EFFECTS OF REMEDIAL MEASURES AGAINST LIQUEFACTION AT 1993 KUSHIRO-OKI EARTHQUAKE

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ABSTRACT

During the 1993 Kushiro-Oki Earthquake of magnitude 7.8, Kushiro Port being located at 15 km from the epicenter was shaken with a peak horizontal acceleration of 0.47g. Many of the quay walls at the Kushiro Port suffered damage due to liquefaction of backfill sand. However, the quay walls having backfill sand compacted as measures against liquefaction survived the earthquake without damage. In these quay walls, gravel drains were also used as a part of the liquefaction remediation in the vicinity of the walls. Since the contrast in the performance of those quay walls are of significance in the engineering practice, the present paper is devoted to report the details including the recorded earthquake motions and the ground conditions.

INTRODUCTION

In recent Japanese practice, measures against liquefaction have often been put into practice in order to mitigate damage to structures sitting on potentially liquefiable foundations. While laboratory data are abundant to evaluate the effectiveness of those measures against liquefaction, in-situ data during strong earthquake shaking are very scarce.

On January 15, 1993, an earthquake of magnitude 7.8 in the scale of Richter-Japan Meteorological Agency (JMA) hit a northern island of Japan called Hokkaido. The epicenter was located 15 km south of Kushiro City as shown in Fig. 1; the earthquake was named the 1993 Kushiro-Oki earthquake. The focal depth was 107 km, being much deeper than the ordinary focal depths of about 40 km for the earthquakes occurring around Japan.

Kushiro Port was shaken with a peak horizontal acceleration of 0.47g. The earthquake caused liquefaction of untreated backfill sand at the port, resulting in damage to many of the quay walls. The extent of the damage was generally moderate; the displacements of the quay walls were less than about 5 percent of the water depth in front of the quay walls. In contrast, the quay walls having the backfill sand compacted as measures against liquefaction survived the earthquake without damage. In these quay walls, gravel drains were also used as a part of the liquefaction remediation in the vicinity of the walls to relieve the influence of compaction to the walls during its installation.

In the present paper, the effects of these liquefaction remediation measures will be reported in detail including the recorded earthquake motions, the ground conditions, and the results of in-situ examination of gravel drains after the earthquake.

KUSHIRO PORT

Kushiro Port is developed at the estuary of the Kushiro River as shown in Fig. 2. As understood by the former beach line shown by the broken line in Fig. 2, most of the port area has been reclaimed from the sea by filling the sand dredged from the nearby sea. The port has been constructed step by step from a quay wall located at the eastern most end toward west direction along the former beach line; the quay wall at the western most end, i.e. Pier No.3 in the West Port District, has been just constructed in 1992.

The natural ground along the former beach line is of sand dune origin, being dense with SPT N-values of about 50. The fill above the original ground gradually increases its thickness from none at the former beach line to about ten meters at the southern end of the piers in the West Port District.

Though Kushiro City is located at the latitude 43 degrees north, at about the same as that of Rome in Italy, the average temperature falls below the water freezing point during winter. When the earthquake hit, the ground surface was said to be frozen with a thickness of about 0.5 to 1.0 meters in Kushiro City. However, the port engineers reported that the materials used for the ground surface layer in the port areas were chosen to serve as anti-freezing layer to prevent the damage to the pavement due to the soil freezing. The anti-freezing layer is made of coarse sand, its thickness about 0.8 meters. At present, the authors could not obtain the definite evidence to determine whether the ground surface in the port area was frozen at the time of the earthquake. When the earthquake occurred, the ground at Kushiro City was not covered with snow. It began to snow the next day and the ground was covered with snow with a thickness of about 0.2 meters.

EARTHQUAKE MOTION AT KUSHIRO PORT

Kushiro Port has a strong motion recording station at its central location. This is one of the station deployed under the network of the Port and Harbour Research Institute in cooperation with the Kushiro Port Construction Office, Hokkaido Development Bureau (Kurata and Iai, 1992).

The strong motion recording station is located on the former beach line at the estuary of the Kushiro River as shown in Fig. 2. The ground condition is shown in Fig. 3. In this figure and the figures to follow, all the borings and the Standard Penetration Tests (SPT) were conducted before the earthquake. The ground at this site consists of a fill with a thickness of about three meters, underlain by a medium to dense sand layer of SPT N-values about 20 with a thickness of about seven meters, underlain by a dense sand layer of SPT N-values of about 50.

The instrumentation at this site consists of two accelerographs; one is installed at the ground surface, the other installed at the base of the bore hole 77 meters below the ground surface. The accelerographs are of force balance servo type, having the flat gain in the frequency range from 0.01 to 30 Hz. The acceleration data are recorded in the digital format in the integrated circuit (IC) cartridge memory with 16 bits/data at the time intervals of 0.01 seconds. The digital recorder is equipped with a pre-event memory to record earthquake motions from ten seconds before triggering.

The recorded earthquake motions during the 1993 Kushiro-Oki Earthquake are shown in Fig. 4. In this figure, main portion of the earthquake motion is plotted out of the original record obtained for a duration of 180 seconds; the origin of the time is ten seconds before the triggering of the recorder. Loci of integrated velocities and displacements computed for the accelerations at the ground surface and the base, shown in Fig.

5, clearly indicate that the predominant motion is in the north-south direction in this earthquake.

By comparing the ground surface motion in the north direction shown in the first row in Fig. 4 with the corresponding base motion at a depth of 77 meters shown in the fourth row, it is easy to notice the sudden change in the response of ground after about 30 seconds; most of the high frequency motions are filtered out; instead, a long period motion with a period of about 1.5 seconds becomes predominant, overlain by a spiky wave form at each peak.

This type of ground response has been often observed in the laboratory (e.g. Lee and Schofield, 1988); it is considered as manifestation of the effect of cyclic mobility of dense sand. The effect of the dilatancy of sand plays a major role when the ground is saturated and shaken with a strong motion as in this earthquake. In the in-situ condition during earthquakes, similar spiky acceleration was observed at loose silty sand deposit at the Wildlife site during the 1987 Superstition Hills Earthquake (Holzer et al, 1989). In the dense clean sand deposit, the present record represents the very evidence that the effect of dilatancy of sand plays a significant role in the ground response during strong earthquake shaking.

In the current practice of ground response analysis such as often done with the equivalent linear model, the effect of the dilatancy of sand has been often neglected. The definite evidence presented here cautions us against the current practice of ground response analysis at strong earthquake motions.

DAMAGE TO GRAVITY QUAY WALLS

The most serious damage to the gravity quay walls in Kushiro Port was found at the North Pier in the East Port District. The location is designated in Fig. 2 as the site A. As shown in Fig. 6, displacement toward sea registered 2.0 meters at maximum, settlement 0.4 meters at maximum. The ground conditions shown in Fig. 7 indicates possibility of liquefaction down to an elevation of about -6.0 meters below the water level.

Among the gravity quay walls affected by liquefaction of backfill sand, quay walls designed with a seismic coefficient of 0.2 suffered relatively minor displacement. An example with a water depth of 12 meters is shown in Fig. 8. This quay wall is located at the south end of the Pier No.2 in the West Port District shown in Fig. 2 designated as the site B. As understood from the height to width ratio of the caisson being close to unity, the quay wall was designed to resist relatively strong earthquake motions.

The backfill sand at this pier was loosely deposited and was of a coarse sand as shown in Fig. 9. It was planned to be compacted as remedial measures against liquefaction; unfortunately the earthquake hit the Kushiro area before the work was actually done. The backfill sand liquefied due to the earthquake. The caisson suffered deformation as shown in Fig. 8 but the displacement toward sea was only about 0.3 meters.

DAMAGE TO A SHEET PILE QUAY WALL

The most serious damage to the sheet pile quay walls in Kushiro Port was found at the Fishery Pier in the East Port District. The location is designated in Fig. 2 as the site C. The structure of this quay wall is of a steel sheet pile type anchored by battered steel piles. The cross section is shown in Fig. 10. As shown in Fig. 11, the ground at this pier consists of a loosely deposited backfill sand with a thickness of about ten meters, underlain by medium to dense sand deposit which forms the original ground.

The original ground level becomes deeper to the level of 11.5 meters at the eastern end of the quay wall so that this part of the quay wall slightly differs from that shown in Fig. 10; the quay wall at the eastern end uses longer sheet piles with an embedment depth of -20.5 meters and longer anchor piles installed to a depth of -14.9 meters on the sea side and -20.6 meters on the land side.

The earthquake caused liquefaction in the backfill sand, resulting in the serious deformation of the sheet pile wall as shown by the solid line in Fig. 10. In accordance with the deformation of the sheet pile wall, the apron exhibited serious settlement between the sheet pile wall and the anchor piles. Detailed investigation of the sheet pile wall by diving in the sea revealed that cracks opened in the steel sheet pile wall at an elevation of four meters below the water level.

PERFORMANCE OF A SHEET PILE QUAY WALL WITH COMPACTED Backfill SAND

In contrast to the damaged sheet pile quay wall described earlier, quay walls with compacted backfill sand survived the earthquake without damage. An example, shown in Fig. 12, is the quay wall at the south end of the Pier No.1 at the West Port District. The location is designated as the site D in Fig. 2.

The structure of this quay wall is of a steel pipe pile wall anchored by a steel sheet pile wall with a water depth of 12 meters. The ground at this wall had originally consisted of fill sand with a thickness of about ten meters, underlain by the original ground of medium to dense sand. The backfill sand was later treated by a sand compaction pile method as shown in Fig. 12. The SPT N-values of the ground before and after the compaction are shown in Fig. 13. In order to avoid the adverse effect upon the existing steel pipe pile wall during the ground compaction work, gravel drains were installed near the wall as shown in Fig. 12.

Despite the strong shaking of the earthquake, the earthquake caused no effect on this quay wall. The measures against liquefaction as well as the overall design of the quay wall according to the current design practice (Ports and Harbours Bureau, 1991) was proven to be good enough to provide resistance the strong earthquake shaking.

LIQUEFACTION AND DAMAGE TO STRUCTURES

In order to grasp an overall picture of the effects of liquefaction on damage to quay walls at Kushiro Port, Table 1 summarizes the relevant data of the quay walls with water depths deeper than 7.5 meters. In this table, "liquefaction" is identified with respect to the soils within "the area of a quay wall cross section," shown in Fig. 14, which includes the backfill area treated as measures against liquefaction. This similarly applies to the quay walls without measures against liquefaction, including the backfill area which is supposed to be treated as remediation against liquefaction.

As shown in Table 1, the quay walls designed with a seismic coefficient of 0.15 suffered serious sea-ward displacement of more than 20% of the water depth. Both the excessive strong shaking and the liquefaction might be the cause for this. In contrast, the quay walls designed with a seismic coefficient of 0.20 suffered sea-ward displacement of less than 10% even when liquefaction occurred.

A closer look at the displacements of the quay walls with a seismic coefficient of 0.20 reveals that those without measures against liquefaction suffered displacement ranging from 0.5 to 10% whereas those with measures against liquefaction suffered displacement less than 3%, suggesting the remediation measures against liquefaction reduces displacement of quay walls.

Among these features observed above, the effectiveness of measures against liquefaction to reduce displacement of quay walls is what was expected through the laboratory studies; the in-situ evidence observed here is the very confirmation of this fact. The effects of seismic coefficient on displacement of quay walls are also what was expected from the balance of inertial forces and resistant force of quay walls. However, the effects of seismic coefficient also include the resistance against the effects of liquefaction. The larger resistance of gravity body, which becomes massive and large if designed with seismic coefficient of 0.20, may contribute to resist the increase in earth pressures due to liquefaction as well.

From the point of view of measures against liquefaction, the backfilled part of ground consisting of rubble behind a wall shown in Fig. 14 might be regarded as treated area as measures against liquefaction. Though the area of this part is not wide enough to cover whole area of active earth failure zone, this part still contributes to reduce the effects of liquefaction. Indeed, past case histories of performance of quay walls during earthquakes suggests that the effects of rubble can be regarded to reduce liquefaction induced displacement of quay walls (Tsuchida, 1981).

PERFORMANCE OF GRAVEL DRAINS

There was some concern about the performance of gravel drains during earthquakes. This was partly because the history of gravel drains as liquefaction remediation is rather new; the case history data on its performance during actual earthquakes has been scarce. After the earthquake, a site designated site-E, shown in Fig. 2, where gravel drain was installed as remedial measures against liquefaction, was excavated in order to examine the clogging of gravel drains due to migration of sand from natural ground. The cross section of the quay wall and the remediation measures are similar to that shown in Fig. 14 (a).

The gravel drain piles are 40 cm in diameter, installed at a space of 5.0 meters. The sand consisting of the ground and the gravels used for the drain have those gradations shown in Fig. 15. The 15% diameter of the filtering material (i.e. the gravel) d_{F15} and the 85% diameter of the sand D_{S85} read off from this figure are

$$d_{F15} = 2.50 \text{ mm} \quad (1)$$
$$D_{S85} = 0.90 \text{ mm} \quad (2)$$

resulting in a diameter ratio as

$$d_{F15}/D_{S85} = 2.8 \quad (3)$$

This ratio should be more than adequate for passing such a filtering criterion as

$$d_{F15}/D_{S85} < 5 \quad (4)$$

as suggested by the Design Standard for Dams in Japan (1978) and Sherard et al.(1984).

Excavation was done by at first lowering the existing water level at +1.20m to a level of +0.00 m, then by digging out to a level of +0.00 as shown in Figs. 16 and 17. Close eye inspection of a gravel drain, shown in Fig. 18, found no migration of sand.

Sieving of the gravels sampled from the gravel drain resulted in Fig. 19, suggesting that there is not variation in the percentage of sandy materials (i.e. those with a particle size less than 2.0 mm) within the cross section of the gravel drain. However, there were about 20 percent of the sandy materials contained in the gravels as shown in the same figure. In order to examine the origin of those sandy materials, mineral analysis using X-ray diffraction method was conducted on the sandy materials in the gravels, to be compared with those results on the sand consisting of the ground and the gravel. The mineral analysis resulted in that tridymite S_iO_2 is commonly found in the sandy materials in the gravel drain and the gravel but not in the sand consisting of the ground. This result also confirmed the fact that no migration of sand into the gravel drain occurred during the earthquake.

CONCLUSIONS

The present paper was devoted to compile the case history data on the effects of remediation measures against liquefaction during the 1993 Kushiro-Oki Earthquake. The further analyses are currently underway and will be reported elsewhere. Implications from the present case history data can be summarized as follows.

(1) The strong motion record with a peak acceleration of 0.47g at Kushiro Port indicates that the effect of dilatancy of sand plays an important role in the ground response during strong shaking.

(2) Performance of the quay walls at Kushiro Port manifests the importance and the effectiveness of the measures against soil liquefaction in the earthquake resistant design of waterfront structures.

(3) No migration of sand into gravel drains occurred during the earthquake, suggesting sound performance of gravel drains.

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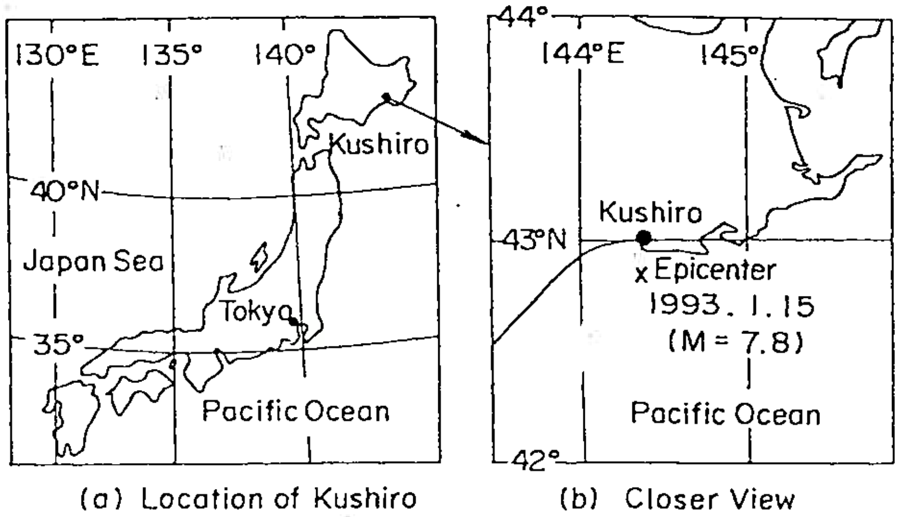


Fig. 1 Location of Kushiro City and the epicenter

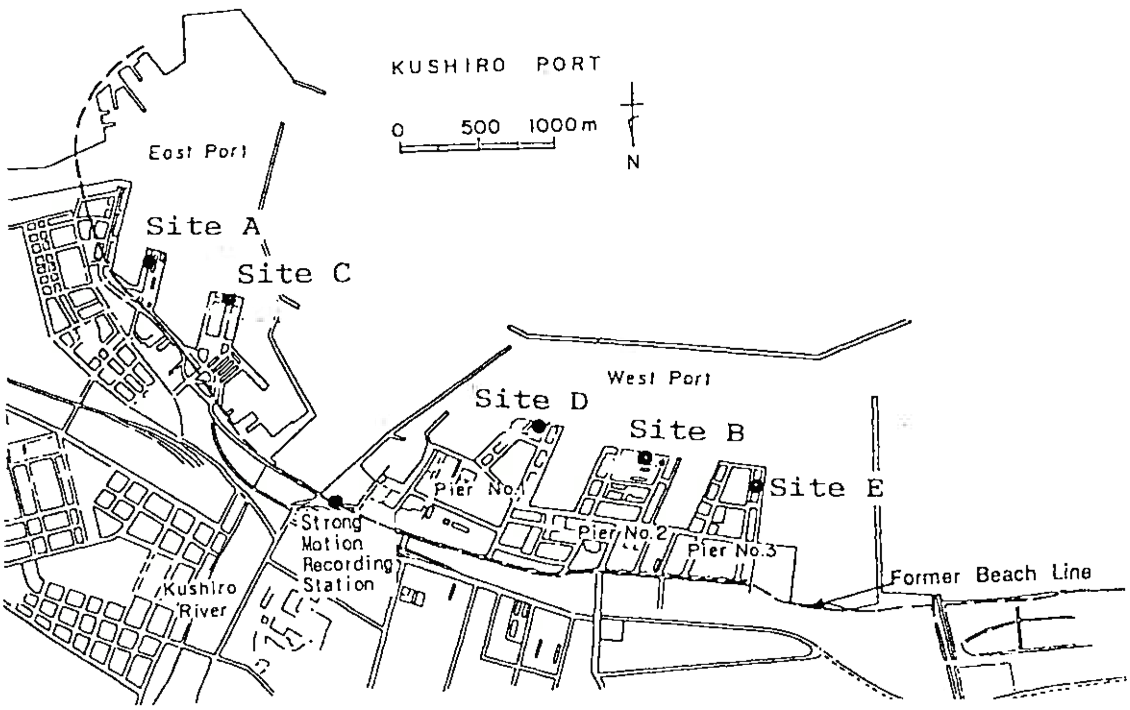


Fig. 2 Plan of Kushiro Port

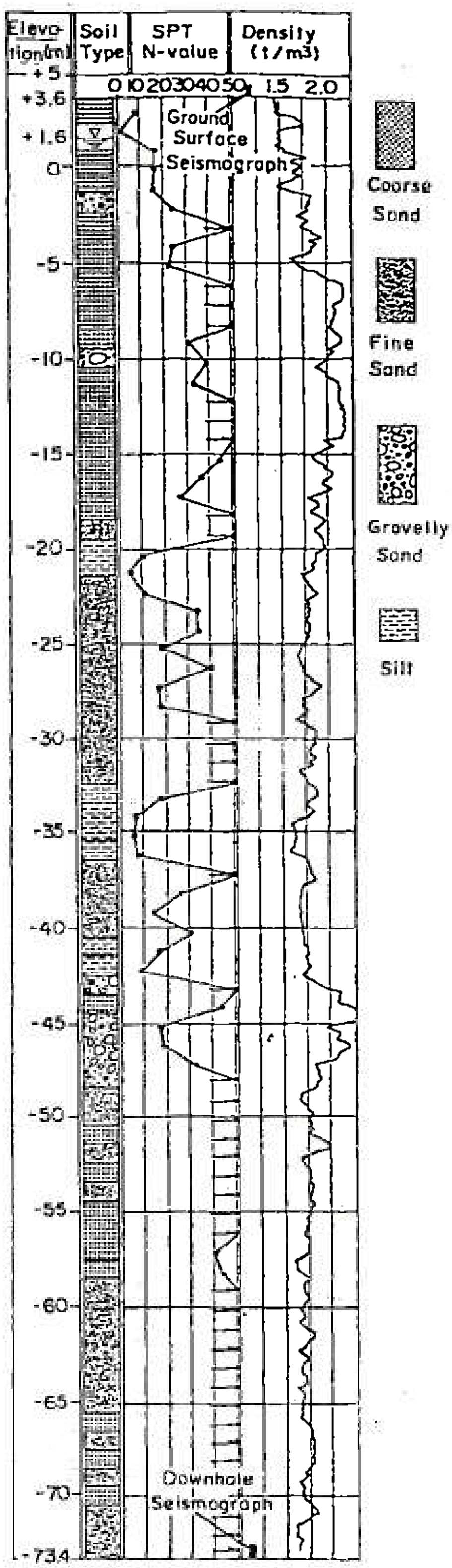


Fig. 3 Ground condition at the strong motion recording station

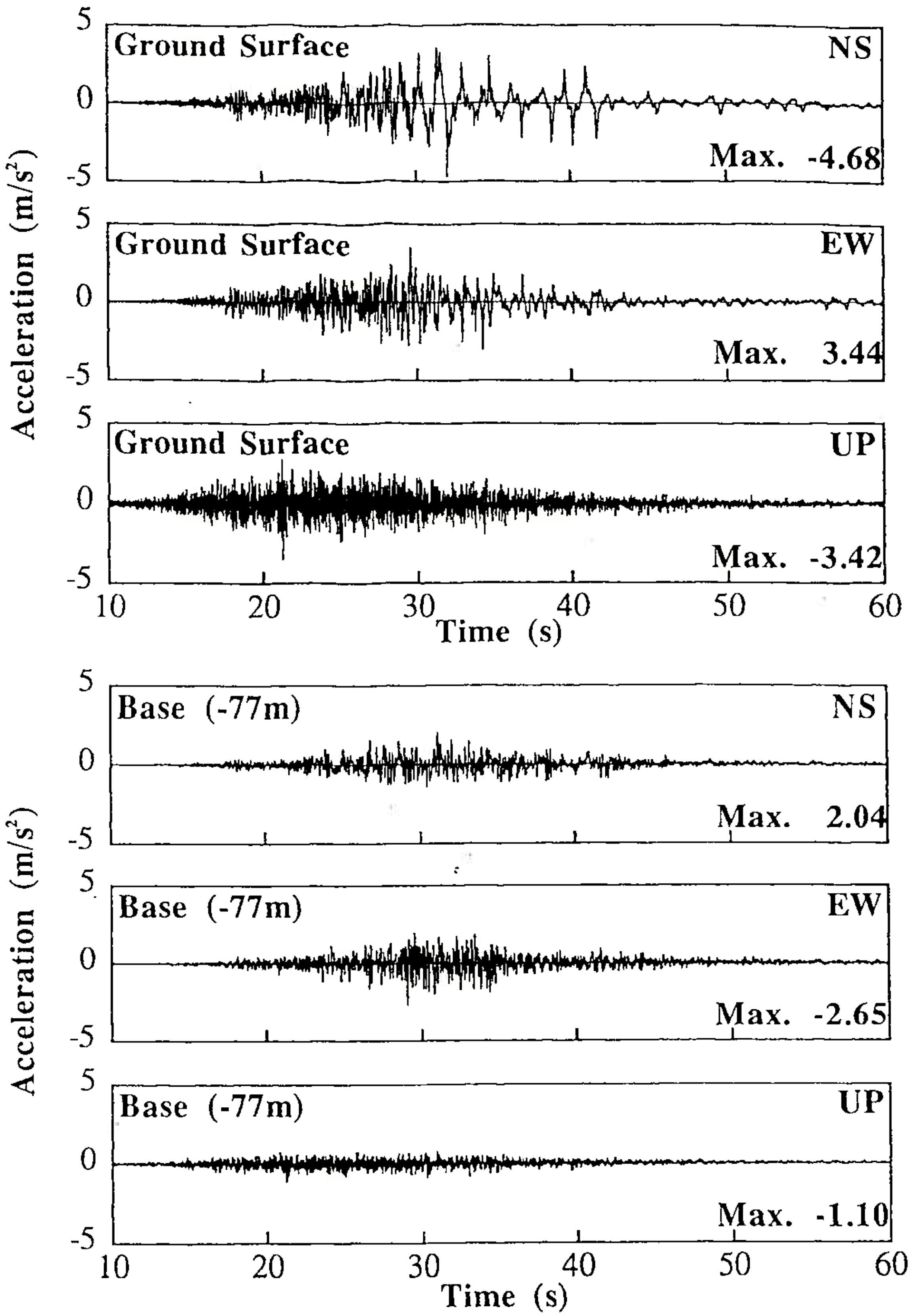


Fig. 4 Strong earthquake motions at Kushiro Port on Jan. 15, 1993

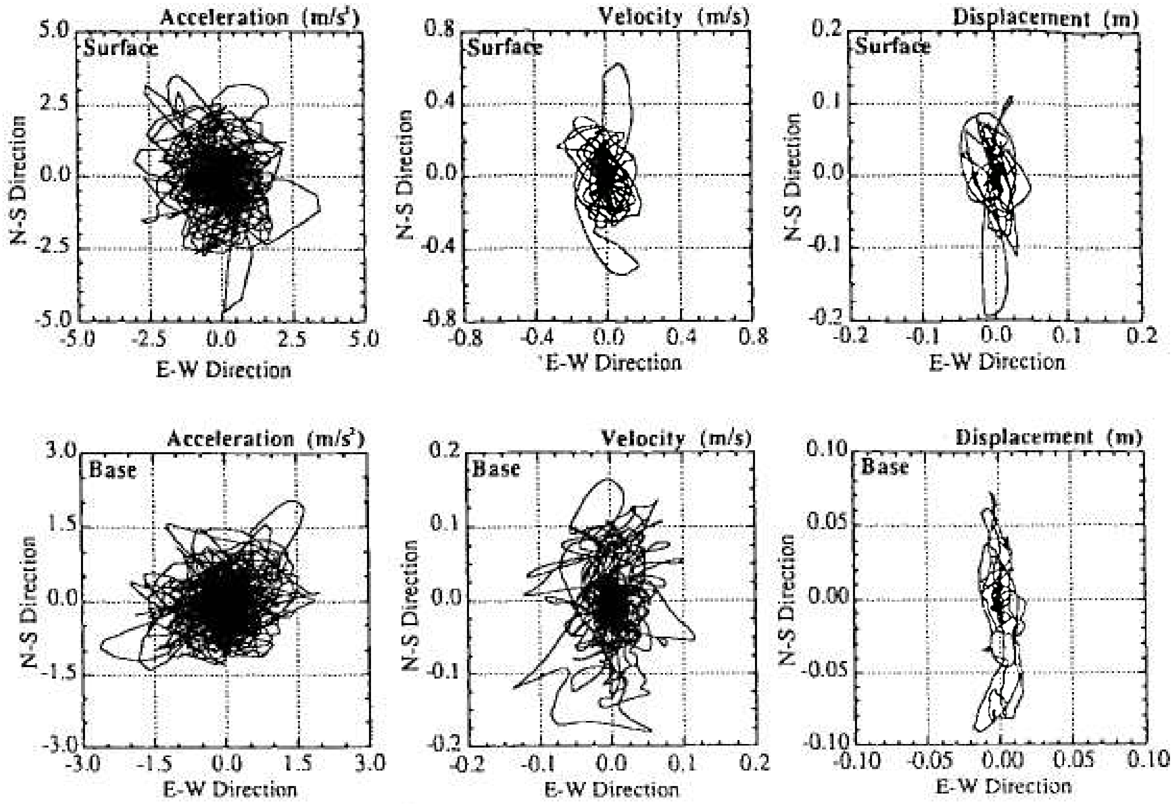


Fig. 5 Loci of accelerations, velocities and displacements

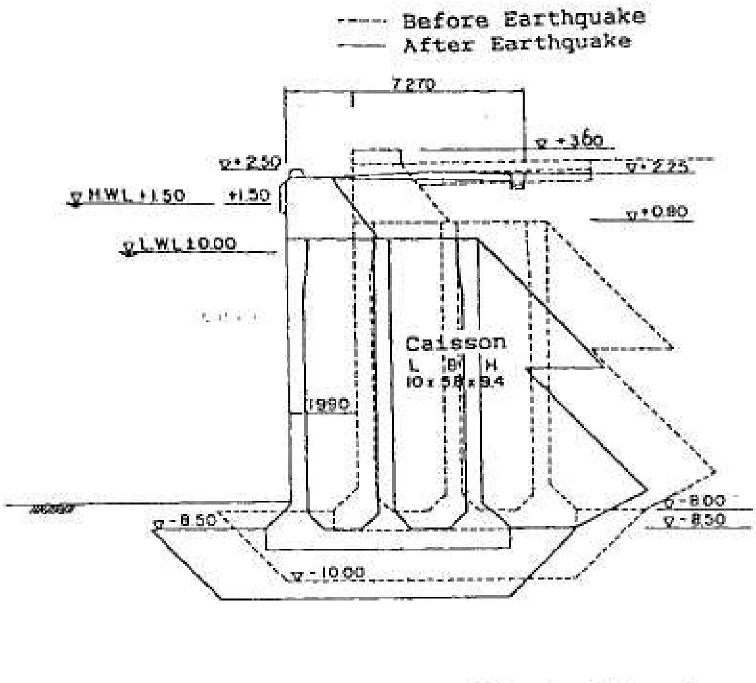


Fig.6 Damage to a quay wall at Site A

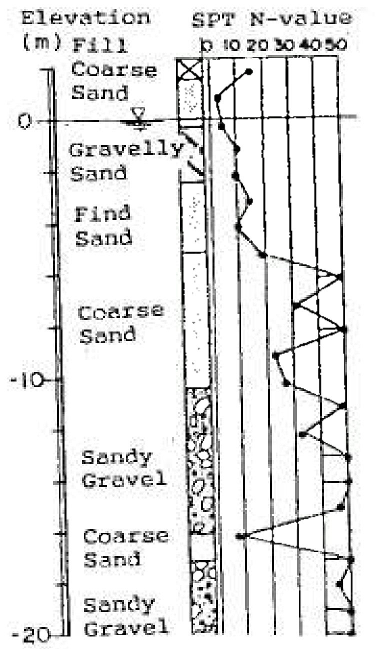


Fig.7 Ground Condition at Site A

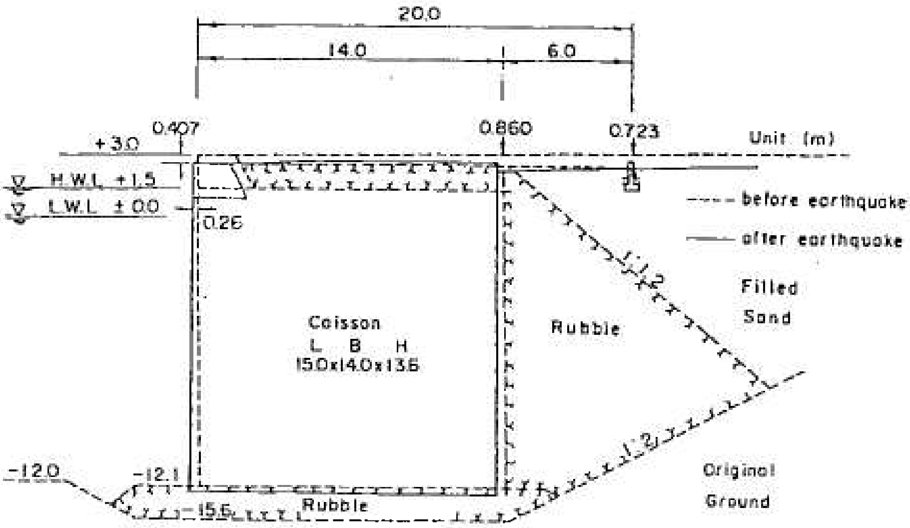


Fig.8 Damage to a quay wall at Site B

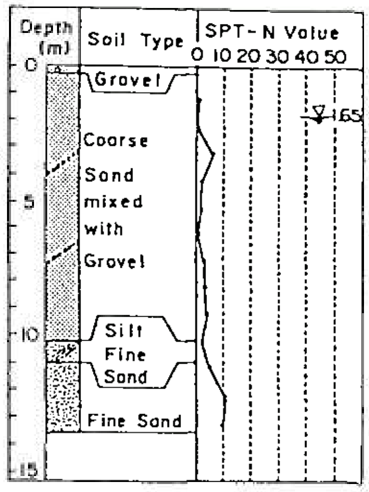


Fig.9 Ground Condition at Site B

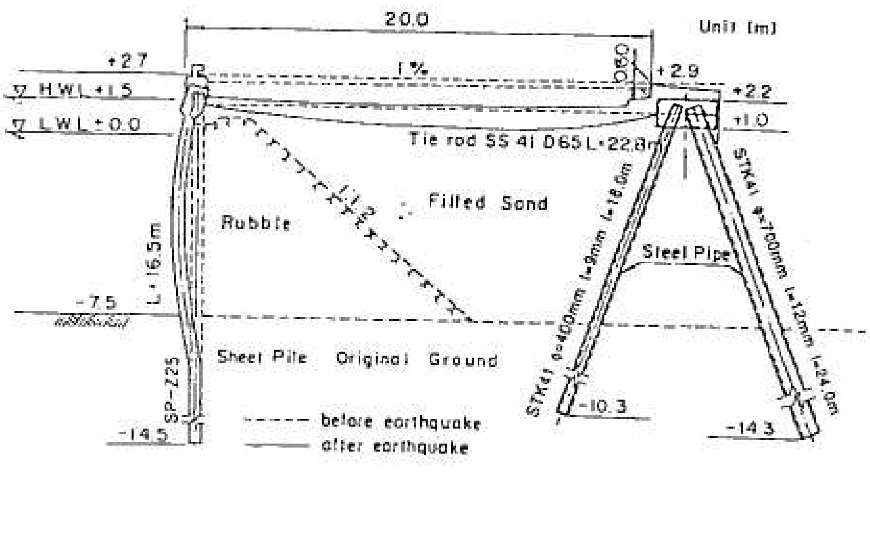


Fig.10 Damage to a quay wall at Site C

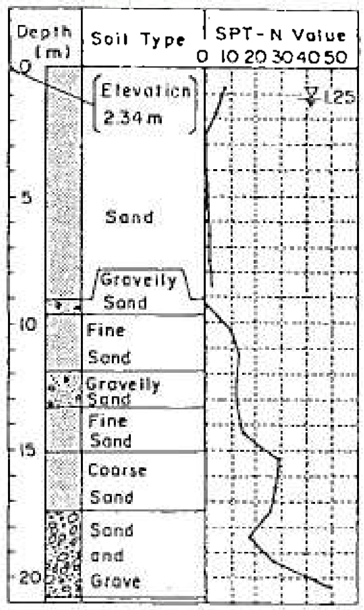


Fig.11 Ground condition at Site C

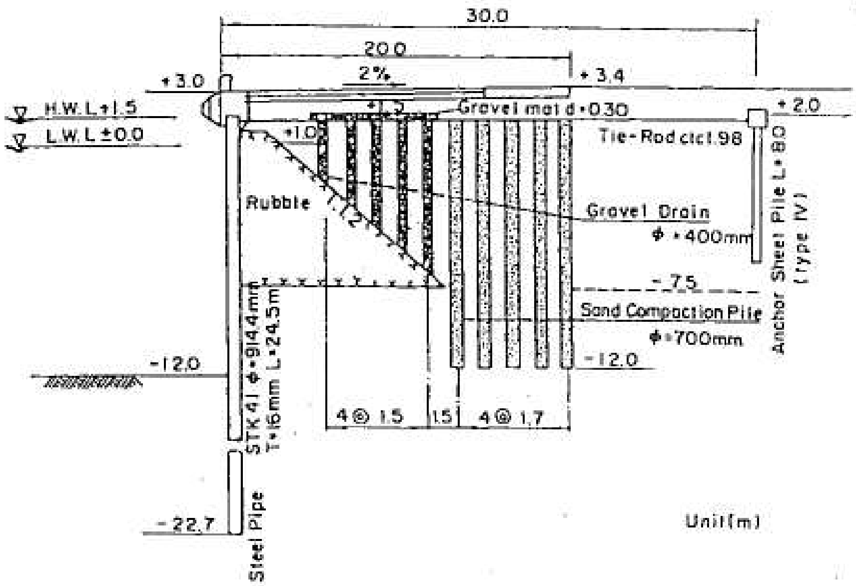


Fig.12 Cross section of a quay wall at Site D

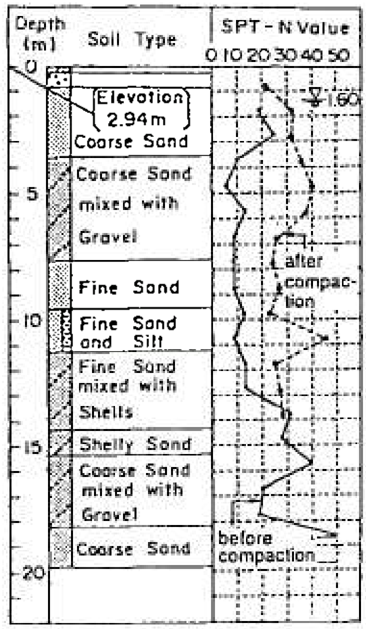
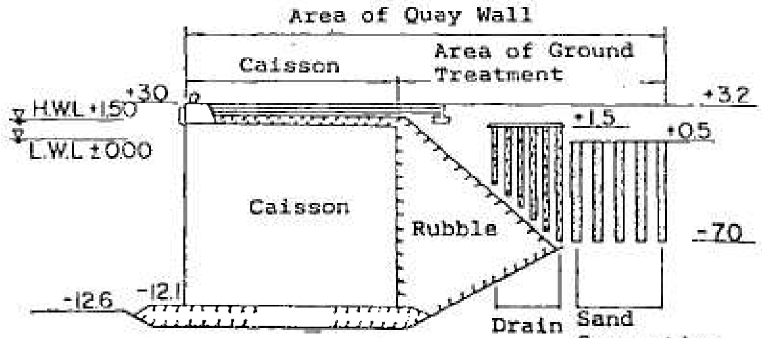
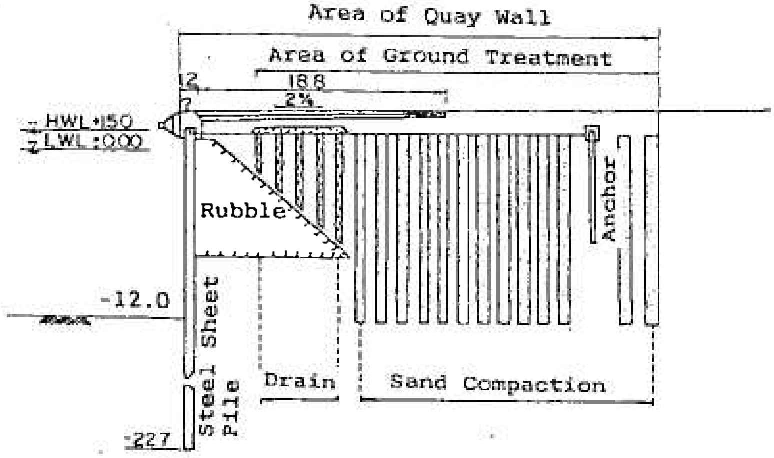


Fig.13 Ground condition at Site D



(a) Gravity type quay wall



(b) Sheet pile quay wall

Fig.14 Schematic figures of the area of quay walls

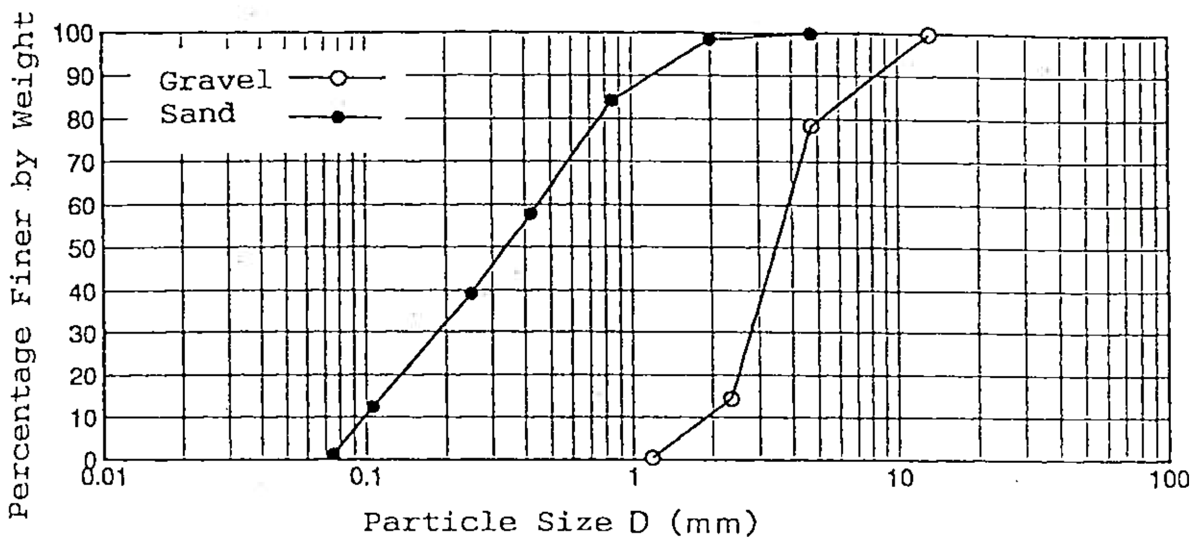


Fig.15 Gradation of sand and gravel

Fig.16 General view of excavation of gravel drain site

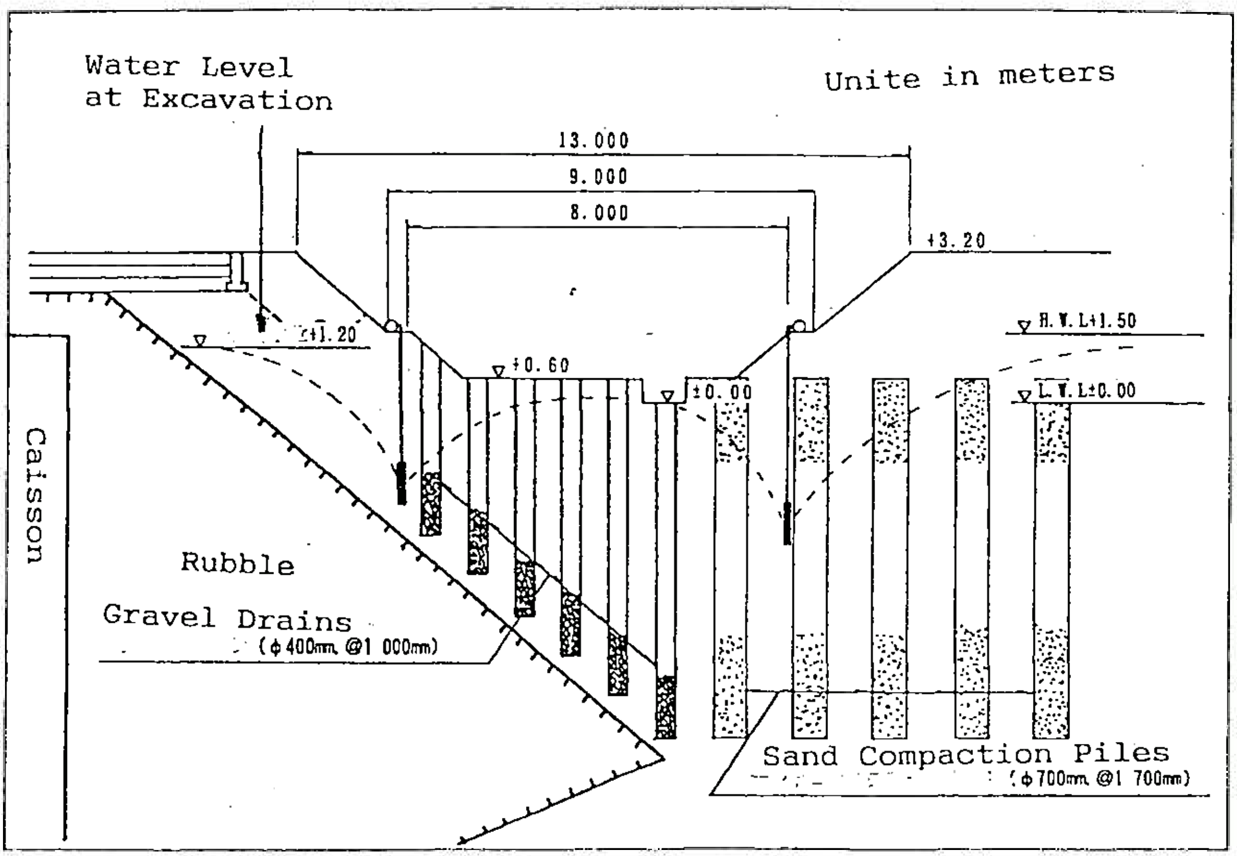
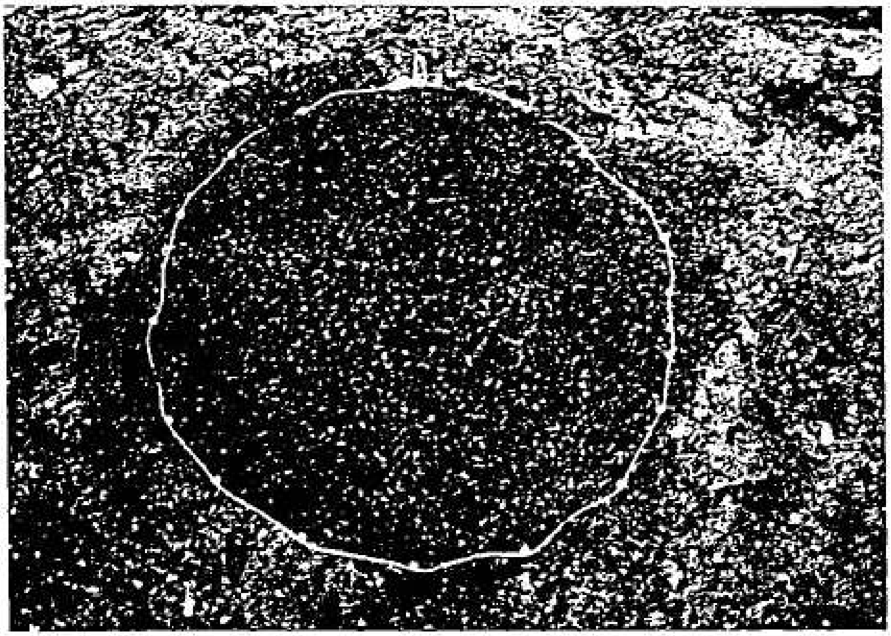
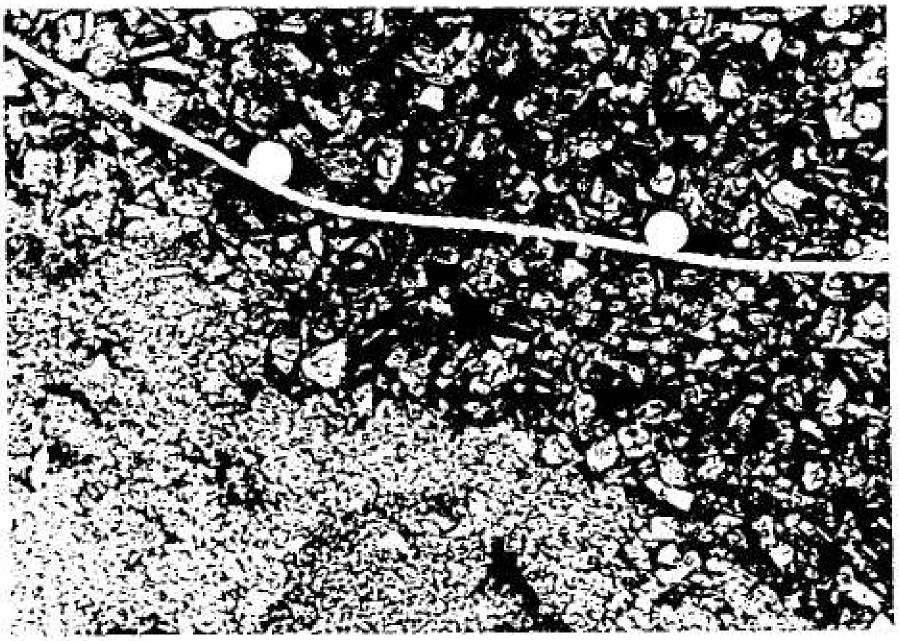


Fig.17 Cross section of excavation details

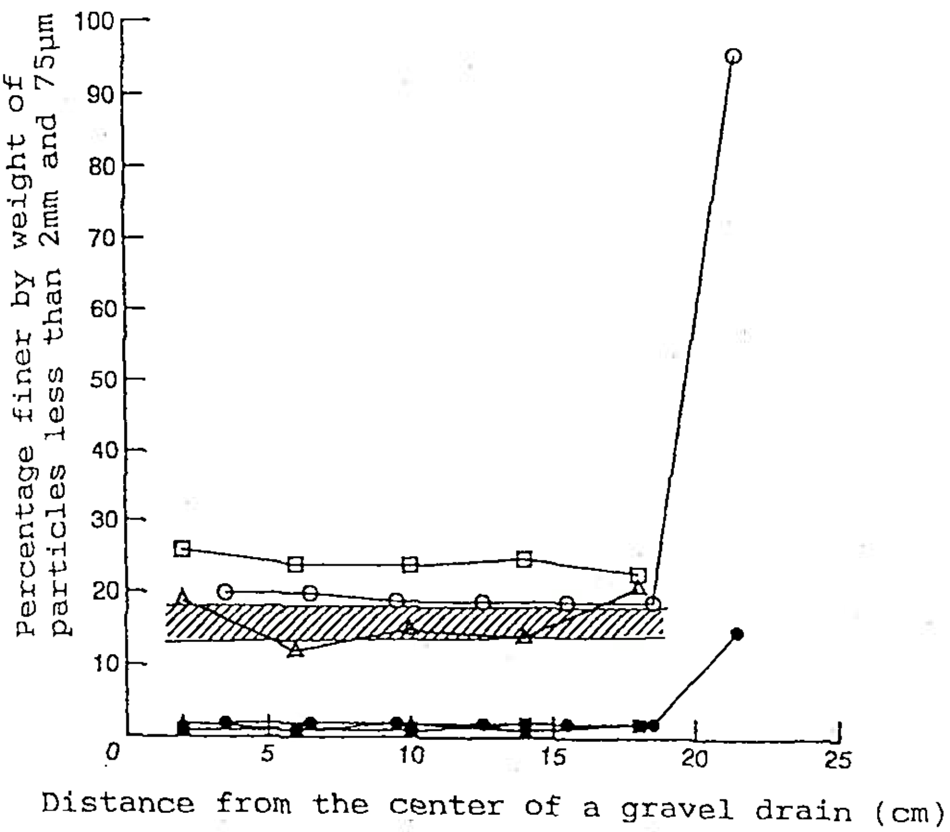


(a) Cross section of gravel drain and surrounding sand



(b) Closer up view

Fig.18 Excavated cross section of gravel drain



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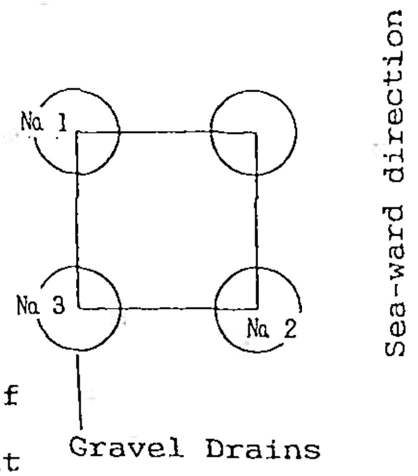
○	No. 1
●	(+ 0.6m)
□	No. 2
■	(+ 0.6m)
△	No. 3
▲	(± 0.0m)



※ ////Percentage finer by weight of particles less than 2.36mm of gravels at the installment of the gravel drain

Open Circle : Sandy materials (less than 2 mm)
Closed Circle : Silty materials (less than 75μm)

Fig.19 Fine grained materials and sand contained in the gravels