water to replace the quantity withdrawn. The lowering of piezometric level in the aquifer creates a hydraulic gradient between the aquifer and the adjacent clay layer (aquitard). As a result, pore water flows

out of the highly compressible clay layer (with an increase in effective stress) and consolidation takes place. The total stress however, remains constant during the consolidation phase. The effective stress is given as follows:

$$\bar{\sigma} = \sigma - u \qquad \qquad \cdots\cdots\cdots (1)$$

and $\qquad \Delta\bar{\sigma} = -\Delta u = -\gamma_w\,\Delta s \qquad \qquad \cdots\cdots\cdots (2)$

where $\bar{\sigma}$ is the effective stress, σ is the total stress, u is the pore water pressure, γ_w is the unit weight of water and s is the piezometric level referring to a datum. It is obvious that reduction in pore water pressure causes an increase in the effective stress which in turn causes the compression of the soil layers. In general, the compression of a highly compressible clay stratum will be substantial, while the compression of a firm stratum (aquifer) will be small.

If the pressure drop Δu, in a soil stratum of thickness H is uniform throughout the thickness, the compression of the layer can be found from

$$\Delta\rho = \frac{\Delta\bar{\sigma}}{D}\cdot H$$

$$= \frac{-\Delta u}{D}\cdot H$$

$$= -\Delta s\cdot\frac{\gamma_w H}{D} \qquad \qquad \cdots\cdots\cdots (3)$$

where ρ is soil layer compression, and D is the constrained modulus of elasticity of the soil skeleton. It can be seen that there exists a straight line relationship between the piezometric level decline in a soil stratum and the compression of the stratum in the form:

$$\Delta\rho = -\Delta s\cdot K \qquad \qquad \cdots\cdots\cdots (4)$$

where K is a constant and equal to $\gamma_w\,H/D$.

In the investigation, the compression indicators and piezometers were installed at many depths to record the soil layer compressions and piezometric levels at each station. The compression was measured between two compression indicators which were installed at different depths. The piezometric levels at depths between these two compression indicators were also recorded from the piezometers. In order to observe the relationship between the piezometric level in the soil stratum and the compression of the stratum that actually occurred in the field, the changes in piezometric

level which were measured at mid-depth between two compression indicators were plotted against the compression that was observed from the two compression indicators. Provided that the piezometric level decline at mid-depth of the layer represents the average decline over the layer thickness and the layer is uniform throughout, a straight line relationship is expected according to the effective stress principle expressed in Equations 1 and 4.

It was found that the plots showed very good correspondence according to the above analysis (Figures 13 and 14). The plots can be divided into two groups, the first is for the shallow zone (0 to 10 m) and the second is for the deep zone (deeper than 10 m). In the shallow zone the present piezometric level decline is small and the level will fluctuate in a certain range without any substantial lowering of level for a long time, while the piezometric level in the deep zone was found to decrease with every passing year. These phenomena can be clearly seen in Figure 9, which shows the observations from Chulalongkorn University. The Δs - Δp plot for the deep zone showed the approximately continued straight line (Fig. 13) while the plot for the shallow zone showed the partial cyclic variation (Fig. 14). The complete picture of the relationship will be found when the observation is continued for a one year period. At that time it is expected that any seasonal variation will come to a complete cycle, and the long term trend of continued subsidence will be revealed.

The Δs - Δp plot, for example as given in Figures 13 and 14, undoubtedly proves the straight line relationship between piezometric level decline in the soil layer and the compression of the layer.

The relationship also tells us that we are observing a very large scale consolidation process (involving a soil layer 50 m thick and at least 100 square kilometers in area). The observation of the consolidation process on this scale, and with the full field instruments installed, has seldom been carried out in the past. The study by LOFGREN (1969) is one such example.

CONCLUSIONS

(1) The investigation of land subsidence in the Bangkok area has been carried out for one year and the results obtained so far strongly indicate that subsidence of considerable magnitude and rate is occuring.

(2) The geohydrological profile of the Lower Central Plain of Thailand is established and it is used as the basis of the structure of mathematical model for the prediction of land subsidence in Bangkok.

74
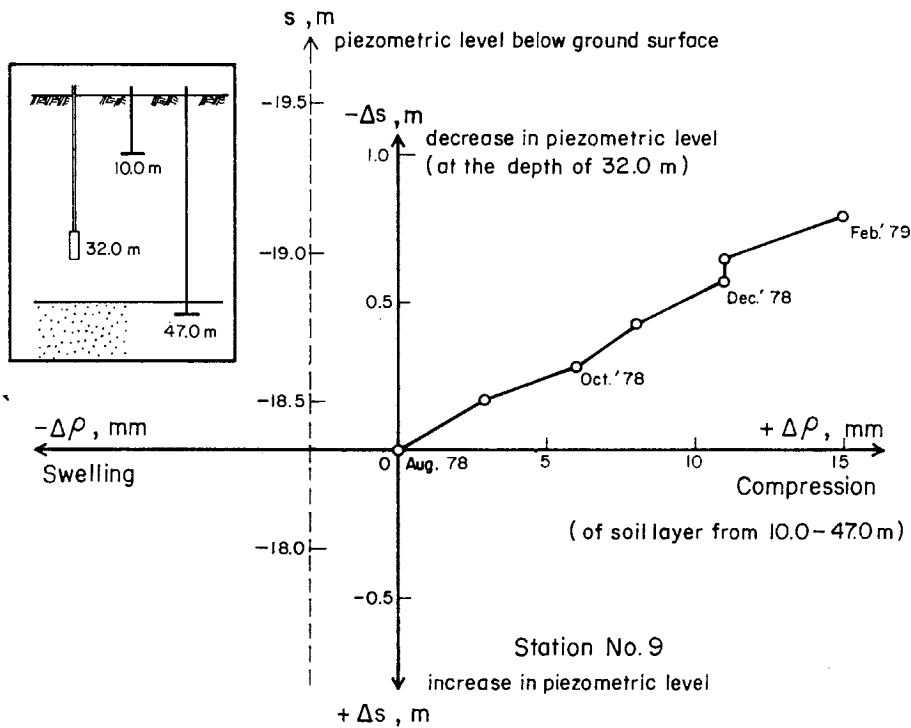
Fig. 13. Relationship between the change in piezometric level and soil layer compression in deep zone.
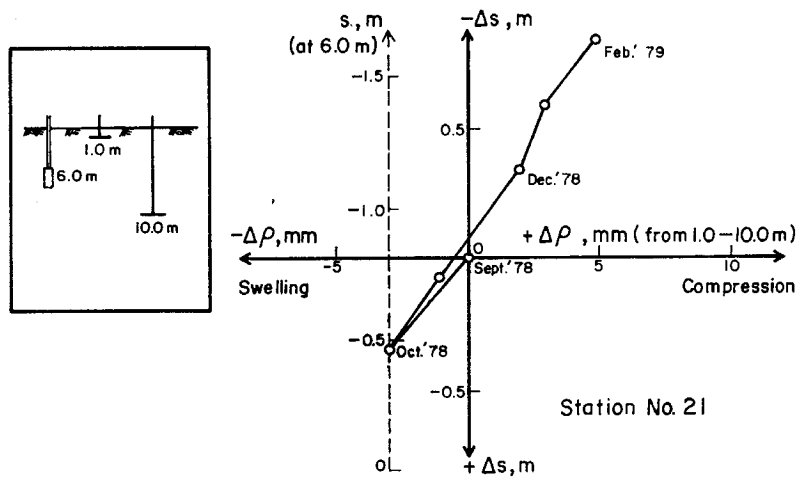
Fig. 14. Relationship between change in piezometric level and soil layer compression in shallow zone.

(3) The piezometric level in the Bangkok Aquifer was found to be about 20 m below groundsurface in the central Bangkok area and it is about 25 m in the area south of Bangkok.

(4) The rate of compression of the upper 50 m zone was observed from the field measurements to be about 3 cm per year corresponding to a rate of piezometric level decline in this layer of about 2 m per year.

(5) Good correspondence was observed from the relationship between soil layer compression and piezometric level decline in the layer which undoubtedly confirms the link between groundwater pumping and land subsidence in the Bangkok area.

REFERENCES

ASIAN INSTITUTE OF TECHNOLOGY (1977), Investigation of Land Subsidence Caused by Deep Well Pumping in the Bangkok Area, *Comprehensive Proposal of the Asian Institute of Technology*, submitted to National Environment Board, Bangkok.

BOWEN, R.W. and BOTBOL, J.M. (1975), The Geologic Retrieval Synopsis Program (GRASP), *U.S. Geological Survey Professional Paper 966*, U.S. Geological Survey.

COX, J.B. (1968), A Review of Engineering Properties of the Recent Marine Clays in South East Asia, *Research Report* No. 6, Asian Institute of Technology, Bangkok.

LOFGREN, B.E. (1969), Field Measurement of Aquifer System Compaction, San Joaquin Valley, California, U.S.A., *Proceedings of the Tokyo Symposium on Land Subsidence*, Tokyo, Vol. 1, pp. 272-284.

PREMCHITT, J. (1978), Analysis and Simulation of Land Subsidence with Special Reference to Bangkok, *D. Eng Thesis*, Asian Institute of Technology, Bangkok.

SODSEE, S. (1978), Geohydrology of the Chao Phraya Basin with Respect to Subsidence, *M.Sc. Thesis*, Asian Institute of Technology, Bangkok.

BRIDGE APPROACHES ON SOFT CLAY SUPPORTED BY EMBANKMENT PILES

Soren Holmberg*

SYNOPSIS

Differential settlements commonly develop between a bridge and its approaches at sites underlain by compressible subsoil deposits. Anticipating such settlements at bridge sites along a highway north of Bangkok, Thailand, the approaches were partly supported by embankments piles. In addition, approach slabs resting on a floating abutment in the embankment fill were included in the design. Six years after the highway completion, despite settlements of the embankment on the order of one-half meter, the pile supported approaches still provide a smooth transition between the bridges and the road.

INTRODUCTION

The use of embankment piles in connection with highway construction on soft ground is not a new technique. In 1938 the Code of Practice of the National Swedish Road Board recommended that embankment piles be used if the soft deposit was more than 6 m thick (FLODIN & BROMS, 1977), and KJELLMAN (1940) discussed the design procedure for embankment piles. However, very little has been published on the practical application of embankment piles.

This paper describes the design, construction and performance of pile supported bridge approaches on the Bang Pa-in to Nakhon Sawan Highway in Thailand (Fig. 1), which was opened to traffic in June 1972.

The southernmost twenty kilometers of this approximately two hundred kilometer long highway traverses an area where
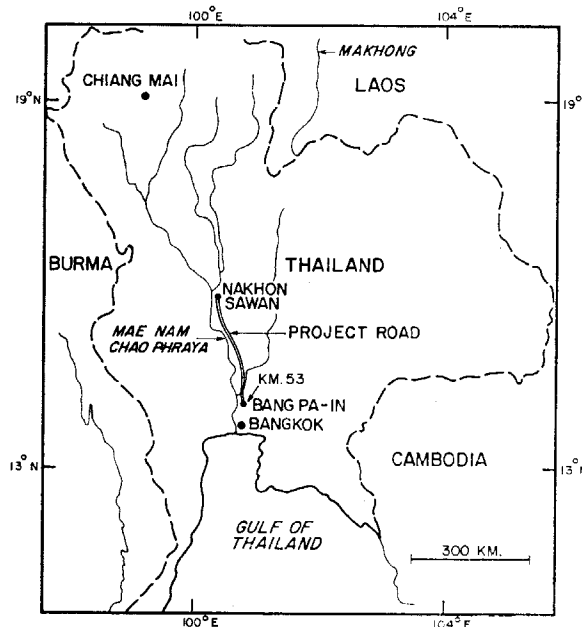

Fig. 1. Location map.

---

* Formerly Associate Professor of Geotechnical and Transportation Engineering, Asian Institute of Technology, Bangkok, Thailand. Present address : Rungsted Strandvej 7, 2950 Vedbak, Denmark.
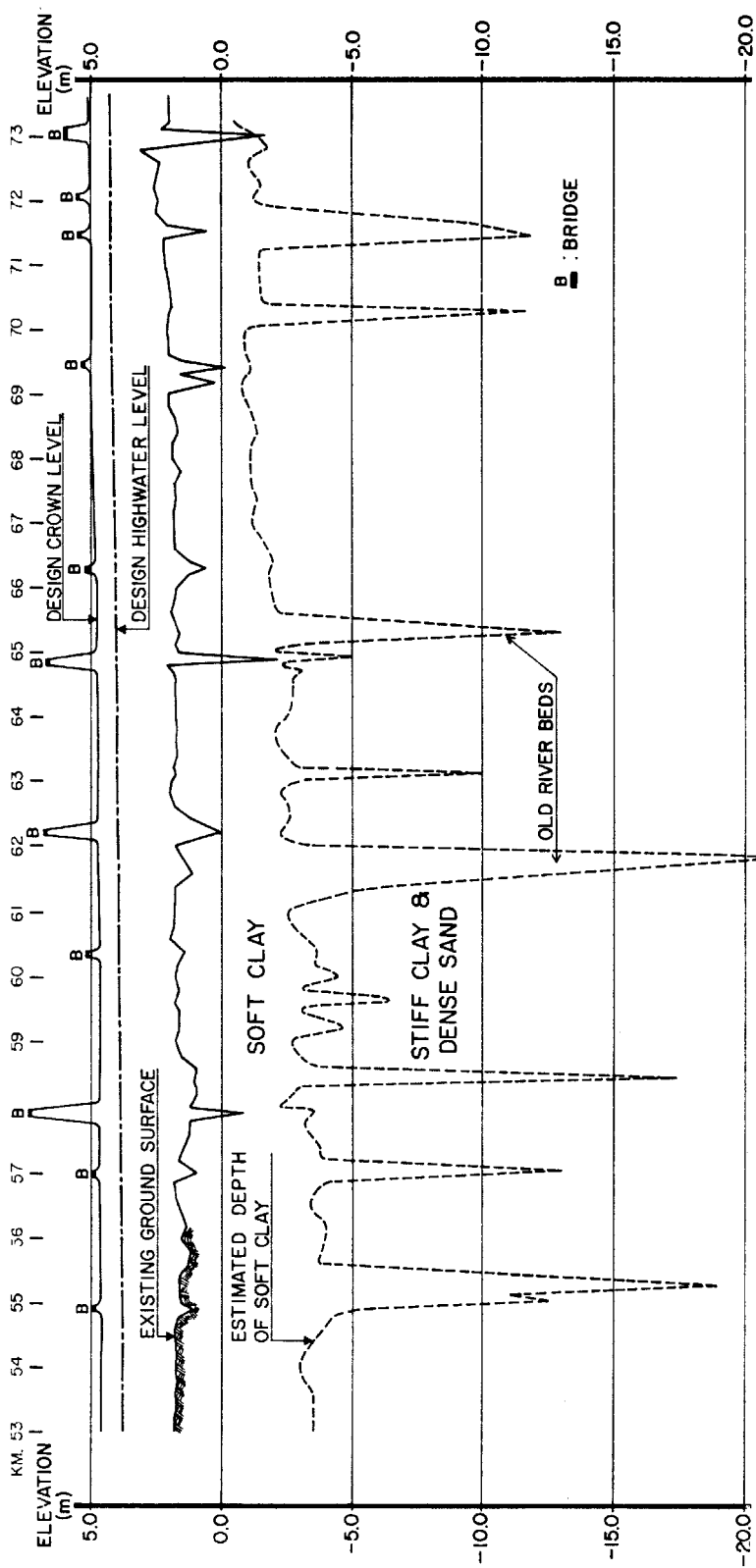
Fig. 2. Longitudinal profile; km 53 to km 73.

the subsoil generally consists of 2.5 to 5.0 m of soft clay. At eight locations the highway crosses what are believed to be old river beds, where the soft clay extends to a depth of between 11 m to 23 m as shown on the longitudinal profile in Figure 2.

EIDE (1968) has described some of the geotechnical problems that this soft clay would pose to the design and construction of the highway. The profile shown in Figure 2 is slightly different from the profile described by Eide because the actual highway was constructed some two to three kilometers east of the originally planned alignment.

The ground elevation of the area along the first twenty kilometers of the highway averages 2.0 m above mean sea level. The land, which is used for rich farming, is flooded once a year. This made it necessary to construct a relatively high (2.5 to 3.0 m) road embankment. In addition, the area is traversed by a number of small rivers (khlongs) that serve transportation, irrigation, or both purposes. As can be seen from Figure 2 it was therefore necessary to construct eleven bridges on this section of the highway.

At these bridge sites it was decided to use embankment piles to support the bridge approaches in order to prevent large differential settlements from developing between the bridge structures and the approaches and also to provide a smooth transition between the embankment proper and the bridges. In addition to the embankment piles the bridges were constructed with approach slabs that were supported at one end by a footing resting in the embankment fill (Fig. 6).

SUBSOIL CONDITIONS

The subsoil conditions in the area have previously been described in details by EIDE (1968), therefore only a brief description will be given here. Figure 3 shows a typical soil profile for the area outside the old river beds. Below a 0.5 to 1.0 m drying crust is found a 2.5 to 5.0 m deposit of soft, highly plastic clay with a shear strength about 2 t/m^2 and a natural moisture content of 80%. The plasticity index is as high as 80%.

Below the soft clay is found layers of stiff clay and stiff sandy clay with a vane shear strength of 25-30 t/m^2.

The results from consolidation tests indicated that the compressibility ratio of the soft clay, $C_c/1 + e_0$, varies in the range of 0.30 to 0.55, while
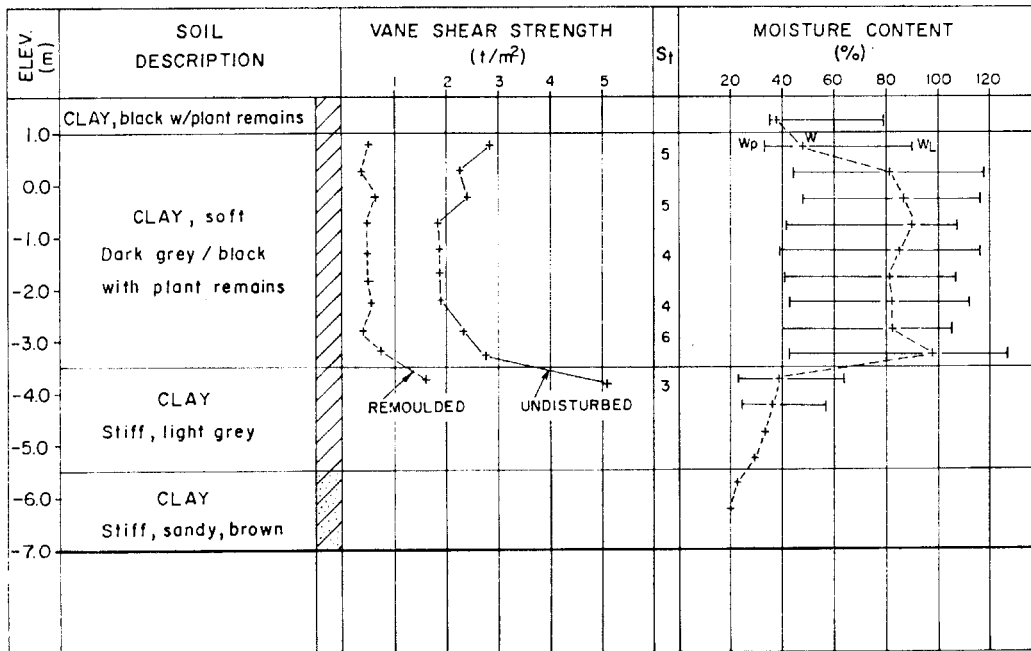
Fig. 3. Soil profile; km 54 + 000.

the coefficient of consolidation c_v, was found to be of the order of 1.5×10^{-8} m²/sec. The consolidation tests also yielded a preconsolidation pressure p_c, of 5 to 8 t/m², decreasing with depth. This p_c-effect is probably caused by weathering as indicated by the vane shear profile.

Figure 4 shows the soil profile of one of the old river beds. Below the drying crust the soft clay generally extends to between 11 m and 23 m. Just below the drying crust the vane shear strength is as low as 1.5 t/m² and it then increases approximately linearly with depth; in the profile shown to 3.5 t/m² at a depth of 11 m. The natural moisture content is 70 to 90 % and the plasticity index is about 50 to 60 %. The profile in Figure 4 shows that the deposits below the soft clay mainly consist of fine sand. However, in some of the other old river beds, the soft clay was found to be underlain by stiff clay.

The compressibility ratio, $C_c/1 + e_0$, as determined from consolidation tests on samples of the soft clay, varied from 0.30 to 0.75, while the coefficient of consolidation was found to be of the order of 1.5×10^{-8} m²/sec. The preconsolidation pressure, was 8 to 12 t/m², and is thought to be the effect of weathering and delayed consolidation.

80

ELEV. (m)	SOIL DESCRIPTION	VANE SHEAR STRENGTH (t/m²)	WATER CONTENT (%)
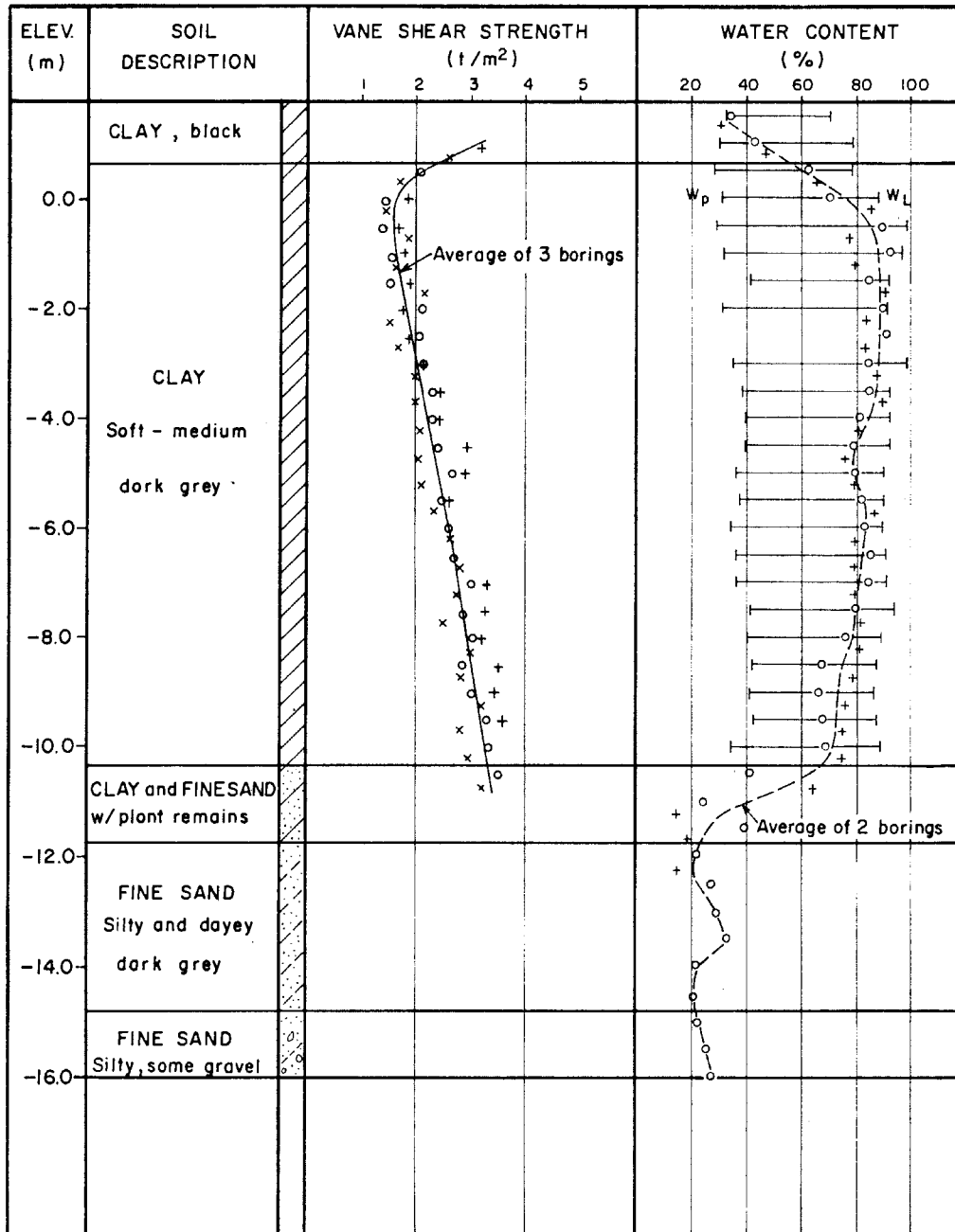
Fig. 4. Soil profile; km 63 + 300.

DESIGN OF EMBANKMENT PILES

The principle of determining how long a section of the approaches should be supported by embankment piles is shown in Figure 5. The figure also shows the design crown level of a bridge approach. The vertical curve has a radius of 5400 m, corresponding to a design speed of 80 to 100 kph. The settlements of the approach without embankment piles were then estimated for a twenty year period and the corresponding elevation of the approach was plotted as shown by the dotted curve. In the design it was decided that the minimum acceptable radius for the vertical curve should be 4500 m. Thus a curve with this radius was drawn and the intersection between this curve and the twenty year settlement curve determined the length of the pile supported section.
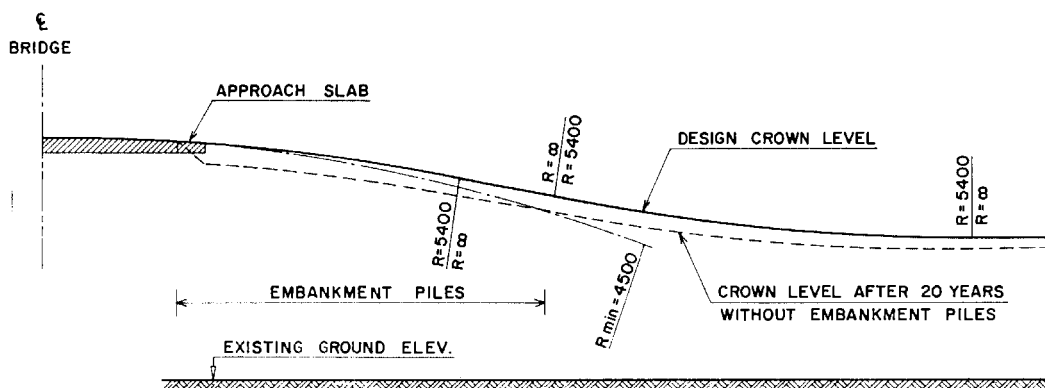
Fig. 5. Determination of length of pile supported embankment section.

A general layout of the pile arrangement is shown in Figure 6. The longitudinal profile also shows the seven meter long approach slab that is supported by a footing resting in the embankment fill. All the piles were driven with a batter of 1:4. For stability reasons the direction of the batter was changed ninety degrees for the piles closest to the bridge (Fig. 6a).

The length and the spacing of the piles were determined so that the piles close to the bridges would theoretically be able to support the full weight of the embankment. With increasing distance from the bridges the pile lengths were gradually reduced and the spacing increased so that the weight of the embankment was gradually transferred from the piles to the ground surface. Close to the bridges the pile spacing was 1.5×1.5 m^2. At the beginning of the piled section it was 2.0×3.5 m^2 in the cross-sectional and longitudinal directions, respectively.

82
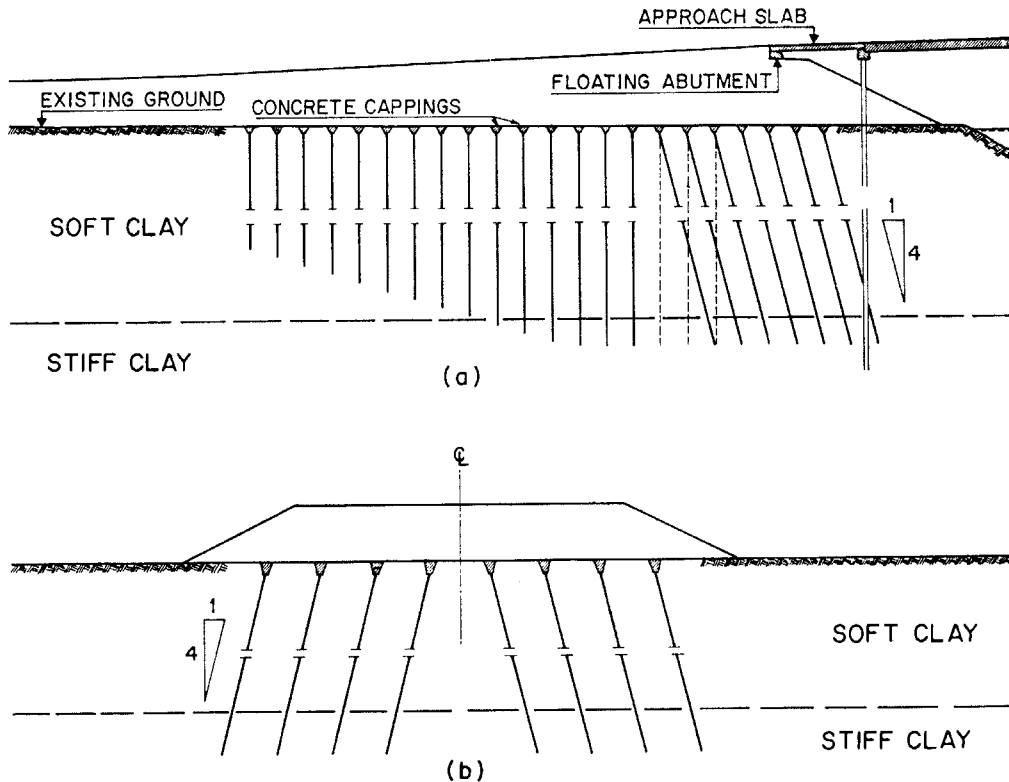
Fig. 6. Typical layout of embankment piles; (a) longitudinal profile, (b) cross-section.

Piles 16 m long or less were timber piles with a diameter of 20 cm, while piles longer than 16 m were 22 × 22 cm² prestressed concrete piles. The top of the timber piles was 60 to 80 cm below the ground surface corresponding to the ground water table during the dry season. On top of the piles were cast 80 cm diameter concrete cappings as shown in Figure 7. The figure also shows the 80 × 80 cm² precast concrete cappings used in connection with the concrete piles.

The embankment fill in the bridge approaches was sand with a built-in bulk density of 1.9 t/m³.

Table 1 is a summary of pertinent data from the eleven bridge sites where embankment piles were used. The total length of the piles was 71,132 m of which 56,760 m were timber piles and 14,372 m were concrete piles. The corresponding numbers of piles were 6033 and 815, respectively. The capping area in percentage of the embankment area at existing ground level is on the average about 10%. In the embankment sections close to the bridges the maximum capping area is 19% with an average of 15%, decreasing from
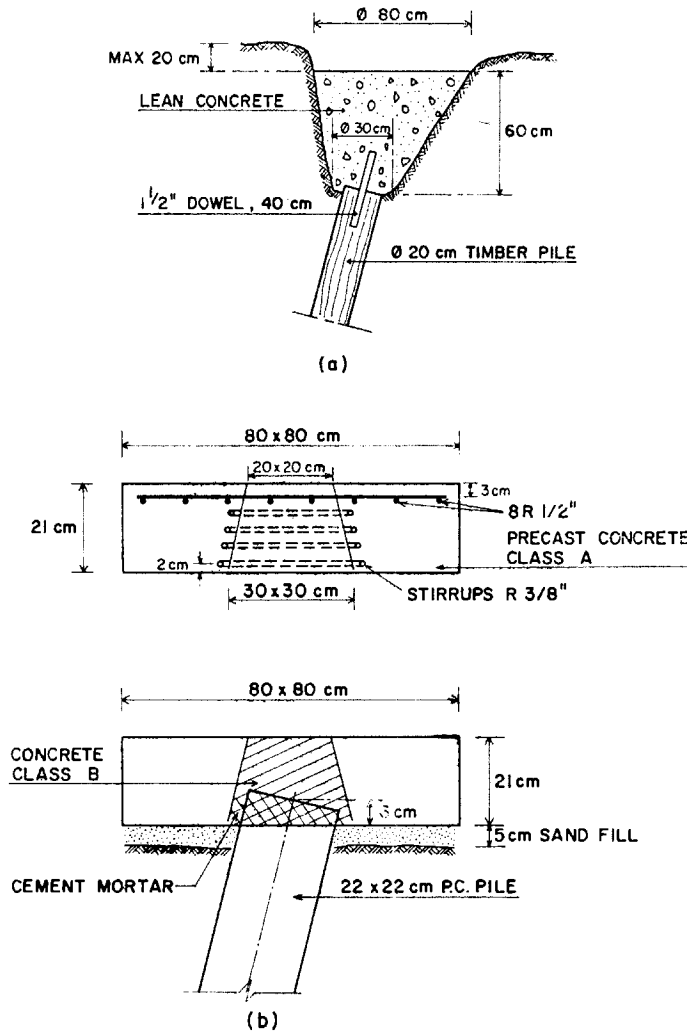
Fig. 7. Pile cappings; (a) timber piles, (b) concrete piles.

south to north. The minimum capping area, at the beginning of the piled approach sections, is 4 to 6%. From Table 1 it can also be seen that the minimum embankment height where piles were used is about three metres.

PERFORMANCE OF EMBANKMENT PILES

At the time of writing, December, 1978, the embankment piles described above seem to have performed satisfactorily. The highway has by now been in use for more than six years and, despite large settlements, the pile supported bridge approaches still provide a smooth transition between the settlement-free bridges structures and the embankment proper. It has not

Table 1. Pile Quantities.

Bridge (km)	Bridge Function	Length of Piled Section (m)	Maximum Emb. Height(m)	Minimum Emb. Height(m)	Pile Type	Pile Lengths (m)	Number of Piles	Total Pile Length (m)	Capping Area (m²)	Capping Area (%)	Capping Area max(%)	Capping Area min%
54+910 S	Flood	52.5	3.8	3.1	T	6-7-9-11	268	2520	135	10	14	6.0
N	Relief	86.5	3.6	3.1	T	12-13-14-15	464	6610	233	11	19	5.0
56+982 S	Khlong	80.0	4.8	4.2	C/T	6-8-9-12-15-17-18	260/168	4570/1692	238	10	16	5.0
N	Crossing	76.0	5.1	4.4	C/T	6-8-10-13-15-17	224/178	3808/1826	226	10	15	5.0
57+891 S	Khlong	89.5	6.1	4.4	T	6-7-8-9-10-11	633	6017	318	11	16	5.6
N	Crossing	89.5	6.1	4.4	T	6-7-8-9-10-11	633	6017	318	11	16	5.6
60+320 S	Flood	47.5	3.8	3.5	T	6-7-9-10-11	231	2121	116	9	16	3.7
N	Relief	47.5	3.8	3.5	T	6-7-9-10-11	231	2121	116	9	16	3.7
62+204 S	Khlong	75.0	5.2	4.6	T	6-7-8-9-10-11	404	3631	203	9	16	4.4
N	Crossing	89.0	6.0	5.2	T	6-7-8-9-10-11	546	4985	275	11	16	6.0
64+829 S	Khlong	67.0	4.9	3.8	T	8-9-10	365	3360	184	10	16	4.9
N	Crossing	96.0	6.3	4.3	T	7-8-10	639	5981	321	10	17	4.6
66+315 S	Flood	53.5	3.5	3.4	T	6-7-9	256	2007	129	10	13	5.4
N	Relief	53.5	3.9	3.5	T	6-7-9	256	2007	129	9	13	5.3
69+430 S	Flood	37.5	~3.5	~3.5	T	5-6-8-9	196	1485	99	10	15	4.1
N	Relief	37.5	3.8	3.5	T	5-6-8-9	196	1485	99	10	14	4.1
71+486 S	Crossing	48.5	3.5	3.0	C	17-18-19	148	2697	95	8	13	5.3
N	Irrigation Canal	52.5	3.3	3.2	C	16-17-18-19	183	3297	117	9	15	5.3
72+058 S	Flood	31.5	3.1	2.9	T	6-7-8.5	105	808.5	53	7	10	4.4
N	Relief	31.5	3.1	2.9	T	6-7-8.5	105	808.5	53	7	10	4.4
72+384 S	Khlong	19.0	3.2	2.9	T	6-8	75	558	38	8	9	5.0
N	Crossing	21.0	3.7	3.4	T	6-9-10	84	720	42	8	9	5.0

T = Timber piles,   C = Concrete piles.

been necessary to carry out any repair work on the surface of the approaches. In contrast, such maintenance is carried out on other highways in the vicinity once a year.
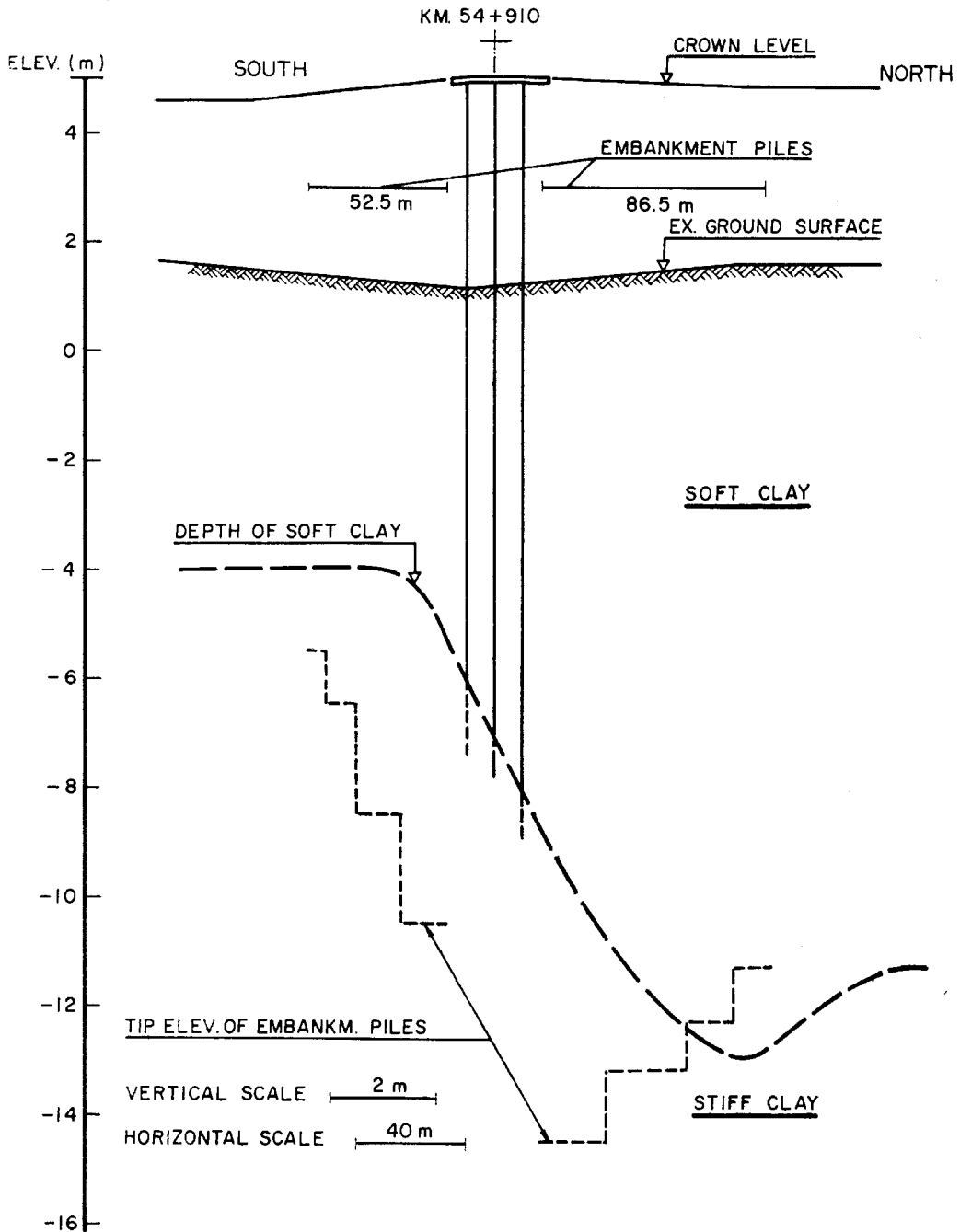
Fig. 8. Bridge at km 54 + 910; longitudinal profile.

Recently a survey of some of the bridge approaches was made in order to measure the actual magnitude of the settlements which have occurred since the highway was opened to traffic. In Figure 8 is shown a longitudinal profile of the bridge located at km 54+910. The bridge is built on the slope of one of the old river beds. The existing ground elevation is about +1.8 m and below the southern approach the soft clay extends to elevation −4.0 m, while under the northern approach the surface of the stiff clay is located between elevation −9 m and −13 m. The maximum height of the approaches is 3.6 to 3.8 m and the lengths of the pile supported sections are 52.5 m and 86.5 m, south and north, respectively.

Figure 9 shows the crown elevation, as surveyed in June 1972, after the completion of the construction, together with the elevation measured in May 1978. On the southern side of the bridge, the embankment proper has settled 18 cm. At the beginning of the pile supported section the settlements are 13 cm, which are then smoothly reduced to zero 25 m from the approach slab. North of the bridge the embankment proper has settled about 60 cm, while the settlements at the beginning of the pile supported approach are 54 cm. Despite these large settlements the piled section provides a smooth transition to the bridge.
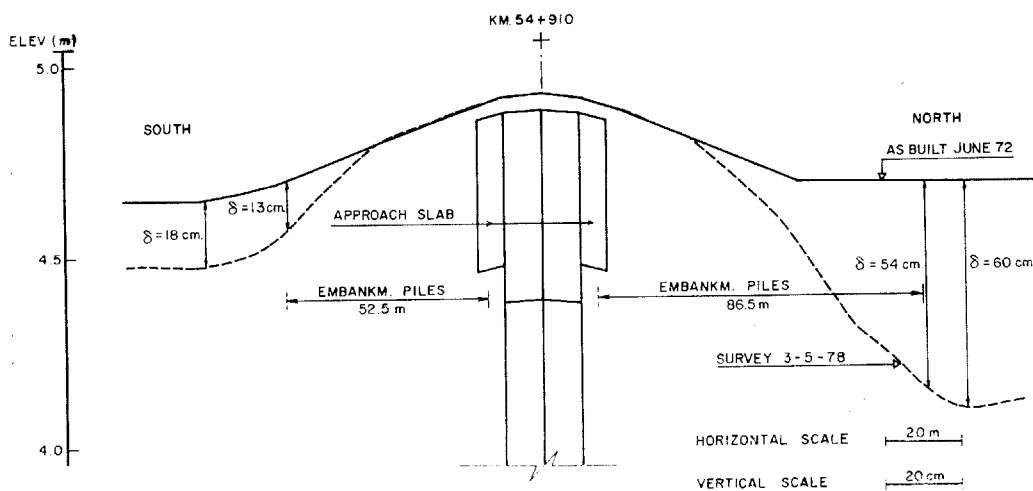
Fig. 9. Bridge at km 54 + 910; settlement of approaches.

Where the highway crosses navigable khlongs the bridge elevations had to be increased considerably and thus also the height of the approaches. One such bridge is the bridge located at km 57+891. The maximum height of the approaches is 6.1 m. The ground elevation is very low, only about +1.0 m, and the soft clay extends to elevation −3.0 m on both sides of the bridge.

The vane shear strength of the soft clay was as low as 1 t/m^2, the lowest value measured along the highway. The pile supported section for both approaches is 89.5 m and the pile patterns are also identical. A survey carried out in August 1978 showed that the two approaches had experienced more or less the same settlements.

Figure 10 is a longitudinal profile of the southern approach, which com-
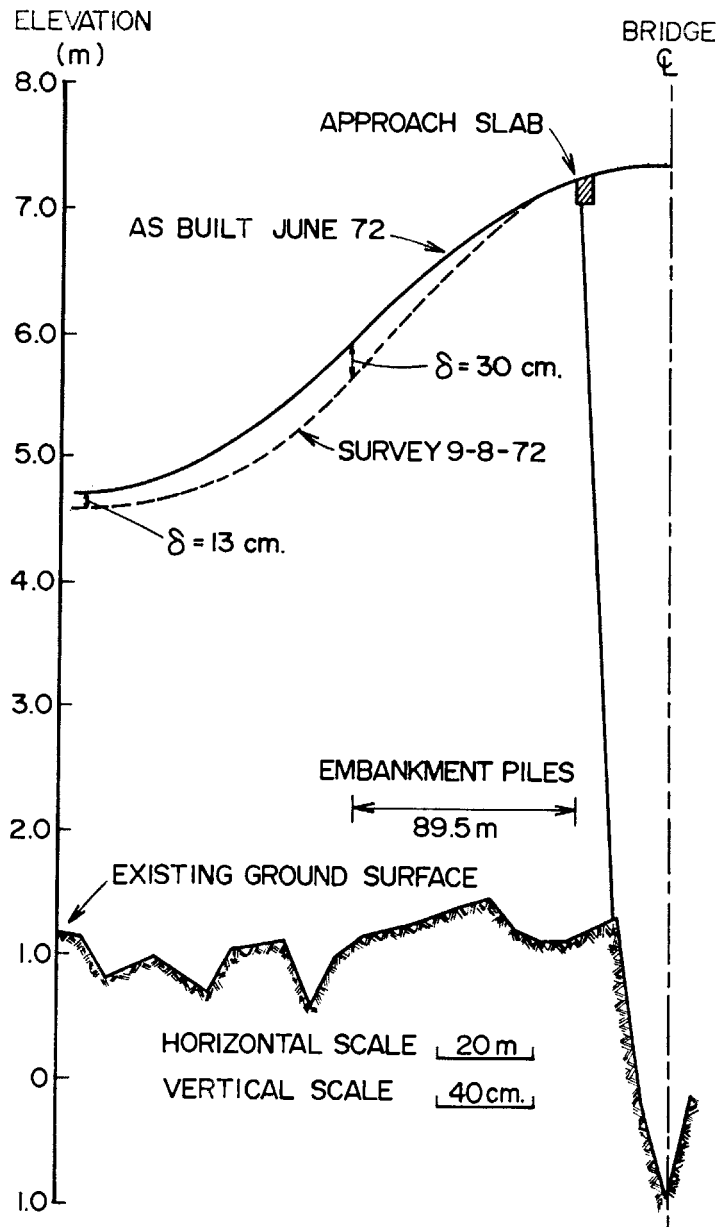
Fig. 10. Bridge at km 57 + 891; settlement of southern approach.

pares the embankment elevation at the end of construction with the elevation measured in the above mentioned survey. The road embankment proper, which originally was about 3.5 m high, has settled 13 cm, while at the location where the piled section starts the settlements are 30 cm corresponding to an initial embankment height of 4.8 m. Through the pile supported section where the pile length varies from 6 m to 11 m the settlements are gradually reduced to zero a few metres in front of the approach slab.

CONCLUSIONS

The experience gained with the use of embankment piles on the Bang Pa-in-Nakhon Sawan Highway confirms that embankment piles beneath bridge approaches on soft ground is a suitable means to eliminate the traditional problem of differential settlements. In the author's opinion the "floating" approach slabs are important in the connection with the use of embankment piles. Although in the examples described above the approach slabs did not seem to have settled, surveys of other approaches showed that slight settlements of the approach slabs had taken place. These settlements may be due to either settlements of the ground surface or due to settlements within the embankment fill caused by the traffic load. The latter settlements are considered to be the major cause in those cases where high approaches were constructed.

ACKNOWLEDGEMENTS

REFERENCES

EIDE, O. (1968), Geotechnical Problems with Soft Bangkok Clay on the Nakhon Sawan Highway Project, *Norwegian Geotechnical Institute*, Publication No. 78.

FLODIN, N.O. and B.B. BROMS (1977), Historical Development of Civil Engineering in Soft Clay, Preprint, *International Symposium on Soft Clay*, Bangkok, July, 1977.

KAMPMANN, KIERULFF & SAXILD, KAMPSAX A/S, (1969), Bang Pa-in-Nakhon Sawan Highway, Final Consolidated Report, Vol. I & II, *Report Submitted to the Highway Department of Thailand*, February, 1969.

KJELLMAN, W. (1940), Calculation of Embankment Piles (in Swedish), *Tekn. Ts. V.O.V.* Vol. 70.

STRESS AND DEFORMATION CHARACTERISTICS OF AN ELASTIC PUNCH IN AN ELASTO-VISCOPLASTIC MEDIUM

T. NISHITANI* and M. ITO[+]

INTRODUCTION

An understanding of the stress-strain behaviour of materials in contact between semi-infinite elasto-viscoplastic and elastic media are important in soil engineering. In estimating the bearing capacity and the time dependent settlement of structure such as buildings and bridges, the viscous properties of the subsoil plays an important role.

The deformation and the stress states near the contact surface are seriously affected by the mechanical properties of the media in contact and the frictional characteristics on the contact surface. Since it is difficult to evaluate these effects in the analysis of deformation, such problems have been analysed as those between rigid and rigid-plastic media (HILL, 1950; KACHANOV, 1971). The theoretical analysis of such problems becomes very complicated under the influence of viscous effects.

In a previous paper (NISHITANI, 1976) the stress distribution and its time dependent variation at the contact between the ground surface (assumed to be an elasto-viscoplastic medium) and a building (assumed to behave elastically) were analyzed using a photo-viscoplasticity technique. As a continuation of this study, the present paper examines the stress distribution and its time dependent variation near the contact surface for a semi-infinite plate of elastic-viscoplastic celluloid compressed under plane strain conditions on a part of the boundary of a flat punch of elastic araldite. The results obtained from the model test are discussed in relation to the time dependent settlement of a building due to the viscous behaviour of the subsoil.

APPARATUS AND TEST SPECIMEN CHARACTERISTICS

Figure 1 shows the apparatus used in the experimental investigation. It consists of a semi-infinite celluloid plate which acts as the ground surface and araldite flat punch in the centre which simulates a building. The

*Assistant Professor, [+]Research Student, Division of Applied Mechanics, Suzuka College of Technology, Suzuka, 510-02, Japan.

apparatus was placed in an optical bench. The semi-infinite plate was constrained on both sides with thick glass plates, so that the deformation was under plane strain conditions. Vertical movements of the glass plates are permissible since they are supported on their outer faces by ball bearings.
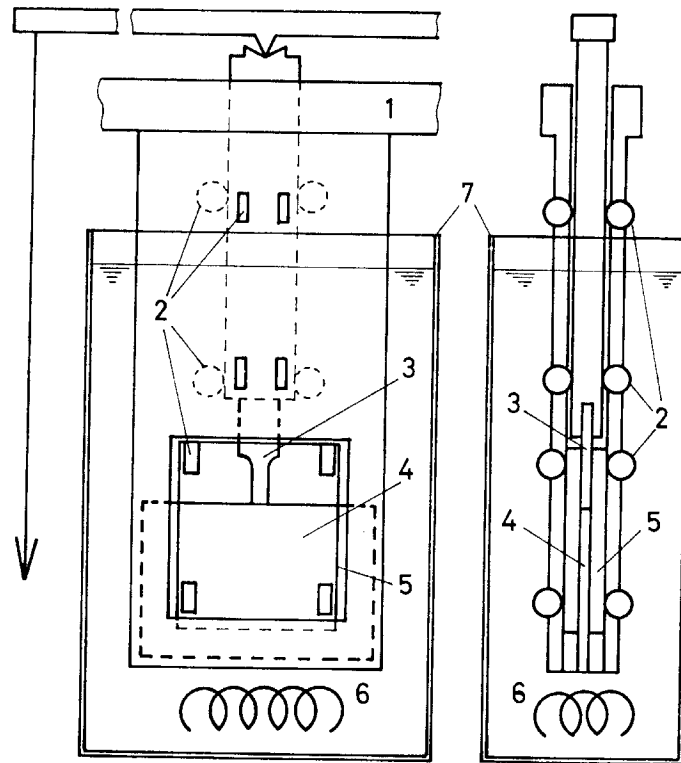
1	Supporter	5	Glass plate
2	Ball bearing	6	Heater
3	Elastic punch	7	Oil bath
4	Semi-infinite plate		

Fig. 1. Apparatus.

The semi-infinite celluloid plate was softened and deformed elasto-viscoplastically when the apparatus was immersed in an oil bath heated at 65°C under the external force; the araldite punch was kept in an elastic state.

Figure 2 shows the geometries of the araldite punch and the 6 mm thick celluloid plate which was wide enough to be considered semi-infinite. From the calibration test performed on the araldite at 65°C, the fringe stress

was 9.6 MPa.mm and Young's modulus was 2340 MPa. A 1 mm square grid (Fig. 2) was incised on the surface of the celluloid plate in order to trace the location of each element at any instant of time. Mechanical properties of the celluloid at 65°C were obtained from the calibration test in which a combined loading of axial tensile stress σ_1, and an oil pressure σ_2, was applied to the specimen. The procedures and the results of the calibration test were presented in the previous paper (NISHITANI, 1976).
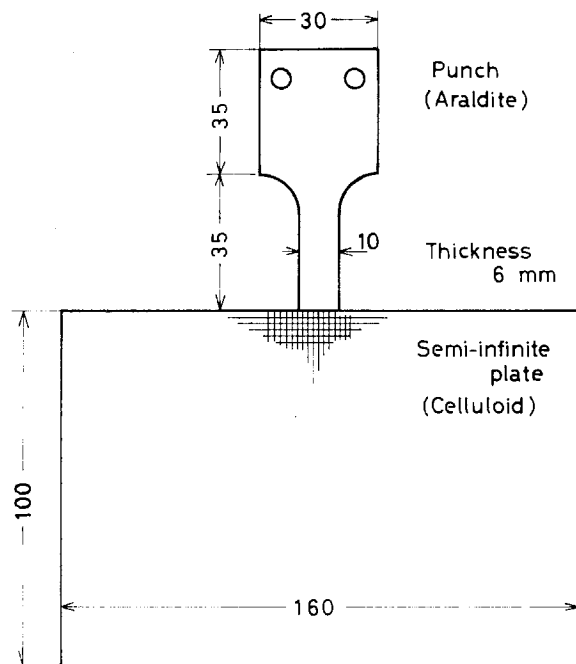
Fig. 2. Dimensions of test specimens.

RESULTS OF THE EXPERIMENTAL PROCEDURE

After the temperature had reached a steady value at 65°C, an instantaneous dead load of 60 kgf was applied. The mean contact pressure on the contact surface was equal to 9.8 MPa. The isochromatic fringe pattern at the instant of loading and the isochromatic and isoclinic fringe pattern of the deformed network at 2, 40, 160 and 300 minutes after loading were photographed.

Figures 3a and 4a show the family of isoclines after loading at times of 2 and 300 minutes respectively. Figures 3b and 4b show the slip-line fields obtained by using the isoclinic patterns at 2 and 300 minutes. Figure 3c

shows PRANDTL's analytical slip-line field for a rigid-perfectly plastic medium (HILL, 1950; KACHANOV, 1971). [The isochromatic fringe patterns at the instant of loading and 300 minutes after loading were also shown in Figures 1a and 1b of an earlier paper (NISHITANI, 1976).]
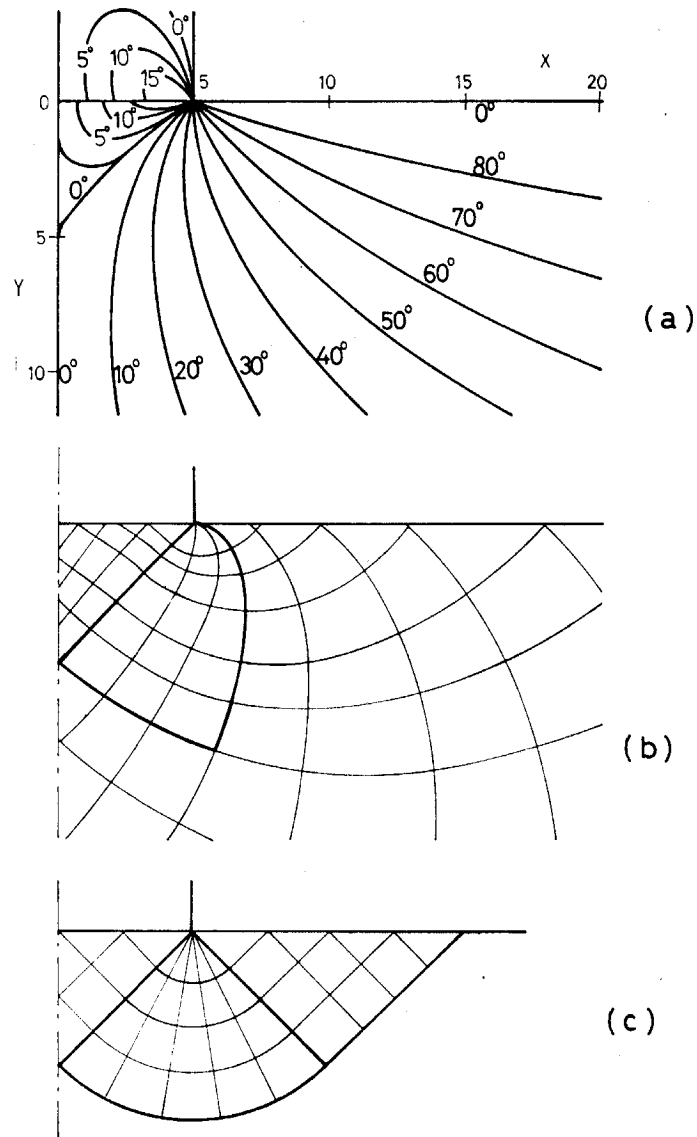
Fig. 3. Family of isoclines and slip-line fields at 2 minutes after loading.
(a) Experimental isoclines
(b) Slip-line field corresponding to experimental isoclines
(c) Prandtl's theoretical slip-line field for a rigid, perfectly plastic medium
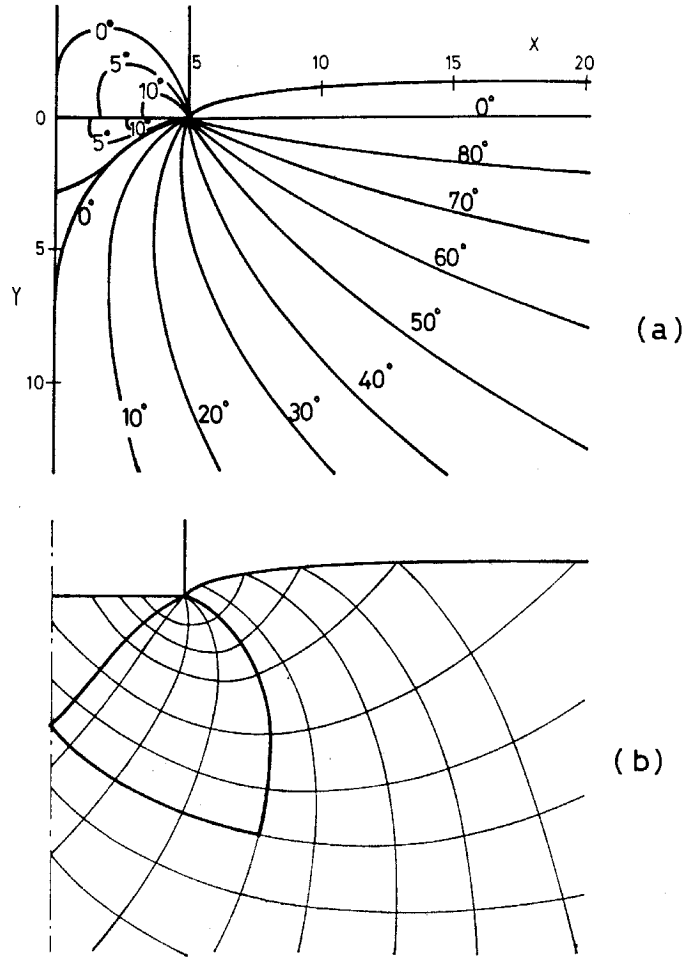
Fig. 4. Family of isoclines and slip-line field at 300 minutes after loading.
(a) Experimental isoclines
(b) Slip-line field corresponding to experimental isoclines

PRINCIPAL STRESS DIFFERENCE IN THE SEMI-INFINITE PLATE

The isochromatic pattern obtained from the photo-viscoplasticity technique was used for determining the distribution of the principal stress difference in the semi-infinite plate of celluloid. The fundamental relations for plane strain state have been given as follows (NISHITANI, 1976):

$$\Delta \dot{e} = B \, (t + s)^{\alpha} \quad \exp \left[b \left(\frac{\Delta \sigma}{2} + a \, \bar{I}_1 \right) \right] \, (\Delta \sigma + 2a \, \bar{I}_1) +$$

$$\frac{2n + 1}{2G} \left(\frac{\Delta \sigma}{2} + \frac{a_\tau}{k} I \right)^{2n} \Delta \dot{\sigma} \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \quad (1)$$

$$\dot{N} = C_1 \Delta \dot{\sigma} + C_2 \Delta \dot{e} \qquad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \quad (2)$$

In these equations the dot represents the differentiation with respect to time t, and s denotes the material constant time; Δe and $\Delta \sigma$ stand for the principal viscoplastic-strain difference and the principal stress difference respectively. $\bar{I}_1 = I_1/(3J_2)^{\frac{1}{2}}$ is the dimensionless first invariant of the stress tensor. $I_1 = \sigma_{ii}$ and J_2 denote the first invariant of the stress tensor and the second invariant of the deviatoric stress tensor respectively. N is the fringe order per unit thickness and B, α, a, b, n, G, k, C_1, C_2 are material constants. These material constants were determined from the calibration test (NISHITANI, 1976). Calculation procedures of the principal stress difference

Fig. 5. Distribution of $\bar{I}_1$ at 2 minutes after loading.

by using these relations were described in the previous paper (NISHITANI, 1976).

The deformation and stress states were fairly dependent on the value of $\bar{I}_1$ as shown in Figure 3 of the earlier paper (NISHITANI, 1976). Therefore, in order to analyse the stress state precisely, the value of the dimensionless first invariant of stress tensor $\bar{I}_1$ should be known for each element of
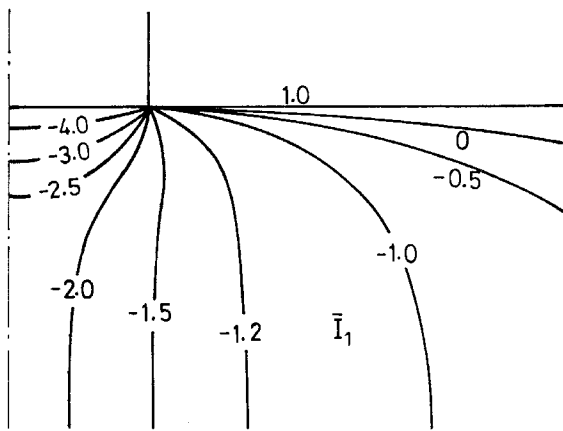
the semi-infinite plate. However, since these values of I_1, cannot be known in advance, the values of the principal stress difference $\Delta \sigma$ for each element at each instant were calculated by using $\bar{I}_1 = -2$ as the first approximation. The values of the stress components were then determined by the shear difference method using the distributions of $\Delta \sigma$ as obtained from the first approximation and the isoclinic patterns. From the stress components thus obtained, the distribution of $\bar{I}_1$ was

(a)
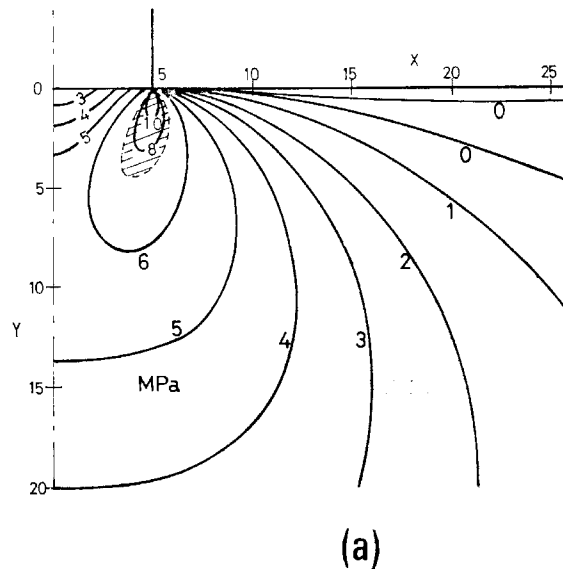
Fig. 6. Distribution of the principal stress difference $\Delta \sigma$ (MPa).
(a) 2 minutes after loading

96
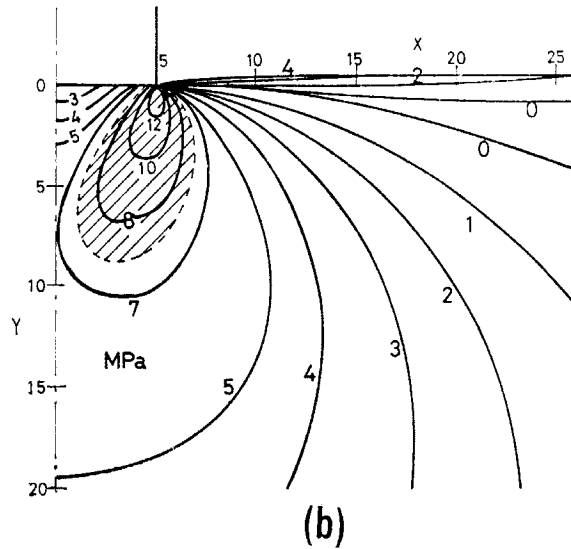
(b) 40 minutes after loading
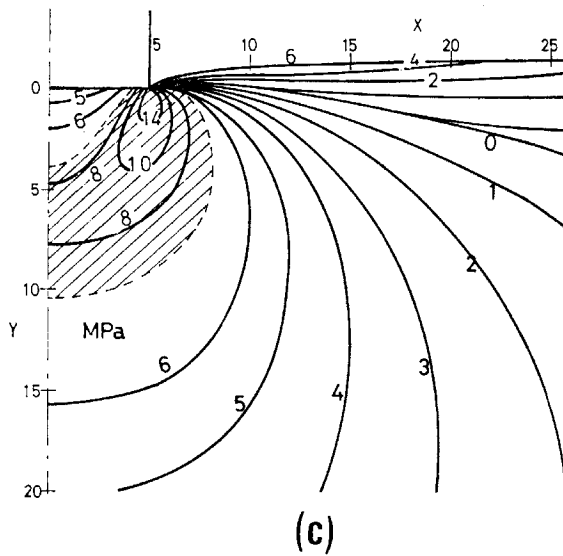
(c) 300 minutes after loading

found at each instant. As an example, Figure 5 shows the approximate distribution of $\bar{I}_1$ at 2 minutes after loading. The value of $\Delta\sigma$ in each element at each instant was calculated again by using the corresponding values of $\bar{I}_1$ at 2 minutes as shown in Figure 5. The distributions of $\Delta\sigma$ at 2, 40, and 300 minutes are shown in Figures 6a, b and c.

97

STRESS DISTRIBUTION IN THE SEMI-INFINITE PLATE

According to the experimental results obtained by FROCHT & CHENG (1962), isoclinic parameters in the plastically deformed celluloid represent the directions of the principal stress regardless of the state of stress and its history. Thus, the stress components in the celluloid plate were obtained by the shear difference method using the isoclinic pattern shown in Figures 3a and 4a and the distributions of $\Delta\sigma$ as shown in Figure 6.

In Figures 3a, 4a and 6, a cartesian coordinate (x, y) system was employed by dividing the half width of the punch into equal 5 parts. Figures 7 and 8 show the distributions of stress components at 2 and 300 minutes after loading on the sections, $y = 3, 5$ and 7 of the semi-infinite plate. The thick curve in Figure 9 shows the stress distribution on the contact surface at the four instants, in which the thin curve corresponds to the results on the punch side as obtained by the photoelasticity technique. In Figure 9, the values of σ_y in the semi-infinite plate and in the elastic punch agree well with each other at each instant. Thus the condition of continuity of the normal stress on the contact surface is well satisfied.
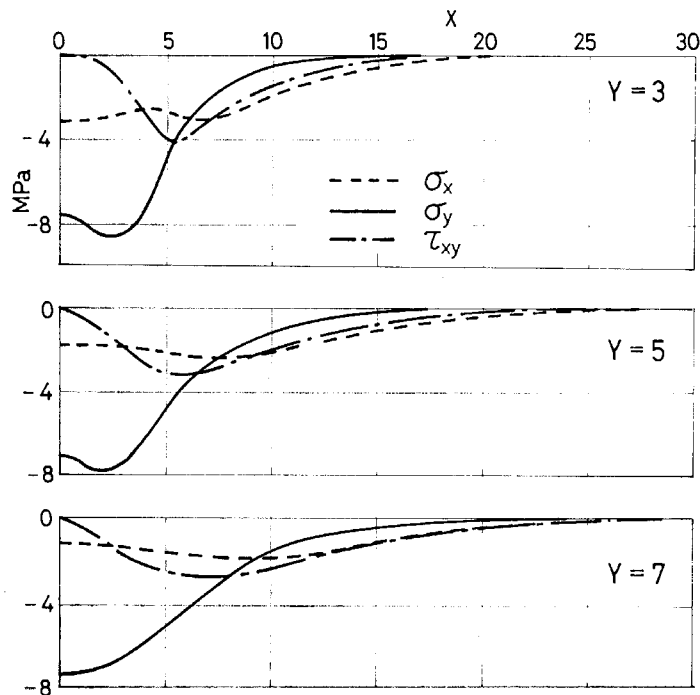
Fig. 7. Distribution of stress components in the semi-infinite plate at 2 minutes after loading.
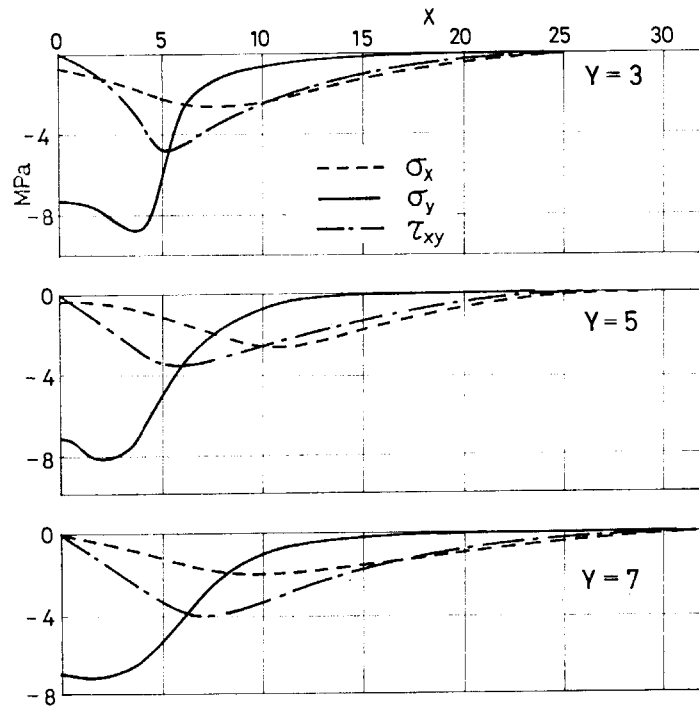
Fig. 8. Distribution of stress components in the semi-infinite plate at 300 minutes after loading.

Thick curve : Semi infinite plate
Thin   "   : Elastic punch
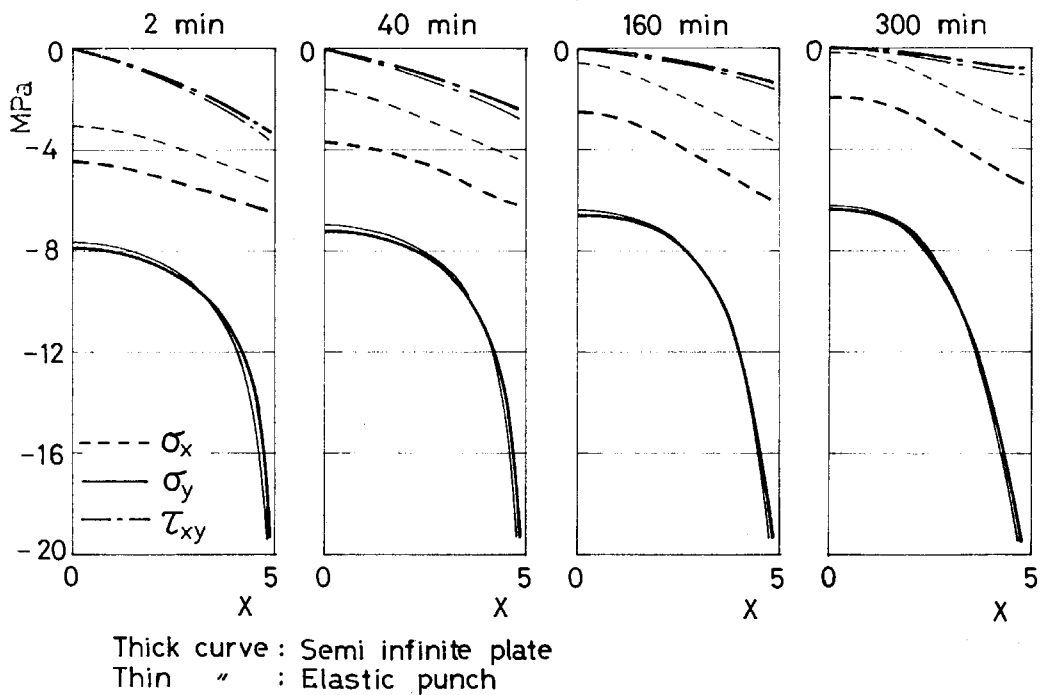
Fig. 9. Distribution of stress components on the contact surface at four instants. Short dashed curve σ_x, solid curve σ_y, long dashed curve τ_{xy}.

99

DISCUSSION AND CONCLUDING REMARKS

Stress components

Figure 9 shows that at 2 minutes after loading the contact pressure σ_y on the contact surface has very large values at both ends and minimum values at the center of the surface. The value of σ_x on the contact surface of the punch is compressive. The corresponding values on the side of the semi-infinite plate is larger due to the constraint of the semi-infinite plate from the parts lying on both sides of the contact surface. With the elapse of time, the value of σ_y in the semi-infinite plate tends to concentrate at both ends on the contact surface as shown in Figure 9. On the contrary, the compressive value of σ_x on the contact surface decreases while the shear stress disappears gradually from the central part to both ends. In Figures 7 and 8, the shear stress in the semi-infinite plate increases (in a certain range) under both ends of the contact surface and also propagates in the lateral direction. The stress distribution after 300 minutes is almost invariably the same as that after 2 minutes of loading.

Distribution of $\Delta\sigma$

The values of principal stress difference corresponding to the equivalent stress, to which the equivalent residual strain is 0.2 % ($\Delta e = 0.3$ %), can be obtained from the calibration test (NISHITANI, 1976) as $\Delta\sigma = 7.8$ MPa for $I_1 = -2$ and $\Delta\sigma = 7.4$ MPa for $I_1 = -1.5$. Assuming that these values correspond to the yield stress of the celluloid in the semi-infinite plate, the plastic regions in Figure 6 correspond to the areas surrounded by the dashed curves with shaded line. This figure shows that the plastic regions first appeared adjacent to both ends of the contact surface. With a lapse of time these regions expanded and joined together. The plastic regions remained invariably the same even after 300 minutes. If we approximate the celluloid medium as a rigid perfectly plastic one, the plastic regions near the both ends of the contact surface are constrained individually first by the rigid part as shown in Figure 6a, which cannot show any unstable large plastic deformations. However, they expand with lapse of time and join together as shown in Figure 6c. At this stage, the rigid part of the celluloid under the contact surface joins with the elastic punch and they are supported on the plastic region in the hollow of the other rigid parts of the celluloid. Such an unstable state may be seriously affected by a slight external disturbance analogus to an earthquake, and plastic deformation may result.

100

REFERENCES

FROCHT, M.M. and CHENG, Y.F. (1962), An Experimental Study of the Laws of Double Refraction in the Plastic State in Cellulose Nitrate, *Proceedings of Symposium on Photoelasticity*, Pergamon Press.

HILL, R. (1950), *The Mathematical Theory of Plasticity*, Oxford University Press.

KACHANOV, L.M. (1971), *Foundation of the Theory of Plasticity*, North-Holland.

NISHITANI, T. (1976), Determination of Stresses and Deformations Using Photo-Viscoplasticity, *Geotechnical Engineering*, Vol. 7, pp. 109-116.

ERRATUM

Erratum: A new method to predict swelling using a hyperbolic equation

V. DAKSHANAMURTHY

Department of Civil Engineering, Indian Institute of Technology, Madras, India. Present address: Department of Civil Engineering, University of Saskatchewan, Saskatoon, S7N OWO, Canada.

Page no.	as printed	to be corrected as
30	SIRDHARAN & NARASIMHA RAO	SRIDHARAN & NARASIMHA RAO
30	$\varepsilon = \dfrac{t}{\alpha\beta t} \ldots\ (2)$	$\dfrac{t}{\alpha + \beta t} \ldots\ (2)$
37	$\varepsilon_{max}\ (\text{pred}) = \dfrac{\log p_s/\sigma}{\tan}$	$\varepsilon_{max}\ (\text{pred}) = \dfrac{\log p_s/\sigma}{\tan\psi}$
37	tvn ψ	tan ψ
37	expensive clay	expansive clay

BOOK REVIEWS

Slope Analysis, by R.N. Chowdhury, Elsevier Scientific Publishing Company, P.O. Box 330, Amsterdam, The Netherlands, 1978, 423 pp., U.S.$ 58.75 (Dfl. 135.00).

The text book is Volume 22 of the series 'Elsevier Developments in Geotechnical Engineering.' There are nine chapters, three appendices and a comprehensive list of references numbering more than four hundred and fifty. In a volume of 423 pages the author has tried hard to make an up-to-date presentation of the material published in slope analysis. In his preface the author rightly states that a subject such as slope analysis includes diversified fields such as soil mechanics, rock mechanics, engineering geology, geomorphology etc., where in each field there are now several up-to-date text books but the book under review is intended to make an integrated approach to the stability of soil and rock slopes. Whether there is a need for such an integrated approach is perhaps, controversial. Further, the author stresses the need for refinements in available methods of analysis and for the development of new concepts and approaches.

Except for Chapters 1, 6 and 7, most of the others are found to be close to fifty pages and the number of mathematical equations starting from Chapter 2 through Chapter 9 are 50, 67, 38, 36, 41 and 32 totally three hundred and twenty nine. There are ninety two sketches and graphs, thirty four tables and no photographs.

Chapter 1, titled Slopes, Geology and Materials, deals predominantly with general aspects of slopes and very briefly touches upon the nature of soils and rocks In Chapter 2, Basic Concepts, elementary concepts on shear strength of soils and rocks, pore pressures and failure criteria are discussed in detail together with a comprehensive presentation of the effects of progressive failure. Chapters 3 and 4 are devoted to limit equilibrium methods of analysis with planar failure surfaces and slip surfaces of arbitrary shapes respectively. These two chapters are found to adequately cover the analytical aspects of slopes in cohesionless and cohesive soils and also rock slopes though sketches illustrating the analysis of rock slopes were lacking. The analytical tools to be used in earth and hydraulic fill dams, slurry trenches etc. are also included.

Chapter 5 is devoted to stress analysis and slope stability wherein the advantages of carrying out stress analysis are emphasized and the finite element methods of analysis is introduced. The linear and non-linear

behaviour of the slope meterials is discussed and aspects such as strain softening, failure criteria and safety factors are presented in relation to stress analysis methods. In Chapter 6, natural slopes analysis is described where considerable emphasis is made on initial stresses and the coefficient of earth pressure at rest.

The plasticity and shear band analysis are briefly dealt with in Chapter 7, while special aspects such as earthquakes, creep and anisotropy are described in Chapter 8. The material presented in these sections is of considerable interest to undergraduate and graduate students, since normal text books in soil mechanics do not deal with such topics. Of course, for those who need a comprehensive knowledge on these topics, several text books are now available on each of the topics which are discussed in Chapters 7 and 8.

The reviewer of this book is pleased to see that probabilistic approaches are included in Chapter 9. These will certainly form a useful section of text book as future analysis of natural slopes and other stability problems should incorporate such studies related to risk analysis.

The author in his preface states that the text book is meant for senior undergraduate and graduate students, and the researcher as well as practioner. This reviewer will certainly recommend the text book to be of considerable interest and use to students and researchers, considering the scholarly approach made in compiling the contents and in making a clear presentation. At the same time he is surprised to find that at least for the design of low embankments on soft clays, a topic that he has been interested for nearly two decades, that there are many more factors to be considered in the design and construction than merely to pick an analytical tool. In that sense he would have been interested to see the inclusion of several case histories which clearly demonstrate the merit and demerit of each analytical method to practising engineers who are often confused with conflicting recommendations.

Finally, the reviewer wishes to congratulate the author for fulfilling the aims of writing this book namely (i) to outline the fundamental principles of slope analysis and to explore both similarities and differences in soil and rock slopes, (ii) to discuss assumptions underlying simple and rigorous methods, (iii) to highlight factors which influence slope performance, and (iv) to discuss alternate method of analysis and to present information on new concepts and approaches to analysis.

A.S. Balasubramaniam

Remote Sensing, Principles and Interpretation, by Floyd F. Sabins, Jr., W.H. Freeman and Co., 1978, 426 pp., 58 King Road, Reading AGT 3AA, England (US$ 27.50); 660 Market St., San Francisco, California, 69104, U.S.A. (US$ 25.00).

This book will become one of the introductory texts on remote sensing. With the exception of the American Society of Photogrammetry and Remote Sensing's two volume Manual of Remote Sensing (1975), no other book on the subject approaches its clarity of explanation nor does any other text have as many detailed examples of remote sensing applications. The text is superbly illustrated with several color and numerous black and white plates and finely drawn figures on quality paper. The volume includes twelve chapters, a 10 page glossary, and indexes.

The first chapter summarizes the characteristics of electromagnetic radiation and principles of images and vision. The next five chapters describe the remote sensing systems of aerial photography, manned satellite imagery, Landsat, thermal infrared, and radar imagery. Each of these chapters describes the physical properties and electromagnetic interactions of materials that control the imaging process, the design and operation of the imaging system, characteristics of the images including geometric distortions, gives guide lines and cites examples for image interpretation. A chapter on digital image processing describes computer techniques for image restoration and enhancement. Practical applications of remote sensing to resource exploitation, environmental and land-use monitoring and natural hazards are the subjects of the following three chapters. Chapters, on comparison of image types, remote sensing limitations and precautions, and future developments complete the main body of the text.

The chapters on application constitute an introduction to the subject for those who have yet to be convinced that remote sensing can benefit soil engineers or engineering geologists, or that this text will find use on a geotechnical reference shelf. Landsat imagery has outstanding applications in mapping regional lineaments, local fractures, and geological mapping. It provides an invaluable tool in updating existing geological maps on scales from 1:250,000 to 1:1,000,000. Great use for Landsat imagery will be found in Asia, South America, and Africa where conventional aerial photography is difficult to come by, and where there can be problems of obtaining basic geological and topographical data. Such data is of obvious relevance to dam site investigations, construction material surveys and corridor surveys. Land-use mapping is another capability of Landsat that is finding more application as mapping and classification of existing land-use patterns becomes a requisite for planning of future developments.

Various forms of remote sensing such as color infrared photography and thermal infrared surveys, have been used in analyzing risks, providing advance warning, and assessing damage of earthquakes, landslides, volcanic eruptions and floods. Seismic risk analysis utilising landsat and thermal infrared imagery is well documented. Land subsidence and areas of landslides have been recognized with the aid of colour infrared photographs. Landsat images have been used to determined the extent of flood damage.

Applications of side-looking radar that have potential use to engineering geologists are in delineating areas of similar surface materials. This application is based on the correlation between surface roughness and radar backscatter of materials with varying surface relief. One cited example delineated flood plain deposits, desert pavement, carbonate-cemented sand, smooth rock salt, coarse gravel, and rough eroded salt on the basis of their vertical relief and image tone.

In the 1980's developments in digital image processing for advanced enhancement and information extraction, improved spatial resolution, and stereocoverage will make these fields of study even more applicable to those whose professions are so closely concerned with earth surface processes.

The author has written the book for an introductory course for students with no previous training in remote sensing. It is perfectly suited to and recommended for that purpose. This reviewer would also highly recommend this text to any engineer or earth scientist with an interest in what remote sensing is or in its applications.

K.V. Campbell

Rockslides and Avalanches, 1, edited by Barry Voight, Elsevier Scientific Publishing Company, P.O. Box 330, Amsterdam, The Netherlands, 1978, 833 pp., US$ 98.00 (Dfl. 240.00).

Rockslides and Avalanches is Volume 14A in Elsevier's series 'Developments in Geotechnical Engineering'. Volume 1 of the two volume work deals with the aspects of natural phenomena and is a comprehensive accumulation of data on the large slides and avalanches of North and South America. Volume 2 will deal with mass movements as related to engineering projects.

The Preface states that Rockslides and Avalanches attempts to provide a foundation for studies of mass movement phenomena in the Western

Hemisphere. There is no question that Barry Voight, of Pennsylvania State University, has made a significant effort toward this goal and produced an admirable treatise with the publication of Volume 1.

Following the Introduction and Kenneth Hsu's dedication to Albert Heim, the volume is organized into four sections of individual contributions. These are Classic Rockslides and Avalanches; Large Scale Prehistoric Mass Movements; Rock Creep, Scale Effects and Related Questions; and Mass Movement of Snow and Ice. The authors have spared no effort in presenting concise texts with numerous maps, sections, ground and aerial photographs, and stereograms. Contrary to the situation in other multi-authored books, there are several places within the volume where authors reference other chapters. This is indicative of careful reviewing by the editor and some of the authors. The result is a cohesive treatment of the subject.

Introductions and dedications are commonly anemic efforts, usually boring, and often pretentious or irrelevant. This is not the case in Rockslides and Avalanches. The Introduction to Volume 1, by the editor and W.G. Pariseau, gives a synoptic view on the factors influencing slides: geologic features, rock weathering, seismic activity, and variation in hydrologic and climatic conditions. A preview is made of the chapters on prehistoric landslides, submarine slides, rock creep, and plate tectonics (considered by one of the chapter authors, W.R. Jacoby, to represent a gravity related phenomena), and mass movements of snow and ice. The 24 contributions are painstakingly summed up and research is placed into an historical context. The consequence of this effort is that the reader is given adequate initiation into the current knowledge and spectrum of mass movement phenomena.

In the Introduction, the editor has chosen to outline the development of the air-layer lubrication hypothesis of slide movement. The concept, proposed by S. Shreve and which gained popularity in the mid-1960's, is critically appraised. Possibly the editor is premature in placing this section in the Introduction, as the topic falls like a boulder with near terminal velocity amidst the summations of the volume's contributions. The air-cushion idea is a drop-kicked again in the dedication and batted back and forth in the ensuing fray throughout the first two sections of the volume. The reader is left with portents of dissipation of the air-lubrication supposition by Voight and Pariseau's announcement that there is no locality yet described where the air-layer mechanism can be demonstrated to have been the dominant means of emplacement. This consideration is directly opposed by conclusions in the same volume by R.K. Fahnestock and B. Johnson regarding the little Tahoma Peak, Washington and Blackhawk, California

landslides, respectively, that strong arguements can be made for the emplacement mechanism having been a cushion of compressed air. Correct as Voight and Pariseau most probably are in their skepticism of the air-cushion concept, their judgement could better have been left to a summary chapter on mass movement mechanisms following the chapters describing the evidence gathered from individual slides.

Kenneth Hsu's chapter dedicated to the Swiss geologist, Albert Heim (1849-1937), is a highly readable account of Heim's significant observations and conclusions regarding the Elm slide in Switzerland, in 1881. Heim's publications of 1882 and 1932 were in German and have been largely ignored by English speaking geologists. This is unfortunate, as Heim's explanation of the movement mechanism of exceedingly rapid debris flows generated by a rockfall or rockslide is as valid today as when it was proposed.

Heim deduced that rockfalls did not slide but their debris flowed in spite of being essentially dry. Heim also realized large debris flows travel farther than expected because of reduction in their internal friction and that this reduction is velocity dependant. In 1932 he proposed the following to explain the movement of large rockfalls. The passage is from Hsu's translation.

> When a large mass broken into thousands of pieces, falls at the same time along the same course, the debris has to flow as a single stream. The uppermost block, at the very rear of the stream, would attempt to get ahead. It hurries but strikes the block, which is in the way, slightly ahead. The kinetic energy, of which the first block has more than the second, is thus transmitted through impact. In this way the uppermost block cannot overtake the lower block and has to stay behind. This process is repeated a thousandfold, resulting eventually in the preservation of the original order in the debris stream. This does not mean that the energy of falling blocks from originally higher position is lost; rather the energy is transmitted through impact. The whole body of the sturzstrom (debris flow, current usage. Ed.) is full of kinetic energy, to which each single stone contributes his part.

Hsu then turns to alternative ideas of the movement mechanism, and again Shreve's air-cushion hypothesis comes under criticism. Hsu states that Shreve's conclusion, that the Elm debris did not flow, was based only on the one observation that the original sequential order of formations in the detached block is preserved in the debris at rest. Shreve argued that the preservation of order would not be possible in a viscous flow, therefore the debris slid. Hsu points out this view is irrelevant because Heim never proposed that the Elm debris flowed viscously. Heim was also the first to note the stratigraphic preservation, and this observation substantiates his explanation of the movement mechanism.

Hsu's refutation of the air-cushion concept, which also invokes forth a paragraph on lunar rock debris masses, does not really have a place in a tribute to Albert Heim. The chapter could well have ceased following

111

the excellent presentation of that great man's work with no detraction from his pellucid observations.

The substance of the volume lies of course in the chapter contents of rockslide and avalanche descriptions. A virtual wealth of information is put forth, which will ensure the necessity of having the volume handy for reference on mass movements. Most readers will never have an opportunity to study one of the mammoth rockfalls and most geotechnical engineers faced with a potential slope stability problem are not too concerned if debris flowed, slid, rolled, or floated down a prehistoric slope. Nevertheless, there are enough investigation experiences and techniques to be of interest to any engineer working on slopes. Some of the salient points of the beginning chapters are as follows.

D.M. Cruden and J. Krahn report on the Frank slide (Alberta, 1903), concluding that the movement was on bedding surfaces, not joint planes as previously thought. B. Voight describes the Lower Gros Ventre slide (Wyoming, 1925) and indicates that a flow mechanism of saturated debris was involved, not slip of sandstone strata over a clay layer as had been concluded in earlier studies.

According to J.B. Hadley, the Madison Canyon rockslide (Montana, 1959) happened on sheared and weathered rock. Air was trapped beneath the upper, faster part of the slide, but escaped and did not contribute much to the movement.

R.K. Fahnestock gives the view that the Little Tahoma Peak rockfalls and avalanches (Washington, 1963) were triggered by an explosion of volcanic steam. Air-cushions are reported by Fahnestock to have assisted in the movement of four out of seven avalanche units. This conclusion is based on the fact that one avalanche passed through a gap in which a two meter high instrument shed was located. The shattered remains of the shed were found on top of this same avalanche unit. The author stresses his opinion that the shed had not been damaged by rocks, and that its movement post-dated emplacement of the debris on which it was found, having been carried there by the air-cushion of the succeeding avalanche. He concludes that the debris had not been in contact with the ground when it crossed the shed site, some 6000 feet from the site of the debris launch.

M.J. McSaveney comes out in support of Heim's mechanism, which he terms mechanical fluidization, to explain the rapid slide of a thin flexible sheet of the Sherman Glacier Peak avalanche (Alaska, 1964). After reviewing his own and previous studies, including Shreve's, on this debris flow, McSaveney discusses the problems and differences of opinion in cal-

112

culating the velocity of debris avalanches. His treatment of this topic is the most balanced analysis of movement mechanism in the volume.

W.H. Mathews and K.C. McTaggart present a meagre update on the Hope rockslide (British Columbia, 1965). This discussion centers on the causes of the slide, which occurred in or adjacent to felsite sills, and on the difficulty that would have been involved in predicting the Hope Slide in spite of the fact that a prehistoric slide at the same site had been recognized before.

G. Plafker and G.E. Ericksen put forth some stupendous data in their report of the massive $(50\text{-}100 \times 10^6 \text{m}^3)$ Nevados Huascarán avalanches (Peru, 1962 and 1970). The average velocities determined range from 170-280 km/hr for the 1962 and 1970 events, and possibly 315-355 km/hr for an earlier prehistoric slide in the same area. The recent Nevados Huascarán avalanche is another debris flow characterized by a long run-out. This and the high velocities are believed to be related primarily to the height of the fall (3-4 km) and fluidity. These authors consider the fluidity to have been caused by the admixture of snow and water. Terminal velocities in excess of 1000 km/hr are proposed for boulders up to 65 tonnes which have travelled in the air up to 4 km. The estimate of the boulder velocity is based on ballistic trajectories and crater size, the photographs of which must be seen to be believed. The authors are admittedly puzzled by the high velocity, particularly when it comes to an explanation of the required velocity prior to the air launch. One idea suggested is that a constriction in the debris flow above the launch point resulted in a significant velocity increase.

E. Kojan and J.N. Hutchinson report that the gigantic Mayunmarca rockslide and debris flow (Peru, 1974), on the order of 10^9 m^3, started on bedding planes as a rockslide and subsequently behaved as a non-turbulent debris flow of granular materials with low cohesion and low water content. The voids of the debris during movement are thought to have been filled with air or vapour.

D.R. Piteau, F.H. Mylrea, and I.G. Blown launch their own assault on the prehistoric Downie slide (British Columbia). Their study is the most comprehensive investigation presented. In addition to photogrammetric surveys, air photo studies, and geological mapping, the work incorporated diamond drilling, ground monitoring field survey, slope indicator survey, seismic refraction and acoustic emmission surveys. The approach and experiences of this study are most relevant to engineering geologists working in areas of unstable slopes.

The next five chapters deal with other prehistoric slides. The mechanisms of movement proposed leave adequate room for future research and discussion. Mechanisms considered are gravity sliding on a regional scale (P.R. Gucwa and R.O. Kehle, Bearpaw Mtn., Montana; B. Voight and W.M. Cady, transported complexes of the Taconide zone - northeastern North America), rapid injection of volcanic gas along a bedding fault (H.J. Prostka, Heart Mountain fault, Wyoming and Montana), block glide (W.A. Braddock, Dakota Group rock slides, Colorado), and air-layer lubrication (B. Johnston, Blackhawk Landslide, California).

D.G. Moore outlines the environment of submarine slides. He gives numerous examples of seismic reflection profile interpretations and sums up the limitations of the technique. These include the limits of stratigraphic resolution (about 4 m), the inability to detect slopes steeper than $15-20^0$, and the vertical exaggeration of the seismic records.

D.H. Radbruch-Hall's chapter on gravitational creep is a must for all who map or interpret geological structures, or who construct on slopes. The contributions of this chapter are the superbly illustrated, numerous examples of creep processes. These include valleyward squeeze of ductile rocks; buckling of dipping interbedded strata of contrasting competency; movement distributed over a thick zone of relatively uniform material; incremental movement along a dipping rough, surfaced plane; deep seated bending, folding, and plastic flow rocks on slopes; and by bulging, spreading and fracturing of steep-sided ridges in mountain areas. The author terms the last of these 'spreading ridges', an unfortunate choice of words as it connotates analogy or relation to the spreading ridges of global tectonics. She considers it worth noting that known examples of spreading mountain ridges are all located in proximity to convergent or transform plate boundaries. This reviewer would agree that spreading mountain ridges are found in young mountain ranges, and that these ranges are located in belts which once were the sites of compression, but to equate directly the formation of plate boundaries to results of much younger creep mechanisms seems pointless apart from stating the obvious geographic situation. Perhaps the author meant to imply that the ridge creep was influenced by the release of regional residual stress derived from tectonism that formed the mountain ranges. If so, her presentation of this one relation is somewhat equivocal.

Additional information on creep in given by R.F. Scott in his description of the instrumentation used to record regular and periodic incremental movement of a rockslide, and by J.J. Emery's finite element analysis of creep mechanism.

114

J. Goguel points out in his chapter that the mechanism of emplacement is scale dependant. Medium scale landslides are often rotational, but that the large scale slides are characterized by fluidization and debris flow. He proposes that vaporization of pore fluids occurs due to frictional heat generated at the base of the slide. The resulting steam affects the fluidization, reducing the friction between solids.

W.R. Jacoby closes the section on rock creep and scale effects with his chapter on the role of gravity in plate tectonics. He considers that the lithospheric plates represent gigantic rockslides driven by gravity. How this supposition is going to benefit us on the surface is not explained, nor is it known why this chapter is in this volume. The topic would be more suitably placed in *Tectonophysics*, as it is not a relevant expression or summation of the natural phenomena (as by the Preface) of either plate tectonics or rockslides and avalanches. In any case Jacoby has done a creditable job of riding the two bandwagons of global tectonics and mass movements.

The volume finishes off with chapters on snow avalanches; both the aspects of the failure process (R.I. Perla) and of dynamics of movement (M.Mellor) and a complete treatment of knowledge on glacier behaviour and movement (C.F. Raymond). The last author points out that conditions present at the base of a glacier are unknown, making present theories of sliding largely theoretical.

This reviewer arrived at the end of the volume overwhelmed, to say the least. One can only feel respect for the man who ran a few paces in front of the Elm debris flow, or for the survivor of the Mayunmarca slide who rode a block down the flow. Nightmares are reported to be a common occurrence among readers of the volume, with visions of attempting to track a 65 tonne boulder travelling at 1000 km/hr, measure the height of an air-cushion, collect samples of volcanic gas injected, or steam generated, beneath a debris flow, or trying to determine if the flow is travelling at 330 km/hr or only 110 km/hr.

There is enough material in the text to warrant the cost and to interest the geologist, geomorphologist or geotechnical engineer for several readings. The controversial nature of the subjects covered, the reviews in some chapters, and especially the data in the descriptive accounts, will result in the text being the source of information on rockslides for years to come.

K.V. Campbell

Lectures of the Seminar 'Failures of Large Dams - Reasons and Remedial Measures', January 6-7, 1977, Aachen, edited by Walter Wittke, Volume 4, Publications of the Institute of Foundation Engineering, Soil Mechanics, Rock Mechanics and Waterways Construction, RWTH (University) Aachen, F.R. Germany, 242 pp. D.M. 23.00 (includes postage).

The volume contains seven articles contributed by German experts in the field of dam engineering with the exception of G.F. Sowers from U.S.A. Five articles deal directly with various aspects of earth and masonry dams with particular reference to their performance, damages due to several causes and also the repair and restoration works. The other two articles deal with the topics of coefficient of friction during a landslide and the problems of intersecting structures in different sections, respectively.

The first article by Gerhard Rouve reviewed several cases of damages related to dams by drawing attention to the failure of over 300 dams of different types and ages. Readers of this article can benefit from a useful study made by the author of the evaluation of the cases of damages according to the type of dams and the relative damage frequency presented with respect to years of operation.

The second paper by Berthold Strack is devoted to the repair of two dams, namely the upper Herbringhauser dam and the Kerspe Dam. Both dams are of the gravity masonry type and designed by Professor Intze of Aachen, Germany. A useful feature of this article is the description of the damages of the dams, which resulted mainly from the long period of operation, and the subsequent investigations related to the restoration works. Several types of grouting works are described and the use of chemical strengthening and electro-osmosis can be considered as unconventional for the cases discussed.

The third article by Karl Heinz Idel and Joachim Beckmann describes the aging and restoration works on six gravity type masonry dams. The geological conditions of the foundation for all six dams are briefly mentioned and the causes of the dam repair in all instances seem to have arisen from the increase in permeability of the dam wall and the foundation. The restoration works described varied from dam to dam but consisted in most cases of several types of grouting. The use of control gallery in grouting works, and the remedial measures related to the safety of the dams against sliding and tilting as well as to the uplift pressures are found to be interesting. The restoration works related to Mohne dam were very informative dealing with the use of bituminous seal coat and the so called smooth blasting technique for the construction of access tunnels in grouting works.

In the article titled 'Problems with Intersecting Structures in Dyke Sections', Martin Hager deals specifically with the description of several intersecting structures and the damage of the Elbe Seikenkanal. A detailed description of soil erosion to avoid failure of canal linings is emphasized.

The fifth article deals with the coefficient of friction during a landslide and was authored by Bohumil Boucek. It describes the sliding of a slope approximately 300 m wide and 400 m long in the area of a reservoir in the French Alps in 1964. A geological cross section of the slope has been given and the geotechnical exploration consisted of borings, seismological and geo-electrical measurements, pore water pressure measurements, inclinometer measurements and also the horizontal and vertical displacements of sixty points on the slope. The sliding could have been due to excess pore pressure or seismic influence. The coefficient of friction, $f = \tan 6°$, appears to be low.

The paper titled 'Earth Dam Failures' by George F. Sowers begins with the description of the Johnstown flood and the subsequent failure of an earth dam in U.S.A. which resulted in tremendous damage. This article is well written and is illustrated with several well known dam failures. The significances of earth dam failures is discussed in detail with special reference to the magnitude cf the problem, the awareness of failure, safety regulations and the value of failures. The dam failures are broadly classified as three types: hydraulic failure, seepage failure and structural failure. Each type of failure is then discussed in detail.

The last article in the series of lectures is on 'Crack Damage of Dams' by Bernhard Gilg wherein emphasis is made on the necessity of the design of earth dams to take cracks into consideration. The author well illustrates his emphasis on cracks by referring to the case history of Mattmark Dam which showed cracks after completion, but since the design of the dam accommodated for such cracks, no permanent damage was caused.

The reviewer of this publication believes that many of the articles included in the volume are of immense value to practising engineers who are directly involved in the design, safety, and operational aspects of earth and masonry dams. It can also be of considerable interest to graduate students who are following courses in Dam Engineering. The volume contains several photographs, tables and charts which are not otherwise found in similar publications. The publication however, needs some editorial improvements since there are many typographical errors.

A.S. Balasubramaniam

Stress in Subsoil and Methods of Final Settlement Calculation, by Jaroslav Feda, Developments in Geotechnical Engineering, Vol. 18, Elsevier Scientific Publishing Company, P.O. Box 330, Amsterdam, The Netherlands, 1978, 215 pp. 64 Figs., 3 Tables, US$ 35.50 (Dfl. 80.00).

This monograph presents a critical review of the various methods used for final settlement calculations. It deliberately does not deal with the theories of primary and secondary consolidation, i.e. it excludes the time development of settlement.

The text is divided into three sections. In the first section *in situ* stresses in the subsoil are reviewed. Particular attention is given to the proper evaluation of the coefficient of earth pressure at rest K_0, i.e. the factors influencing its magnitude and the methods of measurement. The second section deals with the stress state produced by external loads. Only the stress distribution in the elastic half space is considered, plastic or viscous behaviour are ignored. However, the aspects of anisotropy, nonhomogeneity and non-linearity are discussed in detail. The author concludes that the most suitable theory for the elastic half-space should be the nonhomogeneous model since it can approximately also take into account effects of anisotropy and non-linearity. Various methods of stress measurement are also summarized.

Finally, the third section considers the relevant methods to compute final settlement. Four groups are distinguished, namely (i) direct or exact methods, using the theory of elasticity, (ii) indirect or engineering methods, which include the stress path, the state boundary surface, the standard oedometer and the Skempton-Bjerrum method, (iii) empirical methods such as plate load tests, cone penetration tests and pressuremeter and (iv) numerical methods. This last group is particularly devoted to the finite element method, and the author successfully highlights the advantages of this powerful tool over the more classical methods. He concludes further that the stress path method and the finite element method may at present be considered as the most realistic procedures for settlement analysis.

While the book represents a good and well written review and synthesis of the present state-of-the-art, the treatment is often too brief and sketchy to be of direct use to the practicing engineer. Those not familiar with a certain method but would like to employ it will have to go back to the original references. Not a single numerical example is given and illustrations and tables are also sparse. The references are abundant but a good number of them are written in Eastern European languages, particularly Russian, and may therefore not be accessible to Western readers. Nevertheless,

120

the book is useful for the research oriented geotechnical engineer. It will assist him in selecting the most appropriate method of analysis and direct him to the pertinent literature.

R.P. Brenner

Soil Mechanics and Foundation Engineering, by Bharat Singh and Shamsher Prakash, Nem Chand & Bros., Roorkee (U.P.), India, 4th Edition, 1976, 564 pp., Rs. 22.50.

This textbook probably does not need any introduction for Indian readers, since the need of printing a fourth edition within thirteen years may well demonstrate its popularity. Three chapters have been added to this new edition, namely on deep foundations, machine foundations and on embankment dams. The text is well illustrated and contains numerous solved examples and useful problems, most of them with answers. The book is written for undergraduate students at Indian universities and contains hardly any material that cannot be found in already existing standard texts, but the authors' original aim was to create an elementary treatise that incorporates under one cover all that is needed in routine engineering practice. While the reviewer is not familiar with the first edition of 1963, he has no doubt that at that time the book was of great educational value. However, it, appears to him that since then, few revisions have been made to the texts including this latest edition. The style of presentation is old fashioned and there are today better ways to present essentially the same material, but emphasizing new concepts which must be taught at the undergraduate level. In fact most of the references date from the fifties and early sixties. More recent and popular practices, such as the use of the cone penetration test, are mentioned in only a few lines. The new chapters added, such as the one on machine foundations, are completely outdated.

This does not mean that book is no longer useful as an elementary text, but unless the authors take the trouble to entirely revise it in a future edition the popularity of the book might decline, which would be regrettable.

R.P. Brenner

Ring Shear Tests on Clay by Bohumil Boucek and **Determination of the Bearing Capacity and Pile Driving Resistance of Piles Using Soundings** by Dieter Rollberg, edited by Walter Wittke, Volume. 3, 1978, Publications of the Institute for Foundation Engineering, Soil Mechanics, Rock Mechanics and Water Ways Construction, RWTH (University) Aachen, Federal Republic of Germany, 227, pp., DM 23.00 (includes postage).

The production of an English edition of this publication series is a worthwhile undertaking as it helps to make results of German research efforts accessible to a wide readership. It is hoped that other German research institutions will follow this example.

This volume contains two contributions, both of them based on doctoral dissertations. In the first paper, a newly developed ring shear testing device to measure the residual strength parameters is described. Past experience has shown that the usual determination of these parameters in direct shear boxes with reversible shear direction or in triaxial tests with a given surface leads to values which show a considerable scatter and the selection of the correct parameters becomes uncertain. The new apparatus discussed is designed such that no normal pressure and no torque arise in the joint between the upper and the lower side rings. This allows a direct measurement of the normal force and the torque acting on the sample. Drained tests at constant volume can be performed with this device and a distinct shear surface can be observed in the annular sample. Test results from remolded clay samples reveal that upon appearance of such a distinct shear surface a reduction in the shear resistance takes place after failure. This reduction can be quantified by introducing a 'brittleness index' which is a function of the overconsolidation ratio. The reduction is larger than in the case where no distinct shear surface occurs, i.e. when failure with constant shear strain over the sample height takes place, as happens with some of the other shearing devices. In addition, for overconsolidated samples sheared with a distinct failure surface the peak strength was less than for samples which failed under constant shear strain over the sample height.

The new device appears, therefore, to produce results of high quality and will be a useful research tool for the strength properties of clays.

The second contribution deals with the prediction of pile capacity and pile driving resistance from sounding tests and is of great practical usefulness. The author has collected data from 248 pile load tests (both compression and tension loading) from 55 different construction sites and involving various types of driven piles as well as bored piles in cohesionless and cohesive soils. For each test pile, a soil profile and at least one sounding record

were available. Most of the soundings were obtained by the Dutch cone penetrometer, but data from other types of sounding devices were also available. The load-settlement curves of the pile load tests were approximated by a modified hyperbolic relationship and from this a failure load was defined. By employing statistical methods, it was possible to establish empirical equations and graphs to find the bearing capacity, the load settlement curve, the dynamic pile driving resistance (i.e. pile driving work and permanent set) as a function of the penetrometer work. A numerical example is also appended. The predictions from this new technique and those obtained from a large number of conventional methods were compared with the actual loading test results and it was found that the new method gave a considerably better correlation.

This fresh approach to the rather old problem of predicting the bearing capacity is a welcome addition to the literature on the application of soundings, particular static cone penetration. The reader will enjoy the study of this well written thesis and will most likely be tempted to try the method with his own data.

R.P. Brenner

Soil Mechanics for Off-Road Vehicle Engineering by L. L. Karafiath and E. A. Nowatzki, Trans Tech Publications, CH-4711 Aedermannsdorf, Switzerland, 1978, 550 pp., 204 figures, US$ 54.00 (sfr 135.00).

Off-road vehicle engineering is a highly specialized discipline and entire books devoted to it are very rare. In fact, since the last comprehensive treatise was published by M. G. Bekker, almost a decade has passed during which major breakthroughs have been accomplished. Since the running gear of vehicles moving off-road is in direct interaction with the soil surface, soil mechanics must play a major role in the performance of such vehicles. Nevertheless, not until recently have advanced concepts in soil mechanics been applied to off-road locomotion problems and the authors have rendered a great service to both vehicle designers and soil mechanics specialists by presenting a treatise which is based on sound soil mechanics principles and which incorporates the results of the latest research efforts in the analysis of off-road locomotion problems.

The book is divided into three parts. Part 1 contains an overview of soil mechanics principles, particularly those relevant to off-road locomotion problems. Most of this material can also be found in standard textbooks, but some advanced topics are also treated in detail.

124

Part 2 presents a detailed treatment of the theory of plasticity for soils. It is particularly the application of plasticity theory together with numerical methods which has enabled major advances in the solution of mobility problems. Sokolovskii's basic differential equation for plane strain is first derived, followed by the axially symmetric case. Then the effects of nonlinear yield criteria and inertia forces are discussed. The basic differential equations are also given in terms of effective stresses. Full details for the numerical analysis of slip line fields are given and the various boundary conditions thoroughly discussed.

Part 3 deals with the interaction between soil and running gear (rigid wheels, pneumatic tires and tracks) and the theories presented in the first two parts are applied. Mathematical models are developed for rigid wheel and pneumatic tire under a variety of loading conditions. Descriptions of computing techniques as well as flow charts are presented in great detail. The validity of the models is verified by comparison with experimentally determined values of drawbar pull, towing resistance and sinkage. For tracks no mathematical models are available yet, but the concepts of track-soil interaction are clearly stated.

Field tests to determine the necessary soil parameters (e.g cone penetration tests, plate sinkage tests, ring shear tests, etc. are critically evaluated and their limitations pointed out. However, in view of the importance of these parameters, this section is somewhat short and should have deserved more attention.

The book is clearly written and the production is of high quality. It is suitable as a text for a course in off-road locomotion. Engineers working in the design of vehicles for military, agricultural and construction purposes will find it of enormous help. The study of this book can also be recommended to the research-minded geotechnical engineer as he will no doubt be able to extract many useful ideas.

R.P. Brenner

Books Received

Determination of the Water Permeability of Jointed Rock, by Peter Rissler, Volume 5, 1978, Publications of the Institute for Foundation Engineering, Soil Mechanics, Rock Mechanics and Water Ways Construction, RWTH (University) Aachen, F.R. Germany, 150 pp., DM 23.00 (includes postage).

On the basis of the well-known flow laws for one dimensional laminar and turbulent flow in a fissure the seepage phenomena occurring in the vicinity of the borehole during a water pressure test of a joint are investigated. The results show that the parameters decisive for the permeability can be evaluated from the relationship between the flow rate q and the energy head H_o as measured in the test. In the case of more than one discontinuity with different geometries the test section must be subdivided in a special manner. Using the evaluated decisive parameters the anisotropic permeability of the rock mass can be expressed by a tensor.

(From accompanying summary.)

Fundamentals for the Design and Construction of Tunnels Located in Swelling Rock and Their Use During Construction of the Turning Loop of the Subway Stuttgart, by Walter Wittke, Volume 6, 1978, Publications of the Institute for Foundation Engineering, Soil Mechanics, Rock Mechanics and Waterways Construction, RWTH (University) Aachen, F.R. Germany, 131 pp., DM 23.00 (includes postage).

A procedure of calculating the stresses and displacements caused by swelling, which is based upon the Finite Element Method is presented. This method is also suitable for the design of tunnel linings in swelling rock and is based on a three dimensional swelling law, which has been derived from a relationship determined by Grob for the stress conditions of a confined compression test.

Furthermore, the influences of the *in situ* stresses as well as those of the shape of the cross-section, the thickness of the rock overburden and the type of the lining of a tunnel upon the displacements due to swelling are investigated using comparative calculations. Using the example of tunnels, which have already been completed and are described in the literature, the applicability of the calculatory procedure was examined by means of an interpretation of the measured swelling stresses and displacements.

A detailed project study of the design and construction of a tunnel for the Stuttgart subway is reviewed. An economical and safe design of the tunnel tubes was achieved using the rock mechanical parameters, derived from laboratory tests and a measurement program carried out during the construction period, in the calculatory method described as well as by application of adequate construction rules.

(From accompanying summary.)

CONFERENCE NEWS

Fourth Rapid Excavation & Tunneling Conference, Atlanta, Georgia, U.S.A., June 18 to 20, 1979. All enquiries to : Society of Mining Engineers, P.O. Box 8800, Salt Lake City, Utah, 84108, U.S.A.

Sixth Asian Regional Conference on Soil Mechanics and Foundation Engineering, Singapore, July 24 to 27, 1979. All enquiries to : Dr. Tan Swan Beng, Organizing Chairman, 6 ARC, c/o Institution of Engineers, Singapore, BLK. 23, 2nd Floor, Outram Park, Singapore 3.

International Symposium on Soil Sampling, Singapore, July 28, 1979. All enquiries to : Mr. Hiroshi Mori, c/o Mori Geotechnique, Inc., Room 1005 Sunheim Tamachi, 3-2-9, Kaigan, Minato-ku, Tokyo, Japan.

Fourth International Congress on Rock Mechanics, Montreux, Switzerland, Sept. 2 to 9, 1979. All enquiries to : "Secrétariat du Congrès SIMR 1979, Case Postale 98, 1000 Lausanne, Switzerland.

Symposium on Engineering Geological Mapping, Newcastle upon Tyne, U.K. Sept. 3 to 6, 1979. All enquiries to : Symposium Secretary, Engineering Geology Unit, Drummond Building, Univ. of Newcastle upon Tyne NEI 7RU, U.K.

Seventh European Conference on Soil Mechanics and Foundation Engineering, Brighton, U.K., September 10 to 13, 1979. All enquiries to : Conference Secretary, 7th ECSMFE, Institution of Civil Engineers, Great George St., London SWIP 3AA, U.K.

International Symposium on Engineering Geological Problems in Hydrotechnical Construction, Tbilisi, U.S.S.R., Sept. 12 to 19, 1979. All enquiries to : Prcf. Jesef M. Buachidze, Corresponding Member of the AS Georgian SSR, Chairman of the Organizing Committee, Department of Hydrogeology and Engineering Geology, The Academy of Sciences of the Georgian SSR, 52 Rustaveli Avenue, Tbilisi 380008, U.S.S.R.

International Conference on Computer Applications in Civil Engineering, Roorkee, India, October 1979. All enquiries to : Dr. G.C. Nayak, Organizing Secretary, ICCACE, Civil Engineering Department, University of Roorkee Roorkee, U.P. 247672, India.

Thirteenth Congress of the International Commission on Large Dams (ICOLD), New Delhi, India, Oct. 25 to Nov 2, 1979. All enquiries to: Sec-

retary, Indian Natl. Committee for ICOLD, c/o Central Board of Irrigation & Power, Kasturba Gandhi Marg, New Delhi, India.

Sixth Panamerican Conference on Soil Mechanics and Foundation Engineering, Lima, Peru, December 2 to 7, 1979. All enquiries to: Arnaldo Carrillo Gil, Presidente, Comite Organizador del VI CPMSIF, Jr. Emilio Tomandez 296-OF. 703, Apartado Postal No. 11076, Lima, Peru, S.A.

International Symposium of In-situ Testing of Soils and Rocks and Performance of Structures, Roorkee, India, December 19-22, 1979. All enquiries to: Dr. Swami Saran, Organizing Secretary, Symp, In-situ Testing of Soils and Rocks, Room No. 211, Department of Civil Engineering, University of Roorkee, U.P. 247672, India.

International Symposium on Landslides, New Delhi, India, April 7 to 11, 1980. All enquiries to : The Organizing Secretary, International Symposium on Landslides, P.O. Central Road Research Institute, New Delhi 110 020, India.

Third Australia - New Zealand Geomechanics Conference, Wellington, New Zealand, May 1980. All enquiries to: Organizing Secretary, The 3rd Australia-New Zealand Geomechanics Conference, P.O. Box 243, Wellington, New Zealand.

Conference on Structural Foundations on Rock, Sydney, Australia, May 1980. All enquiries to: P.J.N. Pells, Conference Chairman, School of Civil Engineering, University of Sydney, Sydney, Australia 2006.

Sixth Southeast Asian Conference on Soil Engineering, Taipei, Taiwan, R.O.C. May 19-23, 1980. All enquiries to: Secretary General, Organizing Committee, 6th SEACSE, c/o Moh and Associates, 6-1, Lane 137, Yen Chi Street, Taipei, Taiwan, Republic of China.

Seventh World Conference on Earthquake Engineering, Istanbul, Turkey, September 8-13, 1980. All enquiries to : Organizing Committee, 7 WCEE, Deprem Arastirma Enstitusu, Yuksel Caddessi, 7/B, Ankara, Turkey.

International Symposium on Weak Rock, Tokyo, Japan, September 1981. All enquiries to : Secrétariat of the International Symposium on Weak Rock, c/o Japan Society of Civil Engineers, Yotsuya 1-chome, Tokyo 160, Japan.

NEWS OF PUBLICATIONS

Proceedings of the Third International Conference on Applications of Statistics and Probability in Soil and Structural Engineering, Sydney, Australia, January 29 to February 2, 1979, 3 Volumes. Available from: Unisearch Ltd., University of New South Wales, P.O. Box 1, Kensington, N.S.W. 2033, Australia.

Proceedings of Geocon-India, New Delhi, India, December 20-22, 1978, Rs. 250 + postage. Available from: Prof. Shashi K. Gulhati, Dept. of Civil Engineering, Indian Institute of Technology, New Delhi, 110029, India.

Soil Reinforcing and Stabilizing Techniques in Engineering Practice, Proceedings of a Symposium, Sydney, Australia, October 16-19, 1978, A.$40.00. Available from : School of Civil Engineering, New South Wales Institute of Technology, P.O. Box 123, Broadway, N.S.W. 2207, Australia.

NEWS OF
SOUTHEAST ASIAN SOCIETY OF SOIL ENGINEERING

Regional Engineering Geology Group of SEASSE

The Governing Council and the General Assembly of the International Association of Engineering Geology (IAEG) at its meeting in Madrid, 2-8 September 1978, unanimously approved the affiliation of the Southeast Asian Society of Soil Engineering (SEASSE), as a Regional Group with the creation of an Engineering Geology Section. Members of SEASSE should be grateful to the Honorary President, Prof. Marcel Arnould; the Secretary General, Dr. Richard Wolters; Past-President, Prof. Asher Shadmon; President-Elect, Prof. Yergeiy Sergeev, Dr. Tanaka Haruo (V.P. for Asia), Prof. W.R. Dearman and other council members for their strong support to SEASSE.

The General Committee of SEASSE is now working on the office bearers of the Engineering Geology Section and also the membership fees, etc. These details will be announced to members of our Society before January 1979. In view of the creation of the Engineering Geology Section and the initiatives to set up a Rock Mechanics Group, the General Committee will also decide on the proposed change of the title of our Society from SEASSE to SEASGE (Southeast Asian Society of Geotechnical Engineering).

International Tunnelling Association Working Group on Research and Development in Shield Tunnelling

The ITA-working group 'Research' decided at their Tokyo meeting this year to deal with special subjects in the field of research and development. For the Atlanta meeting in 1979, the subject "shield tunnelling" was chosen and Mr. Don V. Deere (U.S.A.) was appointed as the General Reporter. The task of the committee is to gather and analyze the data on shield tunnelling. The co-reporters of the working group are Mr. McCusker for North America, Dr. Wagner for Africa, Prof. Girnau and Dr. Haack for Europe and Prof. Murayama for Asia. Among other information, the working group is interested in:

-- a list of shield tunnels finished or in progress from 1975 to 1978 in each member country,
-- shield tunnel research projects completed or in progress, if any, and
-- promising trends and future developments in shield tunnelling.

Members of our Society who have information on these aspects are kindly requested to forward them to our Secretariat or directly to Prof. Sakuro,

Murayama, c/o Japan Tunnelling Association, Shinko Dai-ichi Bldg., 7-14 Shintomi, 2-chome, Chui-ku, Tokyo 104, Japan.

Committee on Science and Technology for Developing Countries (COSTED)

COSTED is an International Committee on Science and Technology for Developing Countries. One of its primary objectives has been to link science education with national development. To promote this objective several programs have been organized in developing countries and the primary participation is drawn from scientists in such countries. Originally established in 1966 under the chairmanship of Prof. M.S. Blackett COSTED did commendable work in identifying several problems of developing countries. The re-constituted COSTED under the chairmanship of Prof. S. Bhagavantam has been attempting to tackle these problems at different levels of society. COSTED is a committee of the International Council of Scientific Unions and it has been supported by UNESCO in several of its programs. The present membership of COSTED is Prof. S. Bhagavantam, President (India); Mr. D. Bekoe (Ghana), Mr. A. Muhammed (Pakistan), Mr. N.W. Pirie (U.K.), Mr. R. Revelle (U.S.A.), Mr. J. Sahade (Argentina), Mr. Sammani Yacoub (Sudan), Mr. E. Shershnev (USSR), Mr. D. Sastra-pradja (Indonesia), and Mr. S. Radhakrishna, Scientific Secretary (India).

Conference on Solid Waste Management

Some 37 participants from seven Asian countries attended a six-day regional seminar on Solid Waste Management at the Asian Institute of Technology from September 25-30. The seminar was sponsored by the Carl-Duisberg Gesellschaft (CDG) of Germany and AIT's Environmental Engineering Division. The opening address was given by German Ambassador to Thailand, H.E. Edgar Von Schmidt Pauli.

Seminar discussions focused on topics such as Sanitary Landfill, Treatment and Disposal of Hazardous Wastes and Specific problems of Waste Management. The seminar objective was to enable participants from different Southeast and East Asian countries to gain a deeper understanding of the techniques and methods for Waste Management Planning and also, to be better able to make decisions relating to the handling, treatment, recovery and disposal of waste, etc. More information can be obtained from:

> Dr. Nguyen Cong Thanh,
> Division of Environmental Engineering,
> Asian Institute of Technology,
> P.O. Box 2754,
> Bangkok, Thailand

Association of Geoscientists for International Development (AGID)

The Association of Geoscientists for International Development (AGID) is an international, non-profit making and non-governmental organization founded in 1974. The many activities of AGID cover a wide range of the geosciences from groundwater and mineral exploration and resource management to education and information dissemination.

The Governing Council of AGID includes Prof. D.E. Ajakaiye (President), Ing. A. Bellizzia (Secretary) and the following council members: Dr. A. Al-Shanti, Dr. S. Bonis, Prof. A. Bhaskara Rao, Dr. M.B. Katz, Mr. S.D. Limaye, Prof. Lamnitz, Mr. R. Lukman, Dr. P. Nutalaya, Dr. B.K. Tan, Prof. W. Uytenbogaardt, Dr. D.R. de Vletter, Mr. M.E. Woakes and Ing. E. Herrero.

Individual membership in AGID stands now at nearly 1100 from 94 countries. The finances necessary to run AGID's affairs have come from a number of national and multi-national agencies, and from its organizational members. AGID is planning to expand its activities and is developing a major on-going training program which will be held in a different country each year on a topic relevant to that region. The Council Members of AGID in the member countries of SEASSE are Dr. B.K. Tan, Dept. of Geology, University of Malaya, Kuala Lumpur, Malaysia; and Dr. Prinya Nutalaya, Asian Institute of Technology, P.O. Box 2754, Bangkok, Thailand. Dr. Antony R. Berger, the former Secretary General of AGID, recently spent part of his sabbatical leave at AIT making a detailed study of geoscience research priorities and their state of development in S.E. Asia. For further informationon AGID please write to:

> Prof. Deborah Enilo Ajakaiye,
> Head, Dept. of Phsycis,
> Ahmadu Bello University,
> Zaria via Kono,
> Nigeria

or

> Ing. Alirio Bellizzaia G.,
> Director, Direccion de Geologia,
> Ministerio de Energia y Mines,
> Torre Norte - Piso 19,
> Centro Simon Bolivar, Caracus,
> Venezuela

Prof. Dinesh Mohan, Vice-President of ISSMFE for Asia Co-ordinates Program on Human Settlement Development

Prof. Dinesh Mohan, the Vice President of the International Society for Soil Mechanics and Foundation Engineering for Asia and the Director of the Central Building Research Institute, Roorkee, has been invited by ESCAP to set up an Information Centre for Human Settlements Development in Asia. Accordingly Prof. Mohan has recently visited many member countries of SEASSE and has had discussions with specialists in the field of Human Settlement Development.

Prof. Masami Fukuoka, President of ISSMFE to Visit AIT

Prof. Masami Fukuoka, the President of the International Society for Soil Mechanics & Foundation Engineering, will visit AIT during the 3rd Week of July 1979. During this visit Prof. Fukuoka and a team of leading Japanese specialists in Soil Mechanics & Foundation Engineering will deliver a series of popular lectures on subject in Geotechnical Engineering of current interest to practicing engineers in Thailand. Prof. Fukuoka's visit to AIT will certainly add spice to the Technical Program associated with the 20th Anniversary of the Founding of the Asian Institute of Technology, which will include the visits of a large number of leading Professional and academic engineers to AIT in 1979. Further details of the lectures will be announced in the next Society Circular.

Second Conference of the Road Engineering Association of Asia & Australasia

The Second conference of the Road Engineering Association of Asia and Australasia was held in the Philippines International Convention Center in Manila from 16th to 20th October 1978. Over 1,200 delegates from more than 20 countries were present at the conference which was formally opened by His Excellency President & Prime Minister Ferdinand E. Marcos, and the welcome address was given by Hon. Baltazar Aquino, The Minister of Republic Highways in the Philippines. The familiar faces of Dr. Za-Chieh Moh, Dr. Chai Muktabhant and Mr. Nibon Rananand (who are the General Committee Members of SEASSE) were among the Council Members of the Road Engineering Association who greeted the Presidential Party during the opening ceremony. In his keynote address President Marcos stressed the importance of the shift in priority from the building of national highways to the construction of barangay and farm-to-market roads in the villages.

The breakdown of the technical papers in the Conference was as follows: Planning, Financing and Administration (15); Soil Engineering & Soil

133

Stabilization (12); Urban Traffic & Transport Planning (16); Low-cost Roads (7); Construction and Maintenance Practice (14); and Design Techniques (12).

The Road Engineering Association of Asia & Australasia with over 350 individual members and several institutional members organizes regular conferences every two years. The venue for the next conference, to be held in 1980 will be announced in February 1979. Further information on the Road Engineering Association and its activities can be obtained from

> Mr. Harry Huen,
> The Honorary Secretary-General,
> The Road Engineering Association of Asia & Australasia,
> c/o Atlas Industries Sdn. Bhd.,
> 126 Jalan Kasah, Damansara Heights,
> Kuala Lumpur,
> Malaysia

Celebrating the 20th Anniversary of the Founding of the Asian Institute of Technology

During the year 1979, which will be the 20th Anniversary of the founding of the Asian Institute of Technology, several short courses and seminars will be organized at the Asian Institute of Technology. The first in the Series was a short course on "Roads and Transport" organized by AIT and the British Transport and Road Research Laboratory which took place at AIT from 22nd January to 2nd February, 1979. The objective of this ten-day course was to present, through lectures and account of practical experience, important developments in road transport for both urban and rural conditions. Admission to the course was limited to 30 selected participants. Partial travel grants were made available by the British Government for 20 participants travelling to Thailand. The course lecturers included Dr. G.D. Jacobs, Messers A.J. Plumbe, P.R. Fouracre, A.J. Downing and N.W. Marler and Prof. John Hugh Jones. Further information on the course can be obtained from:

> Prof. John Hugh Jones,
> Road & Transport Course Coordinator,
> c/o Division of Geotechnical &
> Transportation Engineering,
> Asian Institute of Technology,
> P.O. Box 2754,
> Bangkok, Thailand

New Academic Division of Energy Technology at AIT

The new division of Energy Technology to be established at the Asian Institute of Technology in early 1980 will focus on practical engineering studies covering a wide range of energy resources and conservation methods, with emphasis on renewable sources of energy.

The energy industry at present is firmly structured around petroleum. Towards the end of this century an increasing demand for energy may shift to fuels that are more abundant or that can be produced domestically, such as the production of oil from shale. In the search for new ways to produce power from fossil fuels, generating systems that combine gas and steam turbine seem to play a central role. Coal is the largest source of energy but research to improve the use of coal has been somewhat limited. Magneto hydro-dynamic generators converting combustion gas to electricity (which require less fuel and produce much less thermal pollution) seem a promising development. In recent years nuclear energy has emerged as a major addition to traditional energy sources. Exploration for and mining of uranium for existing nuclear reactors has thus become a substantial business.

Even if further research significantly improves the use of fossil and nuclear fuels, the development of other energy sources seems a necessary task. Other alternative ways to obtain energy seem to depend mainly on the radiation of sun, the energy of the tides and the heat inside the earth. Among the renewable sources of energy, hydro-power is a well-proven indirect use of solar energy. Scientists believe other direct uses of solar energy, such as solar powered systems for heating and cooling homes, can be developed in the near future at prices competitive with gas or oil.

Geothermal power has already been used in several countries including New Zealand, the U.S.A., Italy, Japan and the Soviet Union. Possible lines of research in Geothermal Energy include the determination of the precise type and size of available Geothermal Resources. Prospecting techniques are to be developed in relation to the exploration for deposits of heat. Environmental effects of geothermal energy include the disposal of waste waters particularly when they are highly mineralized. Re-injection of waste water can help to prevent land subsidence when large quantities of water are removed from underground reservoirs. Also, injection or withdrawl of water near faults may trigger seismic activity and geophysical studies may prove useful in such circumstances.

Thus the establishment of a new academic division of Energy Technology at AIT will result in interesting research and developments, even in other existing fields of studies which will continue to contribute to the development of Asian countries.

135

Graduate Program in Computer Application Technology at the Asian Institute of Technology

Recognizing the significant role that the computers are going to play in the developing countries, the Asian Institute of Technology (AIT) has established a graduate program in the area of computer application technology. The new program, with the aim of producing the computer technologists needed in the region, will concentrate on user oriented education and applied research seeking practical ways in which computer technology can be efficiently utilized within developing countries.

Unique interdisciplinary graduate education and research opportunities are now available to prospective candidates with undergraduate preparation in engineering, physical sciences or mathematics and statistics.

Dr. M.N. Sharif,
Chairman, Computer Applications Division,
Asian Institute of Technology,
P.O. Box 2754,
Bangkok, Thailand

Programs in Computer Application Development at the Asian Institute of Technology

AIT's Programs in Computer Application Development (PCAD) were established in 1976 to help enhance the effective use of computers in meeting Asian challenges. The PCAD provide practical instruction in the use of the computer as a part of the problem solving process. The programs are conducted by the Asian Institute of Technology Regional Computer Center (AIT-RCC), a computing laboratory which includes:

-- an instructional staff skilled in the development of applications of computers to a wide range of problem areas,

-- a computer system and associated software on which participants in the PCAD receive practical training, and

-- a building which houses the instructional program, the computer system, and related laboratory and other working facilities.

Enquiries for admission to the PCAD should be made to:

The Director,
Regional Computer Center,
Asian Institute of Technology,
P.O. Box 2754,
Bangkok, Thailand

136

General Committee of the Southeast Asian Society of Soil Engineering for the period 1978-80.

Dr. Tan Swan Beng (Singapore)	President
Dr. A.S. Balasubramaniam (AIT)	Secretary-Treasurer
Dr. Za-Chieh Moh	Founder-President
Prof. Chin Fung Kee	Past-President
Prof. Peter Lumb	Past-President
Dr. Chai Muktabhant (Thailand)	Elected Member
Mr. S.G. Elliott (Hong Kong)	Elected Member
Prof. Salvador Reyes (Philippines)	Elected Member
Prof. J.J. Hung (Taiwan, R.O.C.)	Elected Member
Dr. Ting Wen Hui (Malaysia)	Elected Member
Dr. Vincent Campbell (AIT)	Editor

The membership application forms and other details can be obtained from:

The Secretary, SEASSE,
c/o Division of Geotechnical & Transportation Eng.,
Asian Institute of Technology,
P.O. Box 2754,
Bangkok, Thailand

A.S. Balasubramaniam

SI UNITS AND SYMBOLS

The following list of quantities, SI (Système International) units and SI symbols, are recommended for use in Geotechnical Engineering.

Quantities	Units	Symbols
Length	kilometre	km
	metre	m
	centimetre	cm
	millimetre	mm
	micrometre	μm
Area	square kilometre	km^2
	square metre	m^2
	square centimetre	cm^2
	square millimetre	mm^2
Volume	cubic metre	m^3
	cubic centimetre	cm^3
	cubic millimetre	mm^3
Mass	tonne	t
	kilogram	kg
	gram	g
Density ρ (mass density)	tonne per cubic metre	t/m^3
	kilogram per cubic metre	kg/m^3
	gram per cubic centimetre	g/cm^3
Unit weight γ (weight density)	kilonewton per cubic metre	kN/m^3
Force	meganewton	MN
	kilonewton	kN
	newton	N
Pressure	megapascal	MPa
	kilopascal	kPa
Energy	megajoule	MJ
	kilojoule	kJ
	joule	J
Coefficient of volume compressibility or swelling m_v	1/megapascal	MPa^{-1}
	1/kilopascal	kPa^{-1}
Coefficient of consolidation or swelling c_v	square metre per second	m^2/s
	square metre per year	$m^2/year$
	square centimetre per second	cm^2/s
Hydraulic conductivity k (formerly coefficient of permeability)	metre per second	m/s
	centimetre per second	cm/s

NOTES: The term specific gravity is obsolete and is replaced by relative density. The former term relative density $(e_{max} - e)/(e_{max} - e_{min})$ is replaced by the term density index, I_D.

139