

Evaluation of soundness and seismic behavior
of long-term-use irrigation dams

Yoichi HAYASHIDA

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Chapter 1 Introduction

Historically speaking, earthquakes have caused significant damage to few dams. On a worldwide basis, only several dams are known to have failed completely as a result of an earthquake. Hence, if one considers the total number of existing large dams around the world, the current performance record appears to be outstanding, based on the limited number of failures that have occurred. From this fact, it is confirmed that modern design and construction methods can guarantee the seismic performance of dams.

Many dams in Japan have been used for long periods of time. In 2010, it was reported that 50% of the dams constructed after 1900 were still in good working condition over 50 years after their construction, and this number is expected to rise to 58% by 2020 (Yamaguchi, 2011). Thus, it is important to manage the safety of these dams and to maintain their healthy condition. In Japan, huge earthquakes have been occurring frequently since the 1995 Hyogoken-Nanbu Earthquake; they include the 2004 Niigataken-Chuetsu Earthquake, the 2008 Iwate-Miyagi Nairiku Earthquake, the 2011 off the Pacific coast of Tohoku Earthquake, and the 2016 Kumamoto Earthquake.

Under such circumstances, social awareness of earthquakes has increased; and thus, checks of the seismic performance of existing dams are being carried out nowadays to evaluate the safety of the dams and to assess their stability against large earthquakes (Hayashida, 2015; MAFF, 2016).

In this thesis, the mechanisms of the possible failures of fill dams induced by earthquakes are revealed, and a countermeasure to moderate these harmful effects on the dams is proposed. Moreover, a new measuring system for evaluating the health of dams is developed and improved, and its performance had been verified for some real dams over the long term.

Chapter 2 reviews the seismic damage to dams during past massive earthquakes and summarizes their features. Then, reviews of both the seismic design of fill dams and of studies on the possible failures of fill dams due to earthquakes are given.

Chapter 3 refers to the new measuring system without cables. This system can improve the efficiency of dam construction and the stability of pore water pressure measurements for the safety evaluation of fill dams. This system is called the “wireless pore water pressure transducer (WPT)”. Test models of WPTs have been verified in some real dams over the past several years, but they became inoperative without fulfilling their specified and expected lifetime of ten years. The reasons why they stopped functioning are examined through capacity tests on their batteries and a look at the voltage data of the batteries measured on site.

Chapter 4 examines the manner in which the modeled region of the bedrock and its material properties affect the response behavior of the dam body by a two-dimensional response analysis, which is one way to check the seismic performance of dams. In particular, the amplitude ratio and the predominant frequency are investigated. In this chapter, a concrete gravity dam is targeted for a

clear interpretation of the results, because its dam body and the bedrock are generally modeled as linear elastic materials.

Chapter 5 aims to acquire fundamental scientific knowledge on the deformation and the fracture behavior of fill dams, especially in view of the liquefaction of their foundations, which is one of the principal factors in the large deformations of dams during earthquakes. Centrifugal liquefaction experiments are carried out to examine the fracturing mechanism of dams developing through the interaction among the liquefiable foundation, the dam body, and the reserved water, and a countermeasure to modify the fracturing development is proposed. Moreover, the manner in which the deformation behavior of the dam body is influenced by the existence and the position of an impervious zone (core trench) in the liquefiable foundation is examined by numerical experiments.

Chapter 6 examines the characteristics of the seismic behavior of fill dams by shaking table tests to verify the effects of the dam shape and the direction of the input wave on the seismic response behavior. It is supposed that the incidents of damage, such as settlement and longitudinal and transverse cracks at the crest, were caused by the complicated three-dimensional seismic behavior of the dam body, and that such behavior will be an important factor for managing the safety of existing dams. Then, three dam models with different shapes are shaken separately in the stream and dam-axis directions.

In conclusion, Chapter 7 summarizes the seismic behavior of dams revealed from this study, and discusses the future development of the evaluation method introduced in this thesis to examine the soundness and seismic behavior of irrigation dams to be used over the long term.

Chapter 2 Seismic damage and seismic performance of dams

2.1 Review of seismic damage to dams during past massive earthquakes

2.1.1 Earthquakes: $5.0 \leq \text{Magnitude} < 7.0$

(1) The 1925 Santa Barbara Earthquake, Magnitude: 6.3

This earthquake caused the catastrophic slide and failure of Sheffield Dam (an 8-m-high earth-fill dam). At the first series of shocks, the downstream portion of the central part of the dam moved out as though it were hinged at the left abutment. A great mass at the center, about 90 m in length, moved 30 m downstream and much of the material was washed away by the escaping water. During its movement, the central part of the dam remained practically intact (Ambrassys, 1960; Seed et al., 1969; Seed et al., 1978).

(2) The 1930 Santa Monica Bay Earthquake, Magnitude: 5.3

Chatsworth Dam (a 13-m-high hydraulic-fill dam) suffered moderate damage due to this earthquake. The fill that was dumped onto the upstream face of the dam slid into the reservoir, and a crack running over the entire length of the crest was opened. Smaller cracks on both upstream and downstream faces of the dam also opened up (Ambrassys, 1960).

(3) The 1939 Ojika Earthquake, Magnitude: 6.6

A substantial number of low-embankment dams, forming reservoirs for irrigation purposes, were damaged during this earthquake. Akiba and Semba (1941) concluded that the majority of the damaged and failed embankments consisted of sandy soils, and that no complete failure occurred in the embankments constructed of clayey soils.

(4) The 1954 Fallon Earthquakes, Magnitudes: 6.6 and 6.8

Two earthquakes with magnitudes of about 6.7 occurred on July 6 and August 23, 1954. Coleman Dam, a composite concrete-earth diversion structure, failed because of the displacement and cracking of the earth-fill abutments, which permitted water to erode around the concrete portion, causing it to partially overturn, crack, and settle. The earth abutments were completely washed out. Rogers Dam was composed of a central concrete spillway with two earth-fill embankments. This dam failed during the earthquake of August 23. The southeast part of the dam was composed of earth fill. It gave way, and a portion of the concrete structure broke off and turned around into the stilling basin (Ambrassys, 1960).

(5) The 1955 San Francisco Bay East Earthquake, Magnitude: 5.4

After this earthquake, it was found that a 90-mm-wide crack had developed along the upstream face of Saint Mary's Dam (a 15-m-high masonry dam). Subsequently, pits were excavated along the crack and it was seen that the crack was directly above the concrete core wall which was 1.2 m below the crest. It was thought that the rigidity of the concrete core wall was directly responsible for the crack (Ambrassys, 1960).

(6) The 1967 Koyna Earthquake, Magnitude: 6.5

Koyna Dam (a 103-m-high straight gravity dam) was shaken by this earthquake in India. This dam had an unconventional cross section based on the principle of maintaining the dam stability and preventing it from overturning, the so-called "middle-third condition". In the upstream and downstream of its non-overflow section, 18 and seven remarkable horizontal cracks developed, respectively (Chopra and Chakrabarti, 1972; USSD, 2003; JDEC, 2005).

(7) The 1971 San Fernando Earthquake, Magnitude: 6.5

A large number of earth dams were shaken by this earthquake. Within a 40-km radius of the epicentral, some 44 dams, ranging in height from 9 to 58 m, were subjected to peak accelerations in the order of 0.2 to 0.7 G. Of the 44 embankments, 13 were non-operational (empty reservoirs) during the earthquake and one was not saturated; therefore, only 30 of the embankments may be considered representative of fully operational dams. Of these operational embankments, 25 consisted of rolled-fill dams of compacted clayey materials, and five were hydraulic sand-fill dams. No damage was observed in any of the 25 rolled-fill embankments, but two of the five hydraulic sand-fill dams (Upper and Lower Van Norman Dams) suffered substantial damage and major slides. In the case of the Lower Van Norman (San Fernando) Dam (a 42-m-high hydraulic-fill dam), the extent of the liquefied zone resulted in a major slide on the upstream side. The overtopping of the crest and flooding were just barely avoided. In the case of the Upper Van Norman (San Fernando) Dam (a 24-m-high hydraulic-fill dam), the result was a slide movement of 1.5 m in the downstream direction and a crest settlement of 0.9 m (Seed et al., 1975a; Lee et al., 1975, Seed et al., 1978). Pacoima Dam (a 113-m-high arch dam) was subjected to estimated maximum horizontal base accelerations of about 0.70 G. This dam neither developed structural cracks nor displayed any conspicuous signs of relative movements between adjacent blocks of the dam (USSD, 2003).

(8) The 1976 Friuli Earthquake, Magnitude: 6.5

Ambiesta Dam (a 59-m-high arch dam) in Italy was shaken during this earthquake, but it suffered no damage (USSD, 2003).

(9) The 1984 Naganoken-Seibu Earthquake, Magnitude: 6.8

Makio Dam (a 105-m-high rock-fill dam) suffered a longitudinal crack, about 1.5 m in depth, on the crest during this earthquake. The crack did not reach the core zone and there was no functional problem with the dam (JDEC, 2005).

(10) The 1994 Northridge Earthquake, Magnitude: 6.7

This earthquake induced ground motions at 105 dams located within a 75-km radius of its epicenter. These dams included most of those shaken in the 1971 San Fernando Earthquake. Eleven earth-fill and rock-fill dams experienced some cracking and slope movements. Yet, none of this damage presented an immediate threat to human lives or property. One of the few embankment dams that did suffer noticeable damage from this earthquake was the Lower Van Norman Dam. This dam has been abandoned as a water storage facility since the 1971 earthquake, but was rebuilt and is still used as an empty reservoir for flood control. It experienced longitudinal cracks, 60 to 105 mm wide and several hundred meters long. Some of the cracks were at least 1.5 m deep. A sinkhole and sand boils were also observed along the upstream face. The maximum crest settlement was 24 mm, and the maximum crest horizontal movement was about 120 mm toward the upstream. The Upper Van Norman Dam was also left with an empty reservoir after being severely damaged in the 1971 earthquake. It experienced transverse cracks near its right abutment on the downstream slope and near its left abutment. Cracks were up to 1.8 m long and 60 to 90 mm wide. Maximum non-recoverable crest displacements induced about 0.72 m of settlement and over 18 mm of horizontal upstream movement. Los Angeles Dam (a 39-m-high earth-fill dam), which replaced the two Van Norman dams, is located in-between these flood-control, dry embankments. Los Angeles Dam showed only minor deformation and superficial cracking. Despite the intense shaking, the crest of the dam moved only 66 mm horizontally and settled only 105 mm at the maximum section. The other eight fill dams suffered minor damage in the form of transverse cracks and settlement. Pacoima Dam (a 113-m-high arch dam) was shaken again during this earthquake. The peak accelerations near the top of the left abutment were 1.76 G horizontal and 1.60 G vertical. The joint between the left abutment and the left end of the arch opened a maximum of 6 mm, due to further movement of the rock wedges on the upper abutment. Minor horizontal cracking of the concrete was seen at the left end of the dam, and several minor horizontal and vertical block offsets occurred at the joints (USSD, 2003).

(11) The 1995 Hyogoken-Nanbu Earthquake, Magnitude: 6.9

Emergency inspections were conducted at a total of 251 dams after this earthquake. No dams were damaged enough to require emergency protective countermeasures. However, some slight effects on the fill dams, such as minor cracks in the pavement on the crest, were reported

(Matsumoto et al., 1996). In Tokiwa Dam (a 33.5-m-high zoned earth-fill dam), transverse cracks developed at the crest pavement near both abutment contacts. Neither the upstream nor the downstream slope of the dam body was damaged. A crack on the left side contact was caused by an existing crack in the base granite, while a crack on the right side contact was caused by a separation of the dam and the rock abutment. The separation occurred because the base rock was cut in steps during the excavation. The crack on the right side extended to the core, but remained confined within the freeboard zone. Minor transverse cracks on the crest were recognized in Taniyama Dam (a 28.2-m-high zoned earth-fill dam) and Otani Dam (an 18.0-m-high zoned earth-fill dam). No other unusual deformations were observed on the upstream or downstream slope of either dam (Hasegawa and Murakami, 1996). In Kitayama No. 1 Dam (a 24.5-m-high earth-fill dam), a shallow sliding of the upstream surface occurred, but the stability of the dam was not affected (Matsumoto et al., 1996). About 1,200 small-scale earth dams for irrigation suffered various forms of damage, such as longitudinal cracking, transverse cracking, settlement, and deformation of the dam body. Of them, 349 suffered serious damage, such as failure of the embankments, sliding, and cracking with a width of more 50 mm. Damage to some of them was caused by liquefaction (Tani, 1996). Niteko reservoir complex suffered serious damage. The upper and the middle dams failed completely, and the lower dam did not fail although it was subjected to significant deformation. It is possible that liquefaction occurred in the dam structures (Towhata et al., 1996).

(12) The 2004 Niigataken-Chuetsu Earthquake, Magnitude: 6.8

Several fill dams were damaged by this earthquake. Asagawara Dam (a 37.0-m-high zoned earth-fill dam), Shin-Yamamoto Dam (a 42.4-m-high zoned earth-fill dam), and Yamamoto Dam (a 27.1-m-high zoned earth-fill dam) suffered remarkable damage. The maximum crest settlement of Asagawara Dam was 200 mm, and several longitudinal cracks with a maximum width of 400 mm developed on the crest. Shin-Yamamoto Dam settled a maximum of 850 mm, and a transverse crack (20 to 30 mm wide) developed at the left side of the crest. Yamamoto Dam settled 100 to 200 mm at the crest, and there were differences parallel to the dam-axis on the upstream slope (Yasuda et al., 2006; JGS, 2007). Kawanishi Dam (a 43-m-high zoned earth-fill dam) settled 31 to 276 mm, and concrete blocks for protecting the slope moved remarkably at the left side of the upstream slope. In Choufukuji Dam (a 27.2-m-high zoned earth-fill dam), Oota Dam (a 26.5-m-high earth-fill dam), and Tsuboyama Dam (a 20.5-m-high zoned earth-fill dam), the differences and the transverse cracks were recognized at the connecting part between the embankment and the spillway (Kohgo, 2006).

2.1.2 Earthquakes: $7.0 \leq \text{Magnitude} < 8.0$

(1) The 1946 Nankai Earthquake, Magnitude: 7.2

Honenike Dam (a 30.4-m-high multiple arch dam) suffered a crack in an arch near a buttress during this earthquake. The damage was not so severe, and the crack was repaired by grouting (USSD, 2003). During this earthquake, approximately 50 fill dams, constructed between 760 and 1944 and with heights ranging from 15 to 40 m, were subjected to ground motions, but no damage was reported (Seed et al., 1978).

(2) The 1952 Kern County Earthquake, Magnitude: 7.7

This earthquake shook dams in Southern California, and some of them experienced moderate damage. The embankments shaken by this earthquake provide a comparison between the behavior of rolled-fill embankments of clayey material and the comparatively poorer performance of hydraulic-fill material. Two hydraulic-fill sandy embankments suffered longitudinal cracking with one developing cracks indicative of a potentially incipient sliding condition. In contrast, three rolled-fill embankments of compacted sandy clays suffered no significant damage (Ambrassys, 1960; Seed et al., 1978).

(3) The 1959 Hebgen Lake Earthquake, Magnitude: 7.1

This earthquake shook the Hebgen Dam (a 27-m-high earth-fill dam). As a result of the shock, the earth fill settled about 1.2 m on the downstream side and 1.85 m on the upstream side of the central concrete core wall at the crest. Several cracks, the largest being about 75 mm wide, were opened in the concrete core wall. The fill on both sides of the core cracked and slumped (Ambrassys, 1960; Seed et al., 1978).

(4) The 1961 Kita-Mino Earthquake, Magnitude: 7.0

Miboro Dam (a 131-m-high rock-fill dam), which had a sloping upstream core, suffered only slight effects during this earthquake, namely, a settlement of about 25 mm and a horizontal displacement of about 50 mm at the crest (Seed et al., 1978).

(5) The 1964 Niigata Earthquake, Magnitude: 7.5

A total of 146 small earth dams were damaged, seven of which burst (Takase et al., 1966).

(6) The 1977 Vrancea Earthquake, Magnitude: 7.2

Poiana Ursului Dam (a 79-m-high buttress dam) in Romania was shaken during this earthquake, but it suffered no damage (USSD, 2003).

(7) The 1968 Tokachi-Oki Earthquake, Magnitude: 7.8

This earthquake caused extensive damage to 93 embankments. Most of them were small earth structures used for irrigation purposes with heights ranging from 4.6 to 18 m. The embankment material consisted mostly of loose volcanic sand characterized by low values of standard penetration resistance constructed on foundation materials of a similar nature (Seed et al., 1978).

(8) The 1978 Miyagiken-Oki Earthquake, Magnitude: 7.4

Numerous embankment dams, mainly small irrigation dams, were shaken by this earthquake. Eight of the dams were damaged, but none of them burst. Most of the damaged dams suffered longitudinal cracking or settling (Tani and Hasegawa, 1987).

(9) The 1983 Nihonkai-Chubu Earthquake, Magnitude: 7.7

Numerous embankment dams, mainly small irrigation dams, were shaken by this earthquake. A total of 238 dams were damaged, ten of which burst. Most of the damaged dams suffered longitudinal cracking or settling (Tani and Hasegawa, 1987).

(10) The 1989 Loma Prieta Earthquake, Magnitude: 7.1

This earthquake affected a wide region of the San Francisco Bay area. Over 100 dams of various sizes, most of them embankment dams, were located within 100 km of the epicenter. All but one of the dams performed well. Austrian Dam (a 60-m-high earth-fill dam) experienced substantial transverse abutment cracking and a maximum crest settlement of nearly 0.9 m. Lower Crystal Springs Dam, shaken by the 1906 San Francisco Earthquake, experienced this earthquake and was again unaffected (USSD, 2003).

(11) The 1990 Manjil Earthquake, Magnitude: 7.5

Sefid Rud Dam (a 104-m-high concrete gravity buttress dam) in Iran was affected by this earthquake. The dam suffered damage, including severe cracking in the upper part of all buttresses and water leaking from the cracks. It represents an example of a concrete dam that was subjected to shaking perhaps more severe than its design loads. This earthquake was probably the equivalent of the Maximum Credible Earthquake (MCE) for this site (USSD, 2003; JDEC, 2005; Wieland and Fallah, 2014).

(12) The 1999 Chi-Chi Earthquake, Magnitude: 7.6

Several embankment dams were shaken significantly during this earthquake. One of them is Shui-Shih Dam (a 30.3-m-high earth-fill dam), which had a central clayey and concrete core wall. Seven longitudinal cracks with a maximum width of 70 mm were observed; three of them were on

the crest, one of them was on the downstream slope, and three of them were on the upstream slope. Liyutan Dam (a 96-m-high rock-fill dam) settled a maximum of 100 mm, and an opening developed at the left end of the dam crest (Ohmachi, 2000). Shih-Kang Dam (a concrete buttress gravity dam) was affected by the fault surface rupture. Over two-thirds of it was uplifted by the thrust block movement of the Chelungpu Fault. Differential displacements were measured as about 7.5 m. Damage was confined to the two bays directly overlying the fault rupture. The 16 other bays essentially remained intact. The failure of the dam was caused by the differential fault movement (Ohmachi, 2000; USSD, 2003).

(13) The 1999 Kocaeli Earthquake, Magnitude: 7.4

Only two major embankment dams were located in the mesoseismal area where this Turkey earthquake occurred. Gokce Dam (a 60-m-high rock-fill dam) suffered a longitudinal crack along the upstream side of the crest, about 9 mm wide. In Kirazdere Dam (a 107-m-high earth-fill dam), a few longitudinal cracks, about 3 mm wide, occurred along the crest gravel road (USSD, 2003).

(14) The 2008 Iwate-Miyagi Nairiku Earthquake, Magnitude: 7.2

Emergency inspections were conducted at a total of 134 dams after this earthquake. A seismograph installed at the crest of Ishibuchi Dam (a 53.0-m-high concrete faced rock-fill dam) recorded a peak acceleration of 1.46 G in the stream direction and 2.07 G in the vertical direction. Ishibuchi Dam suffered the rippling and cracking of the crest pavement and openings at the boundary between the crest pavement and the railings (Yamaguchi et al., 2009). Koromogawa No. 1 Dam (a 35.5-m-high earth-fill dam) was damaged at the upstream shoulder of the crest during this earthquake (Sakamoto et al., 2010). Aratozawa Dam (a 74.4-m-high rock-fill dam) sustained no serious damage, but settled 380 mm at the crest of the maximum cross section. A large-volume land slide occurred at the upstream end of the reservoir with as much as 1.5 Mm³ sliding into the reservoir and resulting in a 2.4-m rise in the water level (Hinks et al., 2012; Kamada et al., 2017).

2.1.3 Earthquakes: $8.0 \leq \text{Magnitude}$

(1) The 1906 San Francisco Earthquake, Magnitude: 8.3

This earthquake affected about 30 medium-sized (6 to 40 m in height) earth dams. Most of them had little or only minor damage, such as minor longitudinal cracks on the crest, due to the shaking (Ambrassys, 1960; Seed et al., 1978). Seed et al. (1978) concluded that such a satisfactory performance under extreme loading conditions could be attributed more to the clayey nature of the construction materials, which generally exhibit little tendency for strength loss during shaking, than to the degree of compaction. Lower Crystal Springs Dam (a 38.1-m-high curved concrete gravity dam) withstood this earthquake without a single visible crack (USSD, 2003).

(2) The 1923 Kanto Earthquake, Magnitude: 8.3

This earthquake affected three embankment dams. Ono Dam (a 48.3-m-high earth-fill dam) was fractured with many fissures, 50 to 250 mm wide, developing along the crest of the dam ranging from 30 to 55 m in length and extending to a depth of about 11 m. One fissure separating the fill material from the core wall was found to extend 21 m below the crest. In addition, two local slides occurred on the downstream side of the dam. In Lower Murayama Dam (a 30.3-m-high earth-fill dam) and Upper Murayama Dam (a 23.7-m-high earth-fill dam), longitudinal cracks developed along the crest (Ambrassys, 1960; Seed et al., 1978).

(3) The 1943 Central Chile Earthquake, Magnitude: 8.3

This earthquake shook a number of dams in Chile. As a result of the earthquake, the crest of the Cogoti Dam (a 84-m-high concrete facing rock-fill dam), which was constructed of dumped rock-fill with a laminated concrete facing on the upstream slope, settled 380 mm, and there were minor rock slides on the downstream slope (Ambrassys, 1960; Seed et al., 1978).

(4) The 1985 Mexico Earthquake, Magnitude: 8.1

This earthquake shook two large rock-fill and earth-rock dams, El Infiernillo (148 m high) and La Villita (60 m high). El Infiernillo Dam has a rock foundation, while La Villita Dam is founded on alluvium. These dams suffered longitudinal cracking on the crest, but there were no functional problems (USSD, 2003; JDEC, 2005). They have been shaken since 1975 by a unique sequence of closely spaced events, including five with a magnitude above 7.1. Cumulative earthquake-induced settlements of La Villita Dam have approached one percent of its original height. A better performance has been observed at El Infiernillo Dam, an earth core rock-fill dam, with deformations that have remained small (USSD, 2003).

(5) The 2008 Wenchuan Earthquake, Magnitude: 8.0

Approximately 1800 dams were damaged during this earthquake, including four dams with heights exceeding 100 m. Cracks along the crest were observed at several embankment dams (Wieland, 2014; Wieland and Fallah, 2014).

(6) The 2011 off the Pacific coast of Tohoku Earthquake, Magnitude: 9.0

After this earthquake, special safety inspections were carried out at about 400 dams. Most of the dams sustained no damage, while some dams did suffer damage. The significant effects of the earthquake on dams were cracking, settlements, and slope failure of embankment dams, and cracking of the asphaltic concrete face of rock-fill dams (Matsumoto and Sasaki, 2012). Hatori Dam (a 37.1-m-high zoned earth-fill dam) suffered longitudinal cracks on the crest. A crack with a maximum width of 30 mm reached the core zone, and its maximum depth was 2.7 m. Nishigo Dam (a 32.5-m-high zoned earth-fill dam) suffered some longitudinal cracks on the crest, the same as Hatori Dam. A crack with a maximum width of 500 mm reached the core zone, and its maximum depth was 3.5 m. The upstream slope of Nishigo Dam deformed and a horseshoe-shaped pocket appeared (Hayashida and Masukawa, 2015). Surikamigawa Dam (a 105.0-m-high rock-fill dam) settled approximately 170 mm at the crest of the maximum cross section, and transverse cracks developed on the crest near both abutments. These cracks terminated at a depth of 300 mm within the protective layer just under the pavement, and did not reach the core zone. Kejonuma Dam (a 24.0-m-high zoned earth-fill dam) settled a maximum of 139 mm and deformed 58 mm in the downstream direction. Several small cracks developed on the crest in the stream direction, and their maximum widths were 25 to 30 mm (Yamaguchi et al., 2012). Koromogawa No. 1 Dam suffered a longitudinal crack on the crest and deformation at the shoulder of the upstream and downstream slopes again (Matsumoto and Sasaki, 2012). Cracks developed on the asphalt facing Minamikawa Saddle Dam (a 19.6-m-high asphalt facing rock-fill dam), Yashio Dam (a 90.5-m-high asphalt facing rock-fill dam), Numappara Dam (an asphalt facing earth-fill dam), and Miyama Dam (a 75.5-m-high asphalt facing rock-fill dam) during this earthquake (Yamaguchi et al., 2012; Tsukada et al., 2013; Okumura et al., 2015; Hayashida and Masukawa, 2015). An old earth-fill embankment of an irrigation pond, Fujinuma-ike, with a height of 18.5 m, breached. In Fukushima Prefecture, about 750 small embankment dams and irrigation ponds were damaged (Tanaka et al., 2012).

2.2 Features of seismic performance of dams

From the above review of past earthquake records, the features of the seismic performance of dams are summarized.

- 1) Concrete dams, especially arch dams and gravity dams, have shown superior seismic performance against massive earthquakes, except for dams with unconventional cross sections. A few buttress gravity dams have suffered horizontal cracking at the buttress portion. The only catastrophic failure of concrete dams was caused by the rupture of the fault.
- 2) The most common types of damage to rock-fill dams have been the settlement at the crest, minor transverse cracking at the connecting part between the abutment and the embankment, and minor longitudinal cracks on the crest. There have been no reports of serious damage to rock-fill dams, and it is considered that rock-fill dams have relatively high seismic performance. The seismic performance of a dam depends on the nature of its foundation, namely, dams above a bedrock foundation show higher seismic performance than dams above an alluvial foundation.
- 3) Earth-fill dams have suffered more various forms of damage than other types of dams. The most common types of damage to earth-fill dams have been settlement at the crest, longitudinal and transverse cracking on the crest, transverse cracking, and a difference at the connecting part between the abutment and the embankment, and sliding, cracking and a difference parallel to the dam-axis at the downstream or upstream slope. These types of damage were mainly concentrated in the upper part of the embankment and the crest, and such portions are the most vulnerable to earthquakes. The seismic performance of earth-fill dams greatly depends on the nature of the embankment and foundation materials. Earth-fill dams built of compacted clayey materials on stiff clay or bedrock foundations, even when obsolete or inefficient compaction procedures have been used, have historically withstood extremely strong levels of ground motion and show a high seismic performance. On the other hand, old embankments built of sandy or silty materials tend to suffer serious damage due to a reduction in strength or the liquefaction of the soil materials during the earthquake. This tendency becomes more remarkable in hydraulic-fill embankments than in rolled-fill embankments. From this fact, the seismic performance of hydraulic-fill dams is the most suspicious among all the dam types. However, when they are built with reasonable slopes on good foundations, they can survive moderately strong shaking with no harmful effects. This indicates that earth-fill dams composed of low-density sandy or silty materials are vulnerable to massive earthquakes, even if they have withstood past moderate earthquakes. And, in the case of earth-fill dams that include a structure whose properties are quite different from

those of the soil materials, such as concrete walls, the difference in their behaviors during an earthquake may cause a disturbance around their interface. In a dam with such a structure, it has been seen that the displacement and cracking of the embankment during an earthquake permitted water to erode around the concrete portion and caused the dam to fail.

From the above summary, it is obvious that the seismic performance of rock-fill dams is excellent, and that concrete dams are superior to embankment dams in seismic performance. While earth-fill dams have suffered more serious damage during earthquakes than other types of dams, their effects were mainly concentrated in the upper part and the crest of the embankment. Earth-fill dams, whose embankment and foundation are composed of clayey materials, show a higher seismic performance than those composed of sandy or silty materials. Rolled-fill embankments with high densities are superior in seismic performance to hydraulic-fill embankments with low densities. Earth-fill dams composed of low-density sandy or silty materials are vulnerable to massive earthquakes, and it is essential to assess the harmful effects of such earthquakes on the dams and to take measures to reduce them. Thus, this study has targeted the safety and seismic behavior of earth-fill dams in view of their seismic performance.

2.3 Seismic design of fill dams and literature review of studies on their possible failures

ANCLD (1998) enumerates the possible effects of earthquakes on fill dams, as follows.

- (a) Settlement and cracking of the embankment, particularly near the crest of the dam
- (b) Reduction of freeboard due to settlement which may, in the worst case, result in the overtopping of the dam
- (c) Instability of the upstream and/or downstream slope of the dam
- (d) Differential movement among the embankment, abutments, and spillway structures, increasing the likelihood of leakage and piping failure
- (e) Liquefaction or loss of shear strength of saturated granular soils in the embankment or its foundation due to an increase in pore pressure induced by the cyclic loading of the earthquake
- (f) Differential movement or instability on faults or other low-strength seams or defects passing through the dam foundation
- (g) Overtopping of the dam in the event of large tectonic movement in the reservoir basin or by seiches induced upstream
- (h) Overtopping of the dam by waves, due to earthquake-induced landslides, into the reservoir from the valley sides
- (i) Damage to outlet works passing through the embankment leading to the leakage and potential piping erosion of the embankment

Similar possibilities were listed by Seed (1979); they are all well-known effects among dam engineering professionals. Recent cases of seismic damage to fill dams coincide with some of the above-mentioned effects.

Generally, the seismic design analyses used in practice cover the instability of the upstream and/or downstream slopes of a dam, while the other possibilities are taken into account by the general design principles, the so-called “defensive design”.

Defensive design measures were presented by Sherard (1967) and Seed (1979), and ANCLD (1998) enumerated the following four measures as the most important ones.

- (1) Provide ample freeboard, above normal operating levels, to allow for settlement, slumping or fault movements which may cause the displacement of the crest.
- (2) Use well-designed and well-constructed filters downstream of the earth-fill core (and correctly graded rock-fill zones downstream of a concrete face for concrete face rock-fill dams) to control erosion in the event that the core is cracked in an earthquake. For larger dams, it might be better to adopt full-width filters (2.5 to 3 m) instead of narrower ones (1.5 m). This would

give greater security in the event of large crest deformations.

- (3) Provide ample drainage zones to allow for the discharge of flow through possible cracks in the core. For example, it should be ensured that at least part of the rock-fill dam is free draining or that extra discharge capacity is provided in the vertical and horizontal drains for earth-fill dams with such drains.
- (4) Avoid, densify, drain (to be non-saturated) or remove potentially liquefiable materials from the foundation or embankment.

Thus, settlement, cracking (especially, transverse cracking), and liquefaction directly affect the safety of fill dams, and they are the essential factors for assessing the seismic performance. However, it is difficult to predict and estimate such damage, and many studies have been carried out.

Newmark (1965) introduced the basic element of a procedure for evaluating the potential deformations of an embankment under earthquake loading. In this contribution, the sliding of a soil mass along a failure surface was likened to the slipping of a block on an inclined plane. Here, it is assumed that failure would be initiated and movements would begin when the inertia forces exceeded the yield resistance, and that movements would stop when the inertia forces were reversed. Once the yield acceleration and the acceleration time history of a slipping mass are determined, the permanent displacements can be calculated by double integrating the acceleration history above the yield acceleration. Thus, this method estimates the settlement occurring due to slipping during an earthquake. This method was modified and coupled with the dynamic response analysis by Makidisi and Seed (1978) and Watanabe and Baba (1981). Such methods have been used for estimating the seismic performance of rock-fill dams (Taniguchi et al., 1983; Watanabe et al., 1984; Elgamal, 1992; Yamaguchi et al., 2005; Okamoto and Shimada, 2009). Satoh et al. (2014) proposed an estimating method for the settlement occurring without slipping and evaluated the relatively large settlement in a real dam during an earthquake. Iwashita and Tsukuni (2001) estimated the permanent deformation in centrifugal tests modeling a rock-fill dam by the nonlinear effective stress method.

Failure modes, such as cracking, have been studied by means of physical modeling tests. The process for the progressive failure of a fill dam during an earthquake, including cracking and settlement, was revealed by Hasegawa and Kikusawa (1981 and 1982). Many researchers have studied the changes in the shaking mode and predominant periods of dam bodies according to the shape of the valley through numerical analyses (Aoyama, 1979; Makidisi et al., 1982; Mejia and Seed, 1983; Dakoulas and Gazetas, 1986; Gazetas and Dakoulas, 1992; Watanabe and Kawakami, 1995). In addition, Yasunaka (1986), Watanabe and Kawakami (1995), and Masukawa et al. (2000) carried out physical modeling experiments. Their results conclude that the behavior of a dam body is remarkably influenced by the abutment when the shape of the valley is steep. Ohmachi and Tokimatsu (1982 and 1983), and Yasunaka (1989a and 1989b) developed a practical method for a

three-dimensional earthquake response analysis of embankment dams. Papalu and Bielak (2001 and 2004) studied the interactive behavior between an earth dam and a canyon by a numerical analysis method. It was revealed that the inclusion of the dam-canyon interaction effects into the analysis leads to a reduction of the acceleration response of the dam with respect to that in which the canyon is assumed to be rigid. It is thought that the damage to the dam, which was concentrated in the upper part of the dam body around the crest, might have been related to the three-dimensional seismic response behavior of the dam body.

Seed et al. (1975b) applied the dynamic analysis procedures to the Upper and Lower Van Norman (San Fernando) Dam damaged by liquefaction during the 1971 San Fernando Earthquake. Gu et al. (1993) conducted post-earthquake deformation analyses using the finite element method. The analysis results showed that a progressive failure under undrained conditions may explain the observed response of the dams following the 1971 earthquake. Ming and Li (2003) analyzed the response of the Lower Van Norman Dam during the 1971 earthquake by using a fully coupled finite element procedure in conjunction with a critical state constitutive model for granular soils. Thus, the case history of the Lower Van Norman Dam has been verified by the latest prediction methods. Adalier and Sharp (2004) carried out a series of dynamic centrifuge tests to study the seismic behavior of a zoned embankment dam founded on saturated sandy deposits. As a result, severe deformations in terms of vertical crest settlements, slumping, and scarp formations were observed. Such deformations are excessive and, in reality, may either result in an immediate loss of the reservoir due to overtopping of the “slumped” crest or, if the crest does not get overtopped, cracking will likely distress the dam to the point that its ability to retain the reservoir will be compromised. Moreover, beneficial remedying effects were revealed by foundation densification. Koga et al (1992), Okamura and Tamamura (2011) and Okamura et al. (2013) performed centrifugal liquefaction experiments for river levees.

Such damage as settlement and cracking will occur from the results of complex relationships among several factors, and their mechanisms are unknown. It is important to reveal the seismic behavior of fill dams and to promote studies on the mechanisms that generate the possible failures, the methods for inspecting them, and remediation methods for properly mitigating the harmful effects on fill dams.

Chapter 3 Improvement in monitoring system for evaluation of dam safety

3.1 Defects of conventional system

Buried transducers, for example, pore water pressure and earth pressure transducers, are installed to ensure the stability and safety of embankment structures such as fill-type dams both under construction and while in use. Most are electrical transducers that change physical values into electrical signals and require wire cables to supply electrical power and to convey the measured data signals.

These wire cables often cause problems, such as (1) decreases in working efficiency while digging trenches for the cables, setting the cables, and burying the trenches (see Figure 3.1) and (2) decreases in measuring stability due to breaks in the cables, a decline in insulation and damage from thunderbolts. Therefore, a wireless transducer that applies underground communication technology with low-frequency electromagnetic waves was developed to improve the efficiency during the installation of the transducers and the stability of the measuring performance (Kohgo et al., 2006a; Kohgo et al., 2006b). This wireless sensor is called the “wireless pore water pressure transducer (WPT)”.



Figure 3.1 Cable trench for installing sensors

3.2 Wireless pore water pressure transducer (WPT)

3.2.1 Outline of underground data communication technology

Low-frequency electromagnetic waves characteristically show larger attenuation with distance, but the effect of the conductivity and permittivity of the transmission is lower than with high-frequency electromagnetic waves. Therefore, it is possible to communicate over short distances in either soil or water by low-frequency electromagnetic waves.

By applying this feature of low-frequency electromagnetic waves (8.5 kHz) and digital data communication technology, that can improve the precision for reading faint signals, it is possible to send and receive data under conditions when the signal to noise (S/N) ratio is greater than two and interactively communicate under the ground within a distance of 100 m.

Figure 3.2 shows the schematic of the underground data communication technology. There is directivity in the low-frequency electromagnetic waves generated around the antenna coil. Therefore, communication can be performed most effectively under the condition whereby the center axis of the transducer's antenna agrees with that of the antenna on the ground surface.

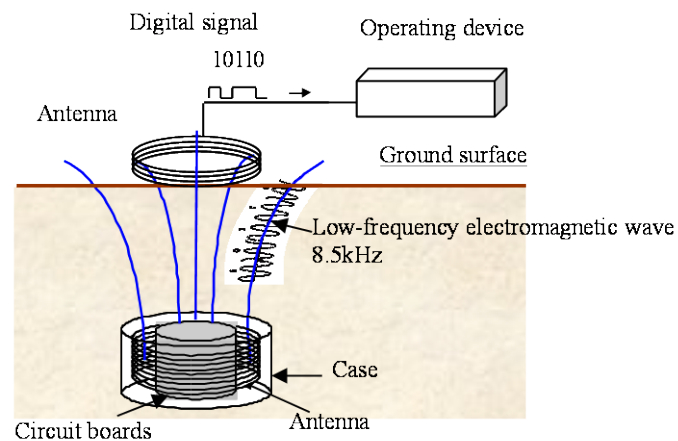


Figure 3.2 Schematic of underground communication technology

3.2.2 Outline of WPT

Figure 3.3 presents the schematic of the WPT. This figure shows the outward appearance and the inner components of the test model of the WPT installed in real dams. The WPT is composed of circuit boards, a communication antenna, batteries, a pore water pressure sensor, and a case. The diameter and the height of the WPT are 125 mm and 205 mm, respectively. The pore water pressure sensor is the same as conventional sensors with cables. In the case of the test model of the WPT, three batteries are set in a series.

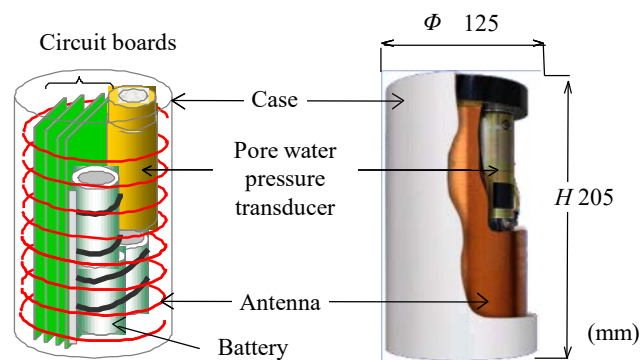


Figure 3.3 Schematic of test model of wireless pore water pressure transducer (WPT)

Figure 3.4 presents the schematics of the conventional measuring system and the underground wireless measuring system of the WPT. In the latter case, the operating device (data receiver) is set on the crest or in the inspection gallery; there are no cables in the dam body.

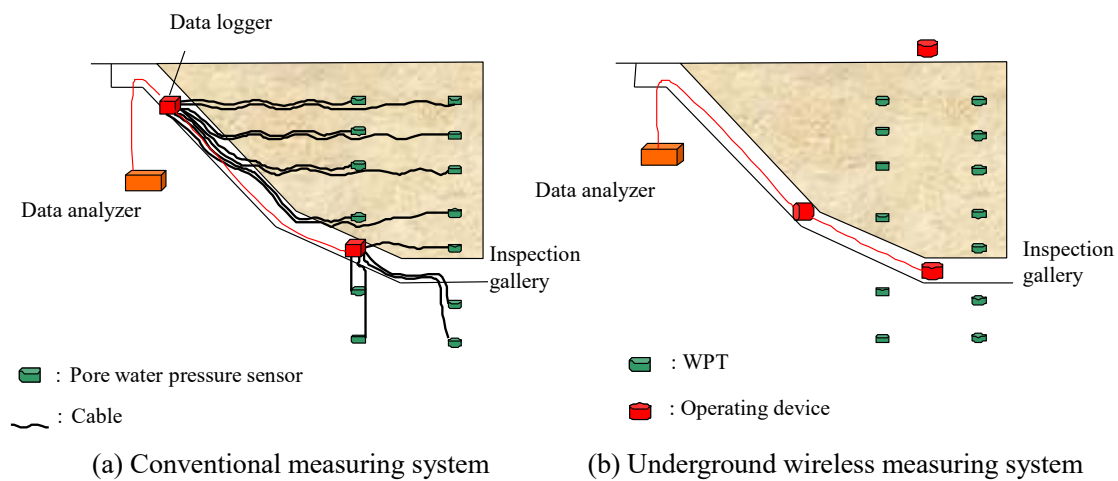


Figure 3.4 Comparison between conventional and underground wireless measuring systems

The outline of installation procedure for the WPT in Dam A is shown below, namely, (1) level a 1m^2 areas on the surface of the embankment to prepare the portion where the WPT will be installed, (2) bore and arrange the shape of the hole into which the WPT will be set, and check the installation elevation, (3) set the WPT, (4) bury the WPT by hand with grain-size arranged fill materials, (5) cover the WPT with fill materials and level off the area, and (6) perform compaction of the fill materials by a tamping machine. Figure 3.6 shows this procedure. It takes about 1 hour to install the WPT.



Figure 3.6 Procedure for installing WPT (core boring method)

Figures 3.7 to 3.9 show the data measured by the transducers installed at EL. 337.50 m near section No. 28 in Dam A. Figure 3.7 presents the data for WP-1, Figure 3.8 presents the data for P18 and WP-2, and Figure 3.9 presents the data for P19 and WP-3. Figure 3.7 shows the time series and the correlation between the WPT and the reserved water pressure, while Figures 3.8 and 3.9 show the time series and the correlation between the WPT and conventional sensors. The time series data include the changes in elevation of the embankments under construction and the reserved water pressure at the elevation of the transducers. Here, the reserved water pressure is the value reflecting the difference between the reserved water level and the elevation of transducers multiplied by the density of the water. If the water level is below the elevation of the transducers, its value is assumed to be 0 kPa.

From these figures, the difference between the values for the WPT and conventional sensors with cables is small, and it remains almost the same during the entire period of the tests. From these results, it can be concluded that the WPTs can measure the data in Dam A as well as conventional sensors with cables. On 5th March 2009, every WPT was able to communicate with the receiver on the ground surface. However, after 9th July 2009, WP-3 was not able to communicate with the receiver. It is clear, therefore, that this test model of the WPT stopped performing properly within five and a half years of its installation (in January 2004).

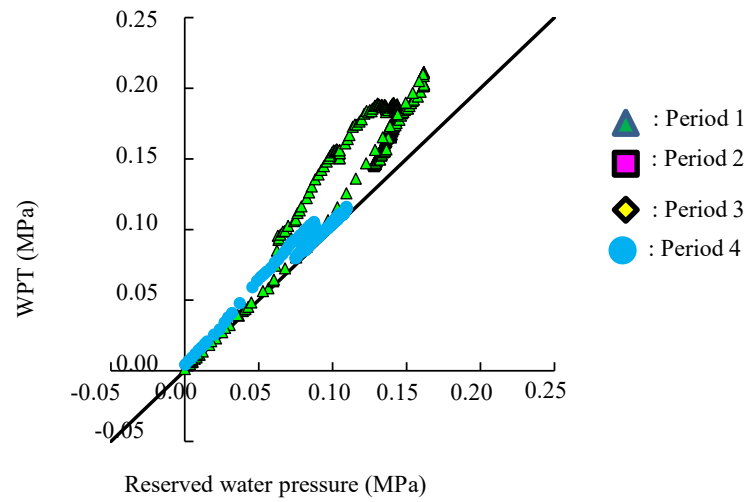
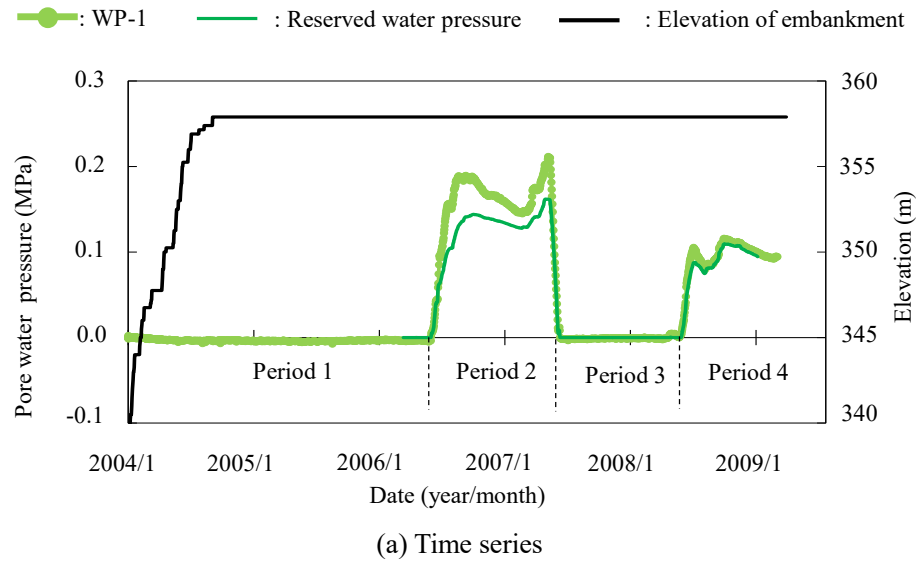
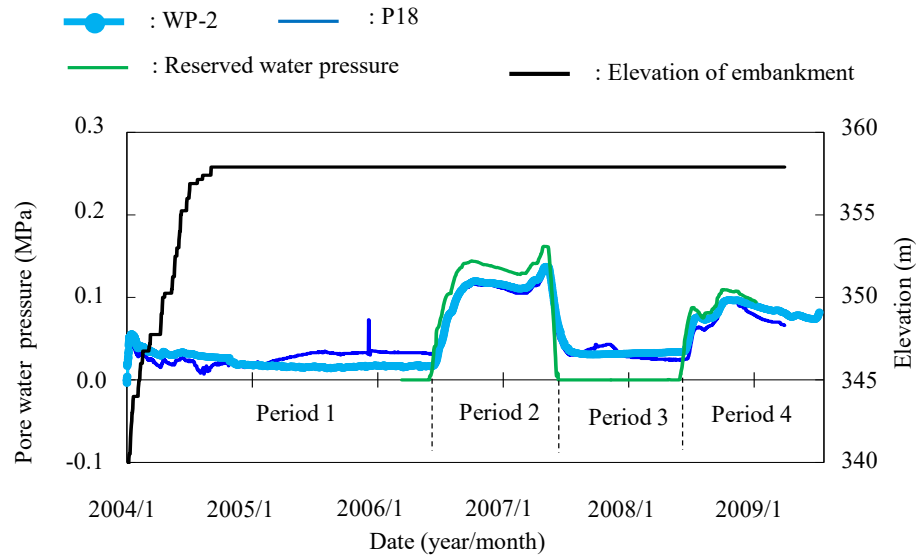
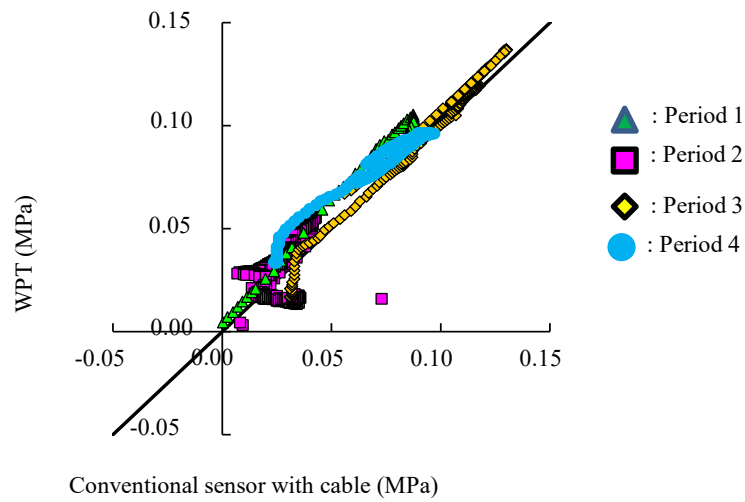


Figure 3.7 Data measured by WPT in Dam A (WP-1)

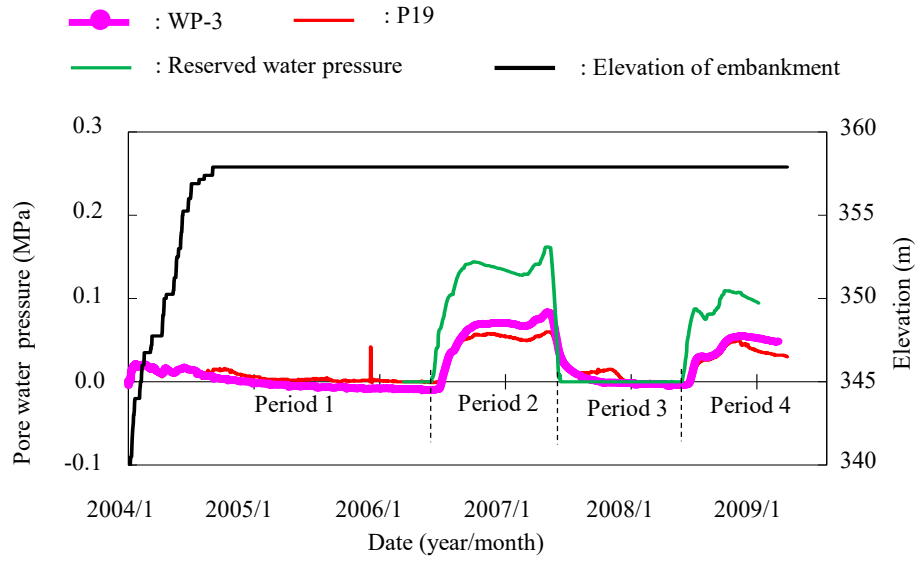


(a) Time series

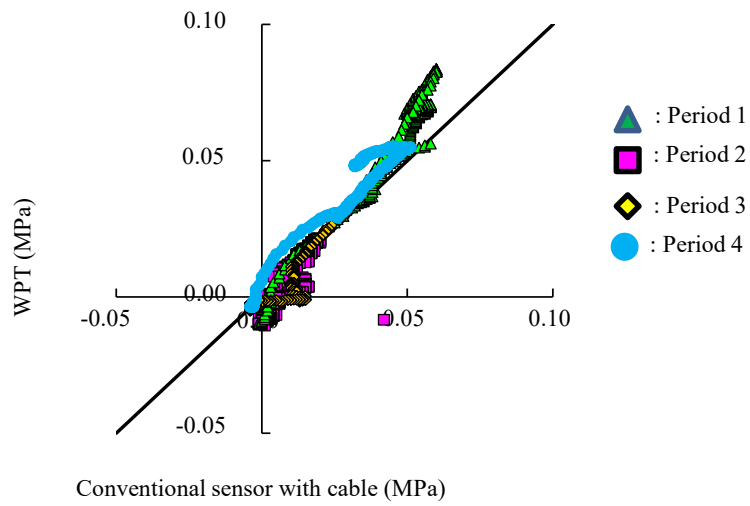


(b) Correlation between conventional sensor with cable and WPT

Figure 3.8 Data measured by conventional sensor with cable and WPT in Dam A (P18 and WP-2)



(a) Time series



(b) Correlation between conventional sensor with cable and WPT

Figure 3.9 Data measured by conventional sensor with cable and WPT in Dam A (P19 and WP-3)

3.3.2 Field test in Dam B

Dam B, shown in Figure 3.10 (a), is a center core type of rock-fill dam with a height of 50.0 m, a longitudinal length 256.0 m, and a volume of 801,000 m³. In May 2004, the test models for the WPT (WP-1 to WP-5) were installed by adjoining conventional sensors with cables (P35 to P39) at EL. 215.00 m of section No. 11+15.00. The arrangement of the test models is shown in Figure 3.10 (b). The test models were installed by the core boring method at the tip of the cable trench 5 m from section No. 11+15.00 in which conventional sensors with cables had been installed. WP-1 and WP-5 were installed in semi-pervious zones, while P36, P37, and P38 were installed in the core zone. Figures 3.11 and 3.12 show outlines of the installation of the WPT in the semi-pervious zones and the core zone, respectively.

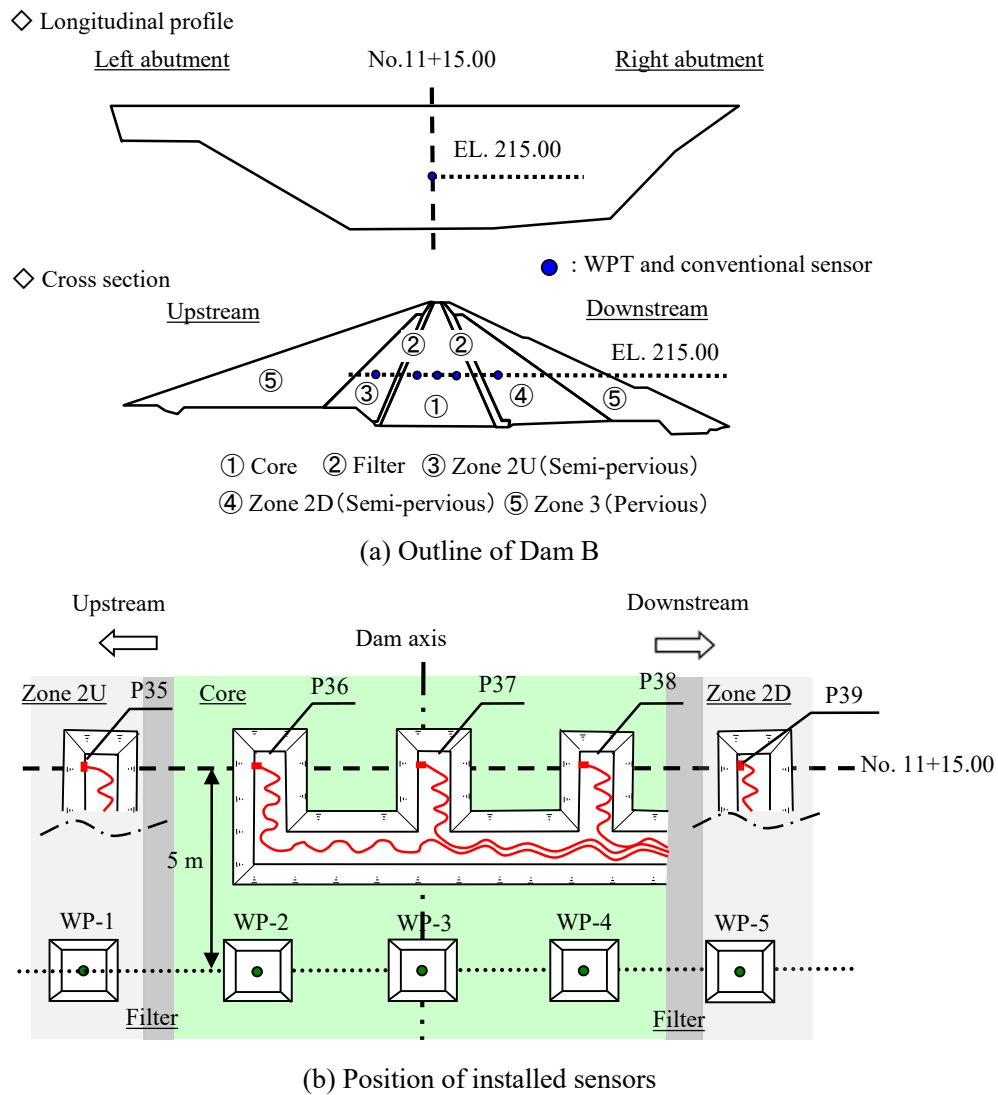


Figure 3.10 Outline of Dam B and position of installed sensors



Figure 3.11 Outline of WPT installation in semi-pervious zone

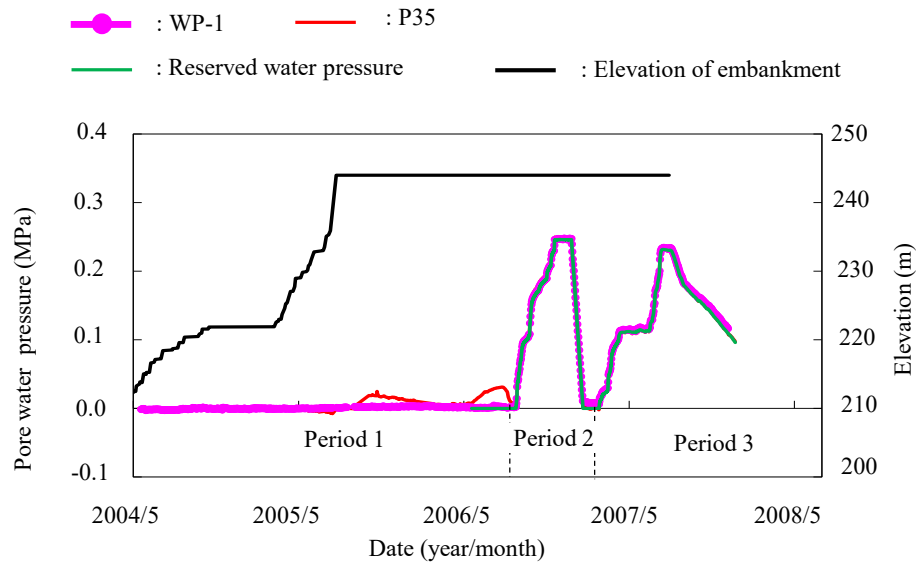


Figure 3.12 Outline of WPT installation in core zone

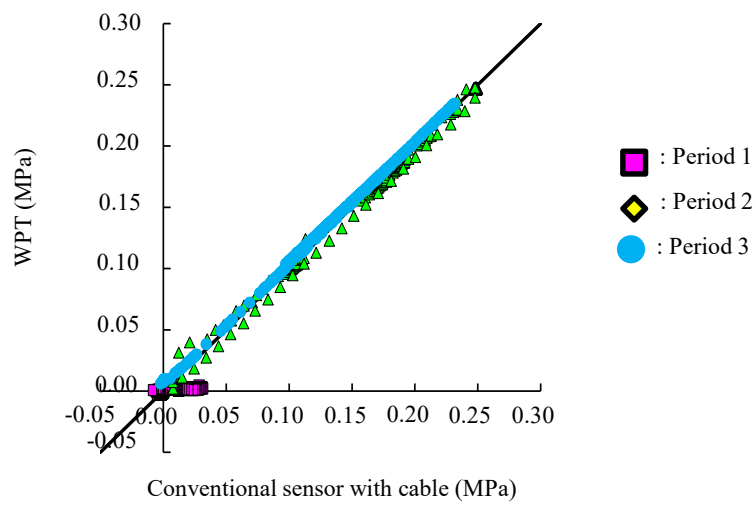
Figures 3.13 to 3.17 show the data measured by the transducers installed at EL. 215.00 m around section No. 11 in Dam B. Figure 3.13 shows the data for P35 and WP-1, while Figure 3.14 shows the data for P36 and WP-2. Figure 3.15 presents the data for P37 and WP-3, Figure 3.16 presents the data for P38 and WP-4, and Figure 3.17 presents the data for P39 and WP-5. All these figures show the time series and the correlation between the WPT and conventional sensors. The time series data for these figures include the changes in elevation of the embankment during its construction and the reserved water pressure at the elevation of the transducers. The reserved water pressure was calculated in the same manner as that for Dam A. If the water level was below the elevation of the transducers, its value was assumed to be 0 kPa.

On 27th December 2007, all the WPTs were able to communicate with the receiver on the ground surface, but after that no data collection was carried out. In September 2009, the test models of the WPTs were not able to communicate with the receiver on the ground surface, because the WPTs installed in Dam B were the same test models as those used for Dam A. Thus, the tests were stopped.

From Figures 3.13, 3.14, 3.15, and 3.17, the values of the WPTs are seen to be almost the same as those of conventional sensors with cables throughout the entire period of the tests. In Figure 3.16, the values of WP-4 are comparably lower than those of P38 during the period from the embanking to the time the reserved water level reached the transducer elevation during the first filling (Period 1). However, the tendencies of the data for WP-4 and P38 were similar throughout the entire test period, and after the reserved water level reached the transducer elevation (Periods 2 and 3), both became almost the same. The values of WP-4 and P38 were likely to have been different at first because of the differences in the installed positions and the installation method, and these influences were moderated by the reserved water. From the results, it is concluded that the WPTs can measure the data in Dam B as well as conventional sensors with cables.

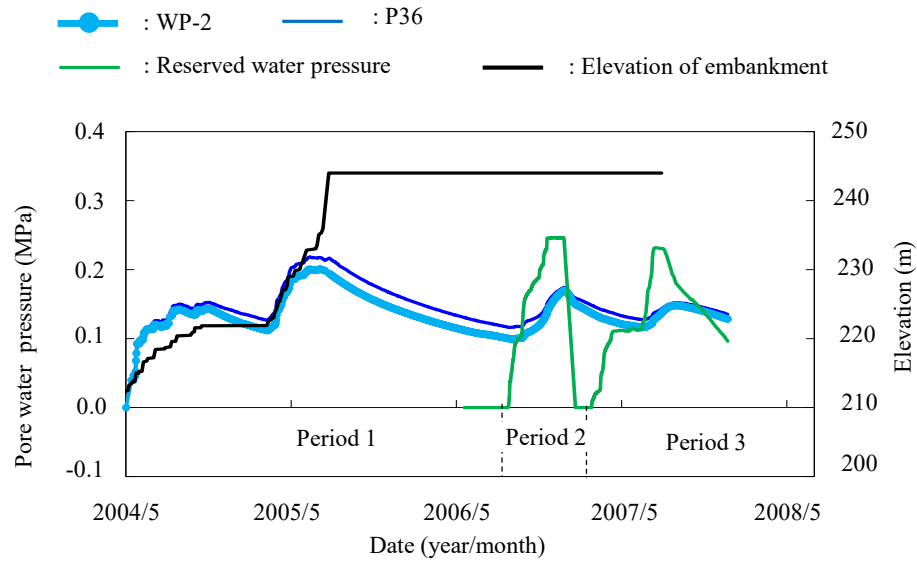


(a) Time series

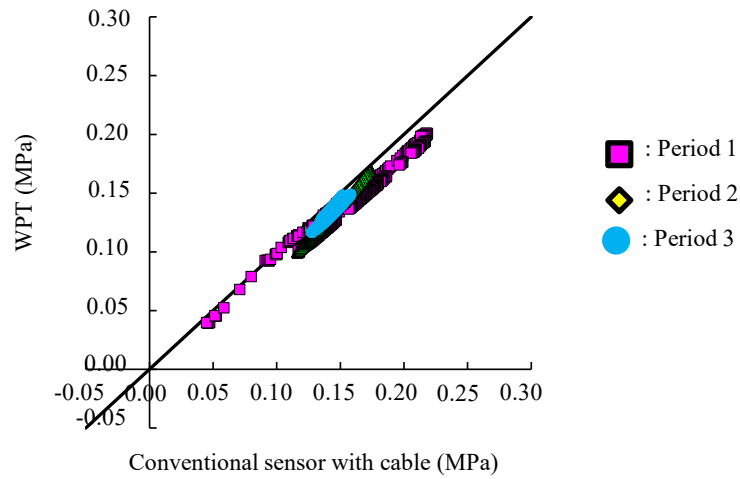


(b) Correlation between conventional sensor with cable and WPT

Figure 3.13 Data measured by conventional sensor with cable and WPT in Dam B (P35 and WP-1)

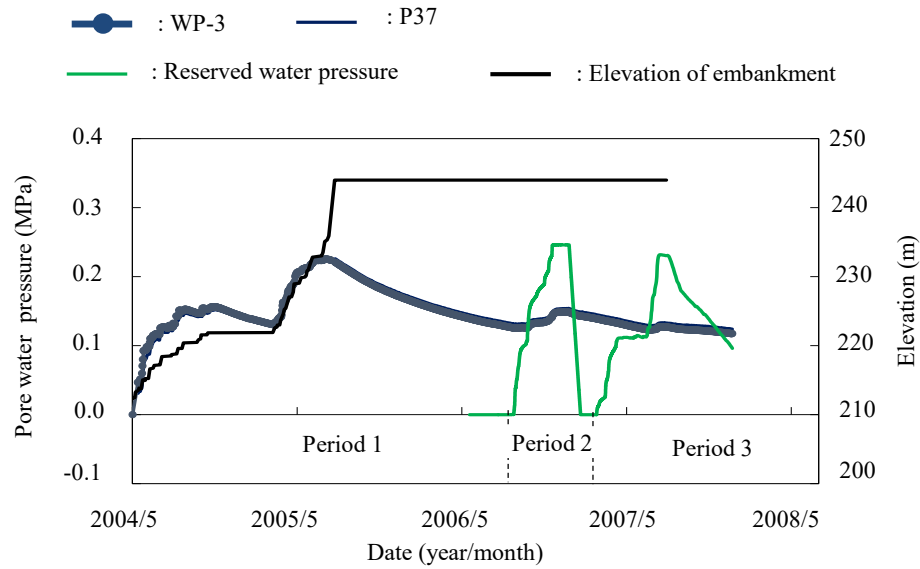


(a) Time series

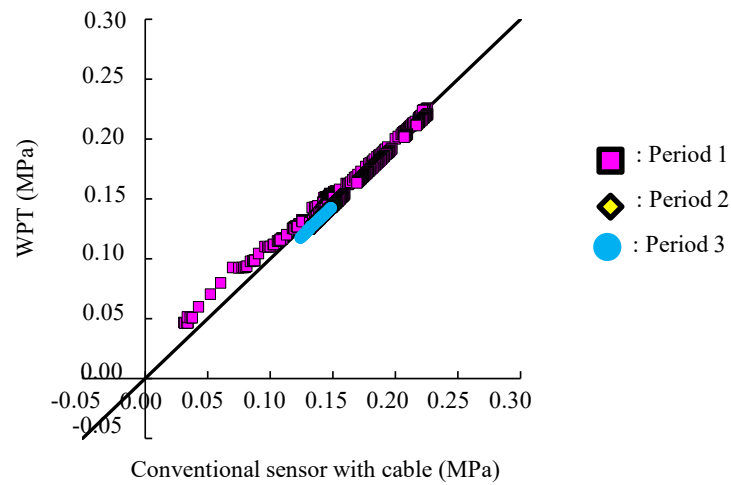


(b) Correlation between conventional sensor with cable and WPT

Figure 3.14 Data measured by conventional sensor with cable and WPT in Dam B (P36 and WP-2)

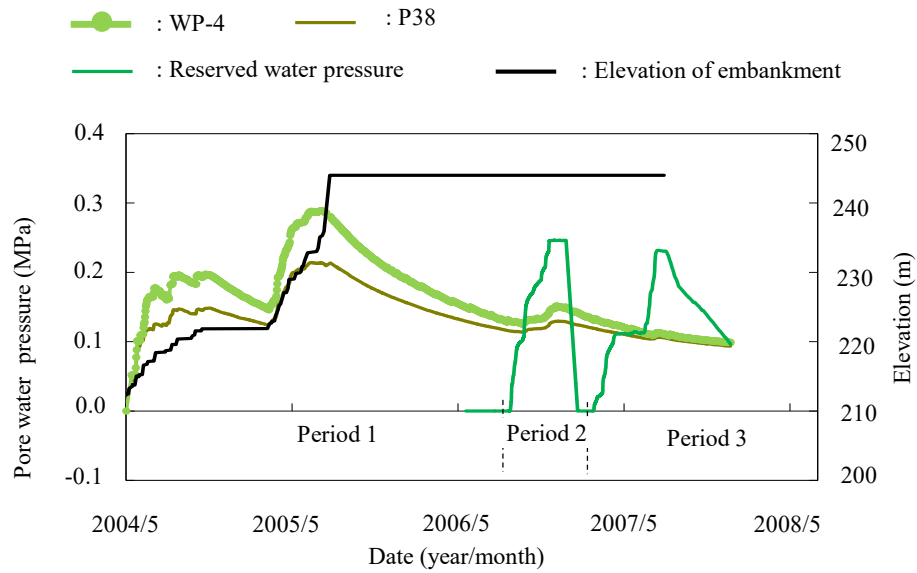


(a) Time series

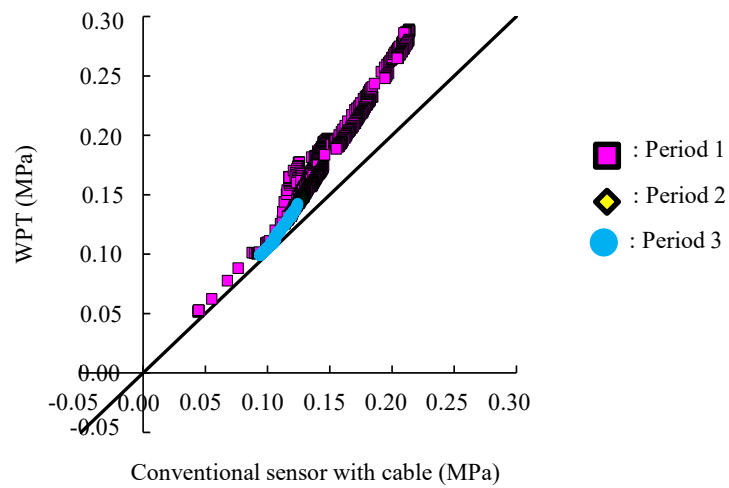


(b) Correlation between conventional sensor with cable and WPT

Figure 3.15 Data measured by conventional sensor with cable and WPT in Dam B (P37 and WP-3)

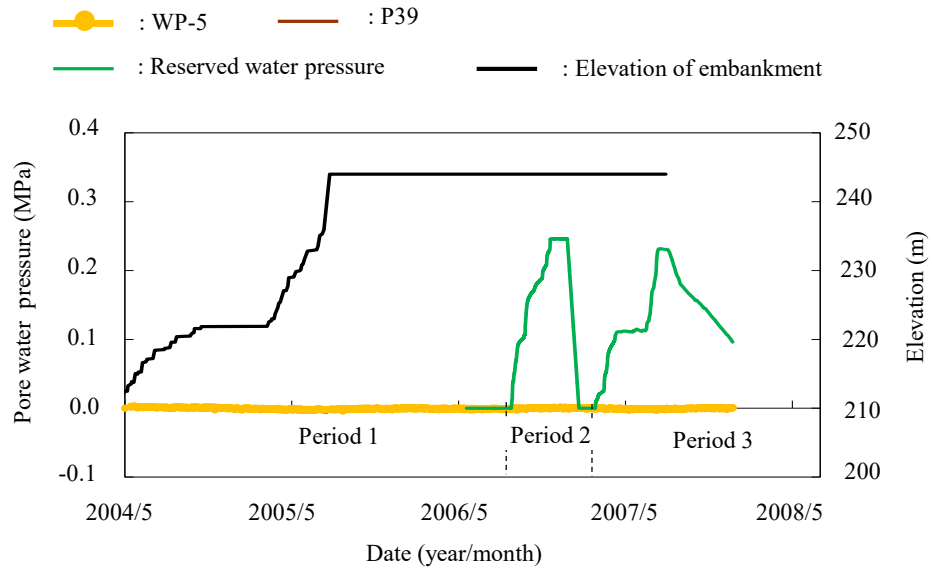


(a) Time series

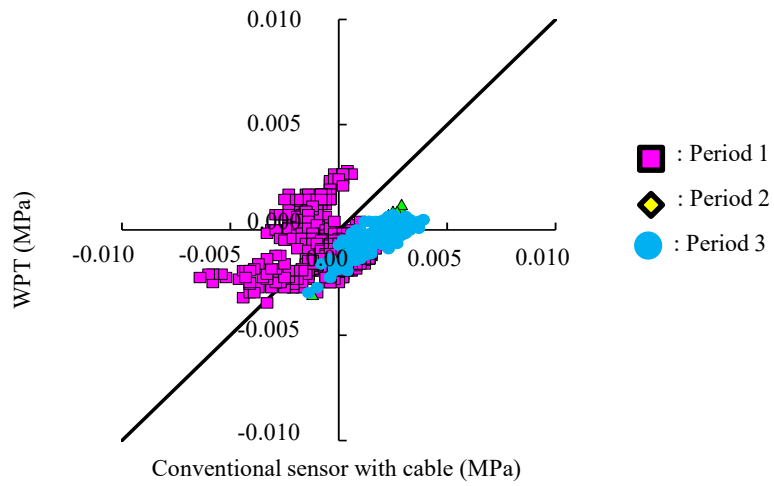


(b) Correlation between conventional sensor with cable and WPT

Figure 3.16 Data measured by conventional sensor with cable and WPT in Dam B (P38 and WP-4)



(a) Time series



(b) Correlation between conventional sensor with cable and WPT

Figure 3.17 Data measured by conventional sensor with cable and WPT in Dam B (P39 and WP-5)

3.3.3 Field test in Dam C

Dam C, shown in Figure 3.18, is a center core type of rock-fill dam with a height of 52.0 m, a longitudinal length of 298.0 m, and a volume of 935,000 m³. In this dam, the WPTs with earth pressure transducers (see. Figure 3.19) were installed. Each sensor consists of a WPT, which is utilized as the wireless data logger, and an earth pressure transducer connected to the WPT by a 1.5-m cable. Figure 3.20 shows the installation procedure for the WPT with an earth pressure transducer. A trench whose area is 3.0 m × 1.2 m must be dug in order to install the earth pressure transducer and cables, but the time required for the installation is not much in excess of that required for the installation of only the WPT. The total numbers of WPTs and WPTs with earth pressure transducers installed in Dam C are both 16. In this dam, no comparison was carried out between the WPTs (the WPTs with earth pressure transducers) and conventional sensors.

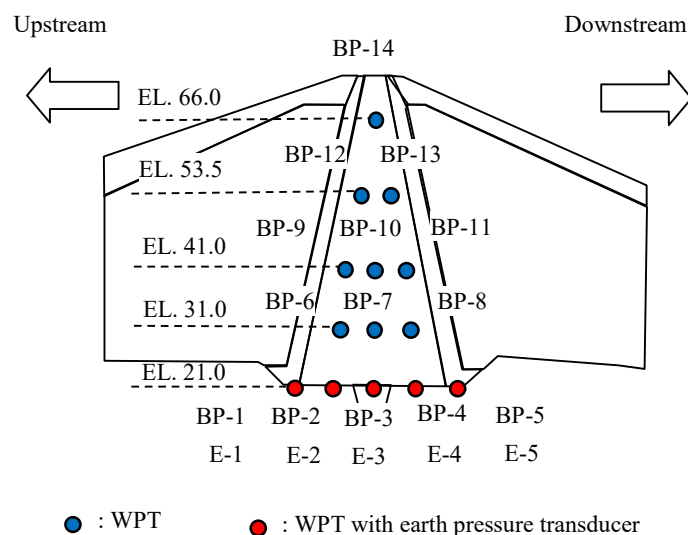


Figure 3.18 Outline of Dam C and position of installed sensors

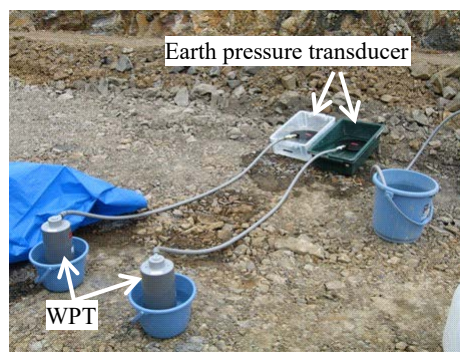


Figure 3.19 WPT with earth pressure transducers

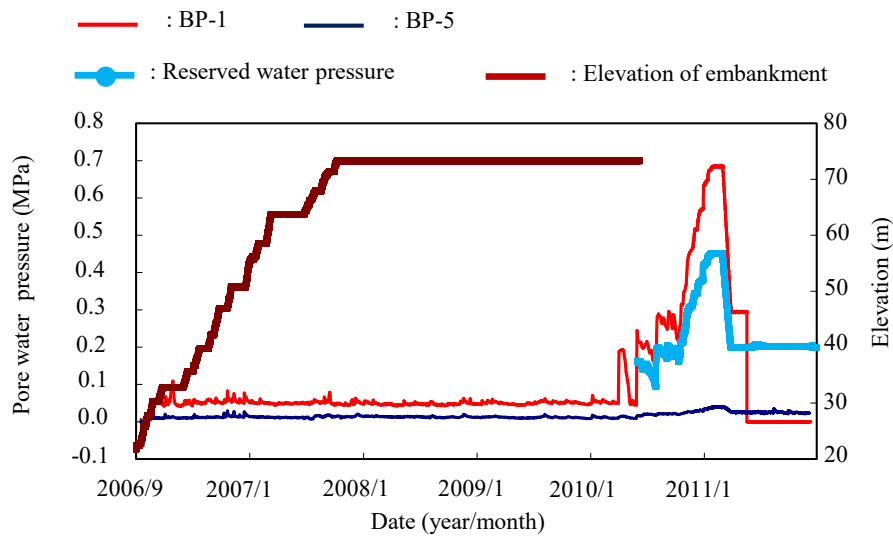


Figure 3.20 Procedure for installing WPT with earth pressure transducer

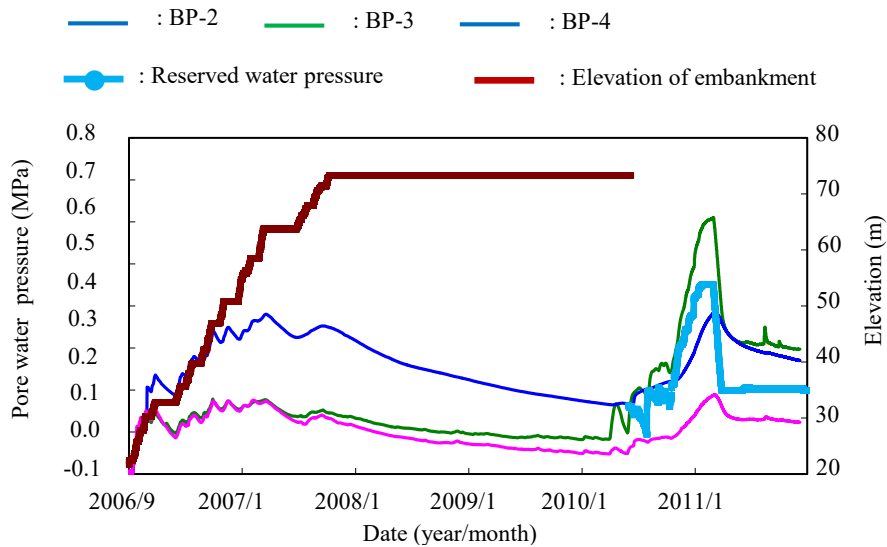
Figures 3.21 and 3.22 show the time series data of the WPTs with earth pressure transducers which were installed at the base of the maximum cross section of the dam body (BP-1 to BP-5 and E-1 to E-5, see Figure 3.18). Figure 3.21 (a) presents the data for the pore water pressures measured by BP-1 and BP-5, installed in the filter zone, while Figure 3.21 (b) presents the data for the pore water pressure measured by BP-2, BP-3 and BP-4 installed in the core zone. These figures show the changes in elevation of the embankment during construction together with the reserved water pressure at the elevation of the transducers. The reserved water pressure was calculated in the same manner as that for Dam A and Dam B. Figure 3.22 shows the data for the earth pressures measured by E-1 to E-5. This figure includes the changes in the soil column weight during the construction of the embankment and the elevation of the reserved water. Here, the soil column weight is the product of the difference between the embankment level and the elevation of the transducers multiplied by the bulk density of the soil.

From Figure 3.21 (a), it is found that the values of the pore water pressure in the filter zone (BP-1 and BP-5) are almost zero during construction. After the first filling, the value of the pore water pressure in BP-1, installed in the upstream, changes according to the reserved water level. However, the value of the pore water pressure in BP-5, installed in the downstream, is continuously almost zero. From Figure 3.21 (b), it is found that the values of the pore water pressure in the core zone (BP-2, BP-3, and BP-4) increase according to the advance of the embankment level and decrease after a pause in the embankment construction or after the construction has been completed. The value of the pore water pressure in BP-3 at the center of the core zone is higher than that in either BP-2 or BP-4 during the construction. After the first filling, all the values of the pore water pressure in the core zone change according to the reserved water level; namely, the value of the pore

water pressure of BP-2 at the most upstream area of the core zone is the highest and the values of BP-3 and BP-4 are lower sequentially. It is seen that the core zone can control the seepage appropriately. From Figure 3.22, it is found that the values of the earth pressure increase according to the advance of the embankment. The values of the earth pressure in the filter zone (E-1 and E-5) are higher than those in the core zone (E-2, E-3, and E-4). This behavior indicates the arching action commonly recognized in rock-fill dams. From these results, it is concluded that the WPTs with earth pressure transducers in Dam C can properly measure both the pore water pressure and the earth pressure, and that the measured data represent the common behavior recognized in rock-fill dams.



(a) Filter zone (BP-1 and BP-5)



(b) Core zone (BP-2, BP-3, and BP-4)

Figure 3.21 Pore water pressures measured by WPTs in Dam C

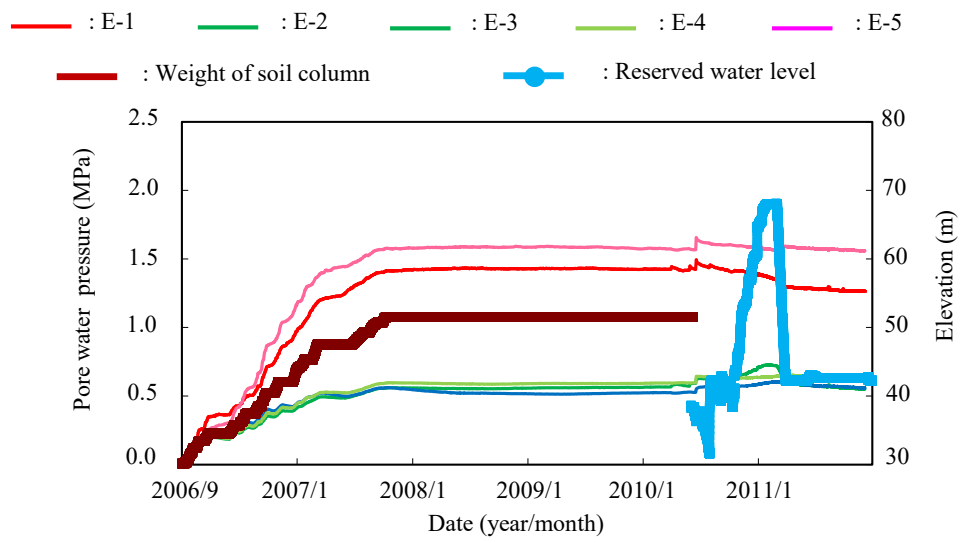


Figure 3.22 Earth pressures measured by WPTs in Dam B

3.4 Life extension of underground wireless system

3.4.1 Battery loading tests

WPTs are expected to work properly for ten years, but the period of development is comparably shorter than the expected life of the WPTs. Thus, accelerated battery loading tests were carried out in the laboratory to estimate the performance of the batteries used in the WPTs. Figure 3.23 shows the schematic of a testing device that imitates the electric consumptions of the WPTs, while Figure 3.24 shows the loading pattern of the tests.

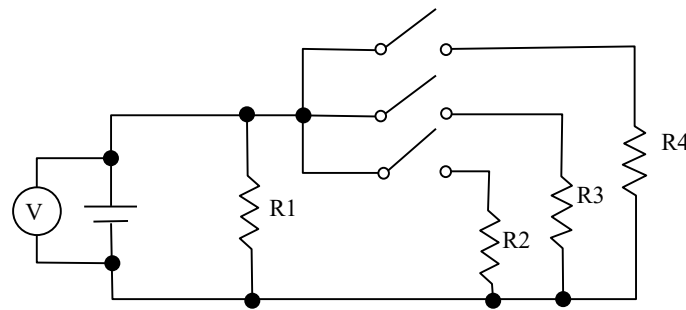


Figure 3.23 Schematic of a testing device

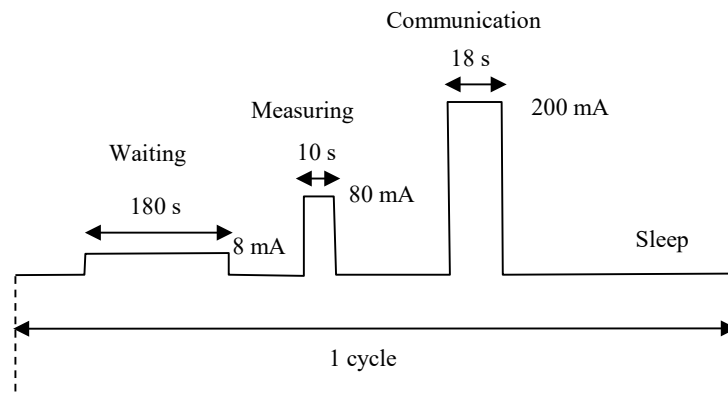


Figure 3.24 Loading pattern

The test device is composed of a voltmeter to measure the voltage of the battery and four resistors imitating the electric consumptions under each mode; R1 is the sleeping mode, R2 is the waiting-for-order mode, R3 is the measuring mode, and R4 is the communication mode. Here, the sleeping mode is the state when only the clock in the WPT works, the waiting-for-order mode is the state when the WPT can receive signals from the ground surface, the measuring mode is the state

when the WPT can measure the pore water pressure, and the communication mode is the state when the WPT can communicate with the receiver on the ground surface.

Tables 3.1 and Table 3.2 show the specifications of the loading conditions and the test series, respectively. In Case 1, the loading pattern, shown in Figure 3.24, was repeated 20 times a day and the amount of electricity normally consumed in ten years was hastily consumed in half a year. Similarly, the loading patterns were loaded ten times, four times and one time a day in Cases 2, 3, and 4, respectively, and the periods of the test were shortened to 1/10, 1/4, and once in ten years, respectively.

Table 3.1 Specifications of loading conditions

Loading condition	Test case	Resistance	Rated current	Loading time
R1 Sleep	Case 1	3.3 k Ω	1 mA	always
	Case 2	6.6 k Ω	0.5 mA	
	Case 3	16.5 k Ω	0.2 mA	
	Case 4	66.0 k Ω	0.05 mA	
R2 Waiting	Common	432.2 Ω	8 mA	180 s
R3 Measuring	Common	39.3 Ω	80 mA	10 s
R4 Communication	Common	14.1 Ω	200 mA	18 s

Table 3.2 Test series

Test case	Period of test	Time / cycle	Frequency of loading
Case 1	0.5 years	72 min	20 times/day
Case 2	1 year	144 min	10 times/day
Case 3	2.5 years	6 hour	4 times/day
Case 4	10 years	24 hour	1 time/day

Table 3.3 Specifications of battery

Nominal capacity	13.0 Ah
Nominal voltage	3.6 V
Cut-off voltage	2.0 V
Self-discharge ratio	Below 3% / year

Table 3.3 shows the specifications of the lithium ion battery used in the WPT. This battery has a nominal capacity of 13.0 Ah and discharges 3% of the nominal capacity in a year. Therefore, allowances must be made for the self-discharge of the battery, and the battery is not guaranteed to perform properly after reaching the cut-off voltage of 2.0 V.

Figure 3.25 shows the changes in voltages of the battery during the sleeping and communication modes in each case. In each case, the four test devices, shown in Figure 3.23, were used simultaneously. Each value shown in Figure 3.25 is the average of the four devices. It is seen from this figure that the tendencies of the voltages in Cases 1, 2, and 3 are considerably different from that in Case 4, during the communication and sleeping modes. In Cases 1, 2, and 3, the voltage during both modes was kept above 3.0 V throughout the entire period and rapidly decreased to 0 V through the cut-off voltage of 2.0 V at the end. Such behavior of the voltage may be caused by the complete consumption of the battery capacity during the test period. On the other hand, in Case 4, the voltage values under the communication mode decreased moderately with the passage of time and reached the cut-off voltage. The voltage during the sleeping mode maintained a nominal voltage value of 3.6 V throughout the entire period of the test. From these results, it is supposed that in Case 4, the voltage of the battery during the communication mode reached the cut-off voltage before the battery capacity had been fully consumed.

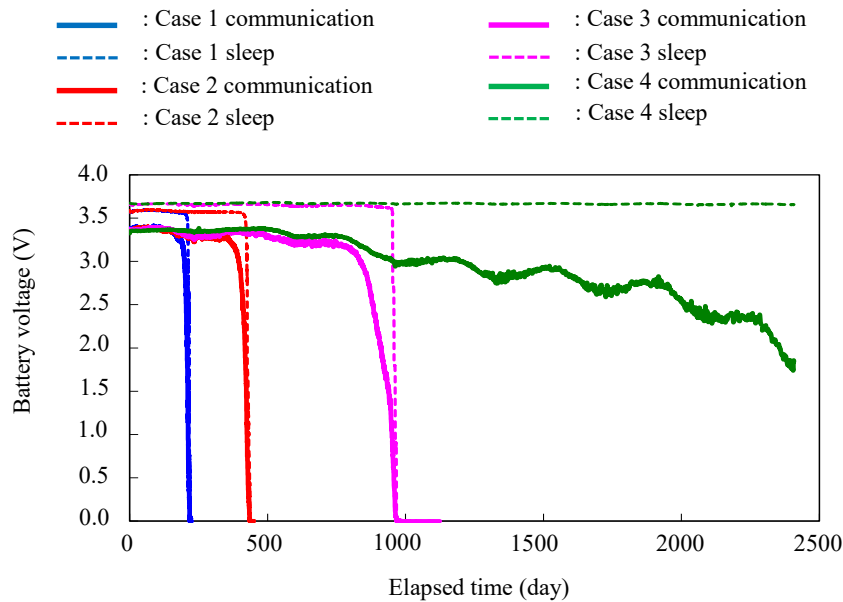


Figure 3.25 Changes in battery voltage under sleep and communication modes

To confirm this hypothesis, the capacities consumed under each mode are estimated. First, the electric current is calculated from the resistance of each resistor and the battery voltage is measured under each mode. Then, the electric current is multiplied by each loading time to obtain the capacities consumed under each mode. These values are added through the actual working period until the voltage during the communication mode reaches the cut-off voltage of 2.0 V. This result is defined as the “consumed capacity by loading”. Next, the self-discharged capacity through the actual working period is calculated as the product of the self-discharged capacity for one year, 0.39 Ah, which is 3% per year of the nominal capacity of 13.0 Ah multiplied by the actual work period. The sum of the consumed capacity by loading and the self-discharge capacity is defined as the “actual consumed capacity”. Finally, the ratio of the actual consumed capacity and the nominal capacity is defined as the “consumed capacity ratio”. Table 3.4 shows the actual working period, the consumed capacity for loading, the actual consumed capacity, and the consumed capacity ratio in each case. Figure 3.26 shows the consumed capacity in each case, including the consumed capacities during the sleep mode, waiting mode, measuring mode, and communication mode, and the self-discharged capacity. From Table 3.4 and Figure 3.26, it is revealed that the capacities of the battery are consumed fully in Cases 1, 2, and 3, but 26 % of the nominal capacity remains in Case 4.

Generally, the terminal voltage at the connecting resistor is lower than the electromotive force (terminal voltage without a connecting resistor). This phenomenon is caused by the virtual resistors in the battery that are connected in a series, and the virtual resistors have an internal resistance. The internal resistance causes a drop in voltage proportional to the magnitude of the electric current. If the electric current is small, the influence of the internal resistance on the drop in voltage will not be remarkable. On the other hand, if the electric current is large, problems caused by the drop in voltage will not be negligible. The internal resistance normally increases with the deterioration of the battery according to the discharge. One of the reasons for the internal resistance is that the effective areas on the pole plates decrease in the battery.

Table 3.4 Consumed capacity ratio in each case

Test case	Supposed period	Actual working period	Consumed capacity by loading	Actual consumed capacity	Consumed ratio
Case 1	0.5 years	0.57 years	13.0 Ah	13.3 Ah	102.3 %
Case 2	1 year	1.14 years	13.0 Ah	13.4 Ah	103.1 %
Case 3	2.5 years	2.49 years	11.4 Ah	12.3 Ah	94.6 %
Case 4	10 years	6.45 years	7.0 Ah	9.6 Ah	73.8 %

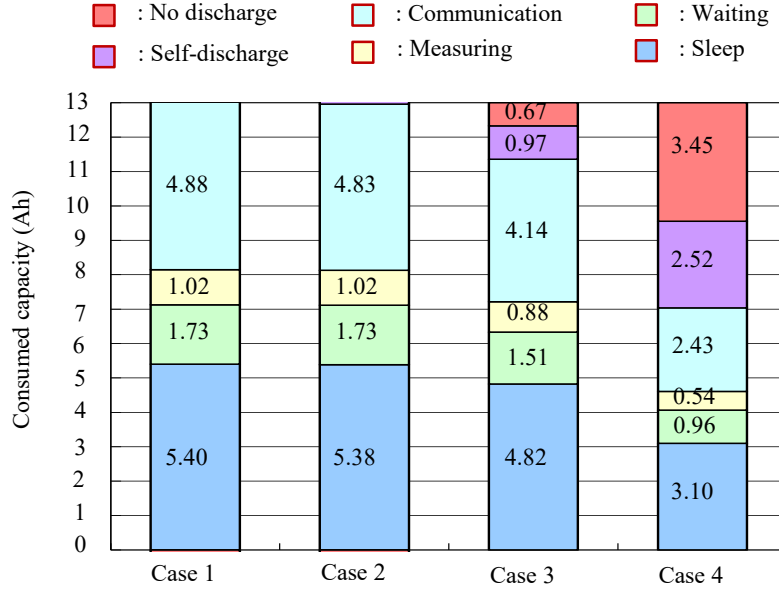


Figure 3.26 Consumed capacity in each case

The values of internal resistance in Cases 3 and 4 are estimated to clarify the influence of the internal resistance on the test results. The internal resistance is calculated from the values of the battery voltage and the electric current under the communication mode by the equations shown below. Here, it is assumed that the battery voltage during the sleeping mode is equal to the electromotive force.

$$R_{in} = (V_{sleep} - V_{send}) / I_{send} \quad (1)$$

$$I_{send} = V_{send} / R_{send} \quad (2)$$

where, R_{in} is the internal resistance (Ω), V_{sleep} is the battery voltage during the sleeping mode (V), V_{send} is the battery voltage during the communication mode (V), I_{send} is the electric current during the communication mode (A), and R_{send} is the resistance of resistor R4 mentioned in Figure 3.23 (Ω).

Figure 3.27 shows the changes in the estimated internal resistance. The internal resistance showed a similar tendency up to 750 days in both cases. Subsequently, the internal resistance in Case 3 increased rapidly to 10 Ω approximately 900 days later, and the battery voltage during the communication mode became lower than the cut-off voltage of 2.0 V. On the other hand, the internal resistance was successively increased in Case 4. The relationship between the internal resistance and the elapsed time is approximated by an exponential curve as equation (3).

$$R_{in} = 1.0094 \exp(0.0009t) \quad (3)$$

where, R_{in} is the internal resistance (Ω) and t is the elapsed time (day).

From these results, it is concluded that the capacity of the battery remained sufficient in Case 4, but that the battery voltage dropped below the cut-off voltage during the communication mode, which requires a large electric current, because the internal resistance successfully increased with time.

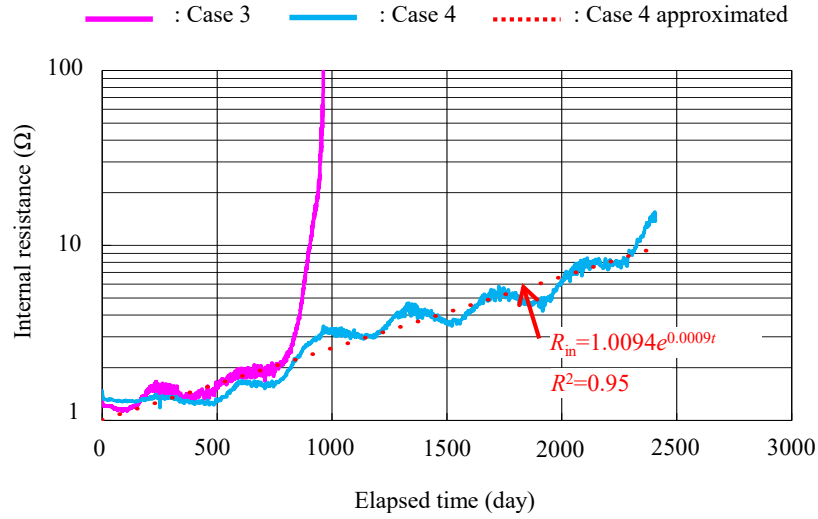


Figure 3.27 Changes in internal resistance in Cases 3 and 4

3.4.2 Comparison with test results for real dam

Figure 3.28 shows the changes in battery voltage during the communication mode of the test model in Dam A and the results of battery loading in Case 4. Here, the results of the battery loading tests were revised for an easy comparison because the number of batteries used in the test model of the WPT and that used in the battery loading tests differ. From Figure 3.28, the data measured in Dam A and the results of the battery loading tests show similar tendencies, namely, that decreasing ratios for the battery voltage become large after approximately 500 days and then they remain almost the same. The capacity consumed over ten years is estimated to be 11.6 Ah by revising the data of the battery loading tests in daily measuring and weekly communication. The value of the consumed capacity over ten years was below the nominal capacity of 13.0 Ah. From these results, the capacity of the battery appears to be sufficient for a proper performance for ten years under the expected conditions. Therefore, it is concluded that the reason the test models of the WPT in Dam A were only able to communicate for approximately five and a half years is not due to the lack of battery capacity, but to a drop in voltage during the communication mode induced by an increase in internal resistance.

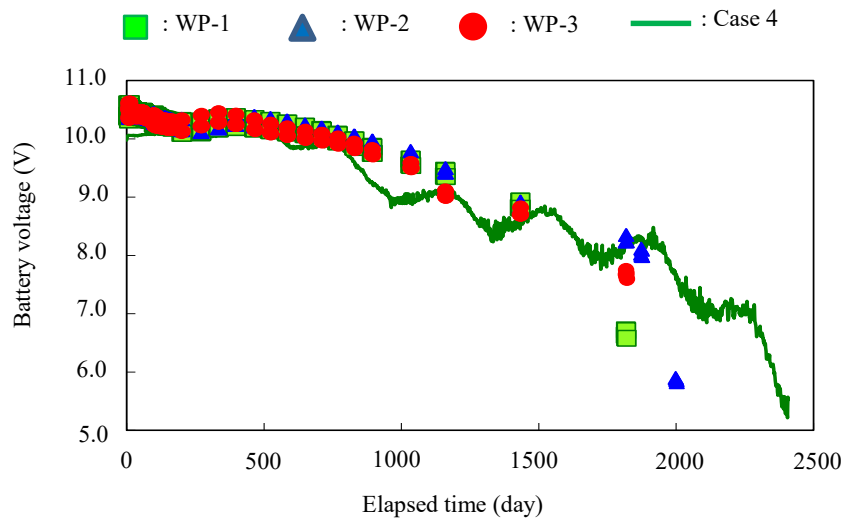


Figure 3.28 Comparison of battery voltage between Case 4 and WPTs in Dam A

3.4.3 Improvement in life extension of underground wireless system

It is essential for confirming the regular operation of a WPT during a supposed period that the influence of the drop in voltage during the communication mode, induced by an increase in internal resistance, be estimated properly and moderated by some improvements.

Reducing the consumed electrical current during the communication mode is an efficient way to moderate the effect of the drop in voltage. If batteries whose numbers are n are set in parallel with the systems, the electrical current of each battery is reduced $1/n$ times, and the drop in voltage induced by the internal resistance becomes $1/n$ times. If it is assumed that the internal resistance of the battery will change as equation (3) and the batteries are set in parallel, the battery voltage during the communication mode after t years will be estimated as follows:

$$V_{send}(t,n) = V_{nomi} - V_{in}(t,n) \quad (4)$$

$$V_{in}(t,n) = 1.0094 \exp(0.0009t) \frac{I_{send}}{n} \quad (5)$$

where, $V_{send}(t,n)$ is the battery voltage during the communication mode (V), V_{nomi} is the nominal voltage of the battery (V), $V_{in}(t,n)$ is the drop in voltage of the battery induced by the internal resistance (V), I_{send} is the nominal electrical current during the communication mode (A), t is the elapsed time (day), and n is the number of batteries set in parallel.

Figure 3.29 shows the changes in the battery voltage during the communication mode according to the number of batteries set in parallel and the elapsed time. Here, it is assumed that the battery voltage under the no-loading condition keeps the nominal voltage (V_{nomi}) at 3.6 V until the capacity of the batteries has been entirely consumed and the electrical current during the communication mode (I_{send}) is always equal to 0.2 A. From Figure 3.29, when the number of batteries set in parallel (n) is two, the battery voltage under the communication mode will be above the cut-off voltage (2.0 V) and the WPT can perform regularly for 3,000 days (8.2 years). Similarly, it is estimated that the number of batteries set in parallel to confirm the regular operation of the WPT for 10 years needs to be over 3.

The test model of the WPT was improved during the development phase and the improved WPT was installed in Dam C. The improved WPT contained an increased number of batteries in parallel and decreased the consumption of the electricity during the communication mode. Figure 3.30 shows the changes in the battery voltage during the communication mode between the test model in Dam A and the improved WPT in Dam C. Their tendencies are quite different and the value of the improved WPT maintains almost the same value as that of the initial condition even after 2,000 days.

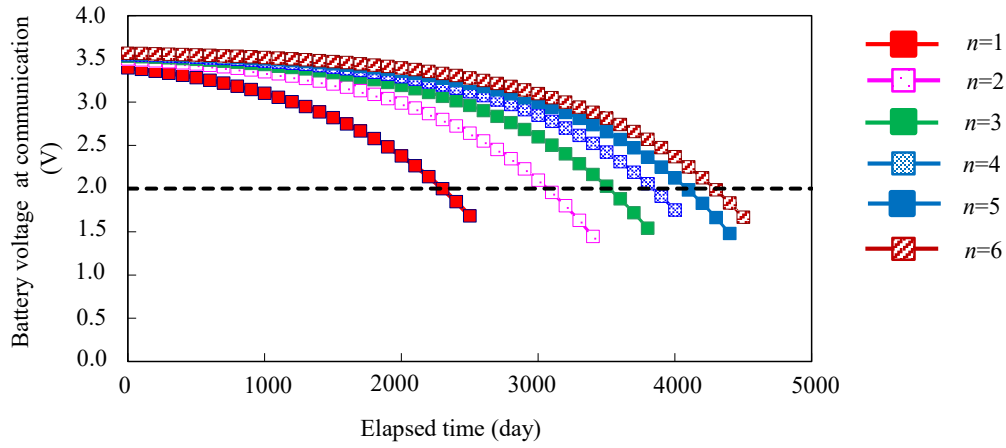


Figure 3.29 Changes in battery voltage estimated by equations (4) and (5)

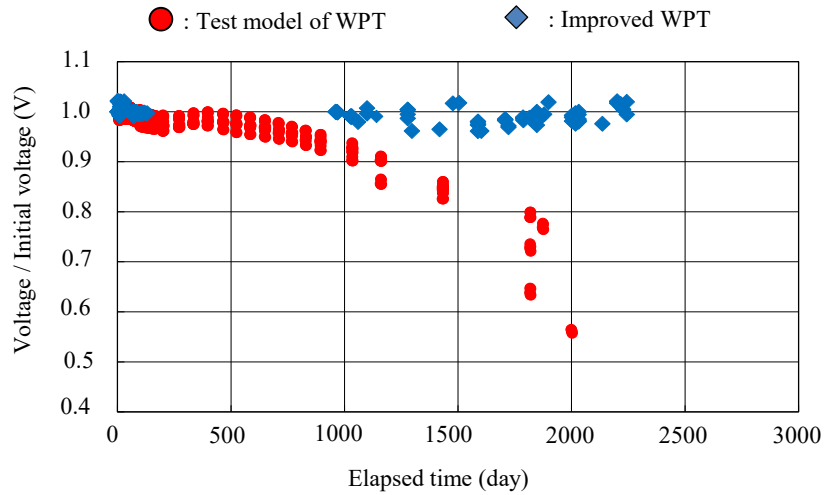


Figure 3.30 Comparison of voltage/initial voltage between test model and improved WPT

Figure 3.31 shows the changes in internal resistance of the test model and the improved WPT. Their tendencies are quite different. In the improved WPT, no increase in internal resistance is recognized even after 2,000 days. It is assumed that the changes in internal resistance can be approximated by the exponential function shown in Figure 3.27, and that both the test model and the improved WPT can be approximated by the exponential curves shown in Figure 3.31. In the test model, changes in internal resistance were approximated well by the exponential curve. In the improved WPT, internal resistance appears to remain below $10\ \Omega$ after ten years based on the approximated functions.

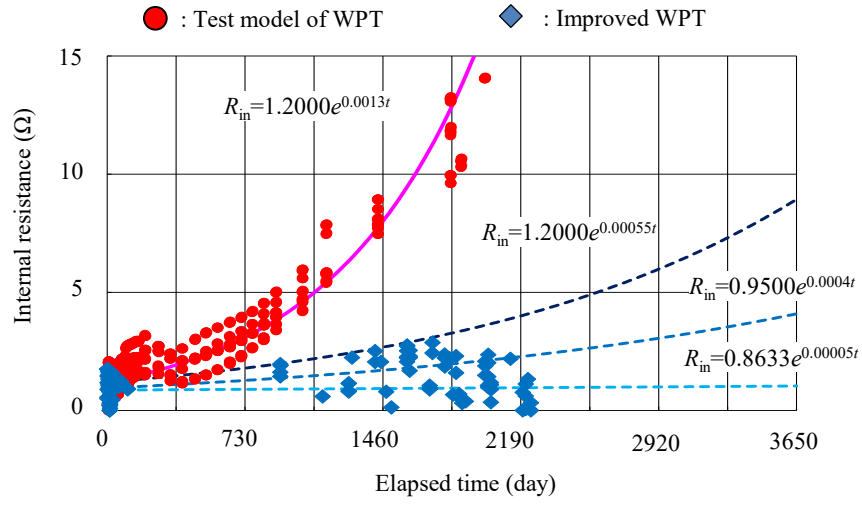


Figure 3.31 Comparison of internal resistance between test model and improved WPT

These results show that the effect of a drop in voltage induced by internal resistance can be modified with improvements, and that the long-term performance of the WPT can be advanced. It is expected that the improved WPT will perform properly for the anticipated ten-year working period.

Chapter 4 Influence of modeled area and material properties of dam bedrock on seismic behavior of dam in numerical analysis

4.1 Introduction

No dams which were constructed according to modern design and construction methods have ever been catastrophically damaged by earthquakes. From this fact, it is confirmed that modern design and construction methods can guarantee the seismic performance of dams. However, social awareness of earthquakes has increased; and thus, checks of the seismic performance of existing dams are being carried out nowadays to evaluate the health of the dams and to assess their safety against huge earthquakes.

A seismic response analysis by the finite element method (FEM) is one way to check the seismic performance of dams. A seismic response analysis is required to properly reproduce the real seismic behavior of dams in spite of the inherent restrictions on the numerical analysis (FEM). It is important to assess the various dams in Japan uniformly. A two-dimensional model, subject to a maximum cross section, is generally selected for this type of check, and the modeled bedrock is cut out from a semi-infinite earth foundation. For a seismic analysis by FEM, it is important that the modeled region and the boundary conditions be set properly so that the virtual boundaries will exert few negative influences on the seismic behavior and response characteristics of the dam. The effective boundary conditions developed by Lysmer and Kuhlemeyer (1969) and Miura and Okinaka (1989) are popularly utilized for seismic response analyses. For the modeled region, it is generally recommended that a sufficiently large area be set. Detailed examinations have been few, except for the study by the Architectural Institute of Japan (1996).

Ueda et al. (1994) and Shiojiri and Ueda (2000) investigated the effects of the modeled region in order to reproduce the actual seismic behavior of dams by a three-dimensional seismic response analysis. Ueda et al. (1994) recommended that the height of the bedrock be set to be the same as the height of the dam body. Shiojiri and Ueda (2000) stated that the height of the bedrock should be almost three times that of the dam. It is seen that their results for the modeled region differ from each other. Shiojiri and Ueda (1998) examined the effects of the elastic modulus and the damping ratio of the dam and the bedrock on the two-dimensional analysis, but the effects of the modeled region were not discussed. Nagayama et al. (1992) examined the effects of the material properties of the bedrock on the seismic characteristics of the dam and revealed that its maximum response value decreased according to the increase in the deformability of the bedrock. From these results, a certain tendency has been recognized for the effects of the modeled region and the material properties of the bedrock; however, its mechanism has not been clarified.

If the seismic response behavior of a dam and the bedrock is simplified and assumed as

one-dimensional shear behavior, the predominant frequency of the dam will depend on the dam's height and its material properties, density ρ and propagation velocity V (1/4 wave length low). It is independent of the material properties of the bedrock. The amplified ratio of the dam depends on the impedance ratio of the dam and the bedrock. The dam amplifies the vibration when its wave impedance (ρV) is smaller than that of the bedrock. And, the dam attenuates the vibration when its wave impedance is larger than that of the bedrock. However, such phenomena were not reported in previous studies, and it is supposed that the response behavior of the dam is affected by its multidimensional seismic mode.

By means of a two-dimensional response analysis, the manner in which the modeled region of the bedrock and its material properties affect the response behavior of the dam is examined. In particular, the amplitude ratio and the predominant frequency are investigated. The mechanism for the seismic behavior of the dam is considered based on the seismic response modes. In this study, a concrete gravity dam is targeted for a clear interpretation of the results, because its dam body and the bedrock are generally modeled as linear elastic materials. The material properties of the dam are set to be constant and those of the bedrock cover from the CM to A class bedrock, which are suitable for the bedrock of concrete gravity dams. The effect of the interaction between the dam and the bedrock is examined. And, the manner in which the dam affects the propagation characteristics of the bedrock is investigated. From the results, the important points are revealed in order to set the modeled region and the material properties of the bedrock in a seismic response analysis.

4.2 Conditions of numerical experiments

4.2.1 Models of dam and bedrock

Six models, which have different regions of bedrock, are selected as the models for the numerical experiments. The finite element mesh for the dam is shown in Figure 4.1. The height of the dam is 59.3 m and its bottom width is 56.6 m. The dam is divided into quadrilateral elements whose side length is about 2 m.

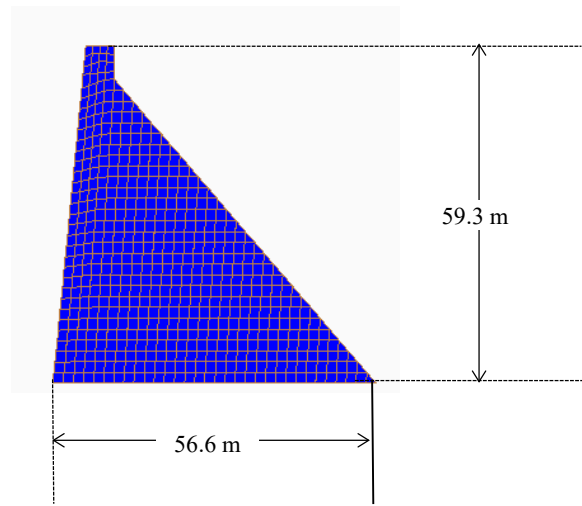


Figure 4.1 Finite element mesh of dam

Figure 4.2 shows the finite element meshes of all the models. The width of the bedrock is 512.0 m, which is about 9 times the bottom width of the dam, and its material property is homogeneous. The depth of each model is different; the depths of Model A, Model B, Model C, Model D, Model E, and Model F are 36.0 m, 72.0 m, 128.0 m, 168.0 m, 288.0 m, and 568.0 m, respectively. The ratios of the depth of the bedrock to the height of the dam for each model are 0.6, 1.2, 2.2, 2.8, 4.9, and 9.6 in order from Model A to Model F. Similarly, the ratios of the depth of the bedrock to the bottom width of the dam for each model are 0.6, 1.3, 2.3, 3.0, 5.1, and 10.0 for Model A to Model F. The bedrock is divided into quadrilateral elements, whose side length is about 2 m around the dam body. The side lengths of the elements become stepwise 4 m and 8 m apart from the dam.

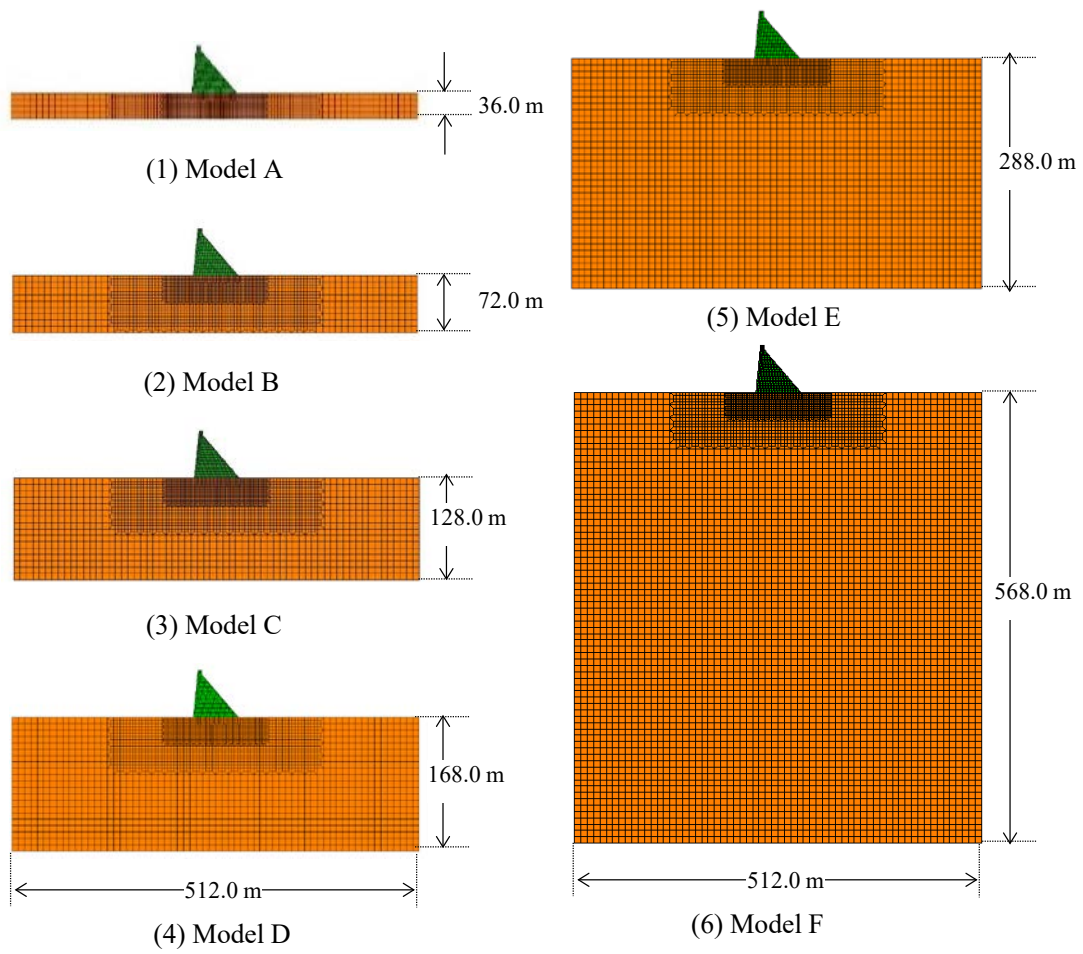


Figure 4.2 Finite element meshes of all the models

4.2.2 Numerical analysis conditions and input wave

In addition to the effects of the modeled region, the manner in which the material properties of the bedrock affect the response behavior of the dam is examined. Thus, several material properties are selected for the bedrock. Here, the material properties of the dam and the bedrock are set as the general values of concrete gravity dams referred to in previous studies (Hatano and Watanabe, 1970; Sato et al. 1997).

The material properties of the dam are an elastic modulus E of 28,000,000 kN/m², a Poisson's ratio ν of 0.20, and a unit volume weight γ of 23.0 kN/m³. Six material properties for the bedrock are set supposing A to B, CH, and CM class bedrock. The velocities of the elastic wave V_p of the bedrock in A to B, CH and CM class are supposed as above 3.7 km/s, 3.0 to 3.7 km/s, and 1.5 to 3.7 km/s, respectively, referring to Kikuchi et al. (1984). The material properties of the bedrock are shown in Table 4.1. The velocities of the elastic wave V_p of Mat 1, Mat 2, Mat 3, Mat 4, Mat 5, and Mat 6 are 1.7 km/s, 2.6 km/s, 3.3 km/s, 3.6 km/s, 4.1 km/s, and 4.9 km/s, respectively. The elastic moduli of the bedrock are calculated from the prescribed V_p , unit volume weight and Poisson's ratio, shown in Table 4.1. The damping ratios of the dam and the bedrock are set at 2% as the Rayleigh damping.

The boundary conditions of the seismic response analysis are set as the viscous boundary at the bottom and both sides of the bedrock in order to radiate the energy out of the modeled bedrock. The reserved water is not modeled in order to limit the target as the interaction between the dam and the bedrock for an easy interpretation of the effects of the modeled region of the bedrock on their response characteristics.

Table 4.1 Material properties of bedrock

	Rock classification	Shear modulus E (kN/m ²)	Possion's ratio ν	Unit weight γ (kN/m ³)	Velocity of shear wave V_s (m/s)
Mat 1	CM	5,102,000	0.25	20.0	1,000
Mat 2	CM	11,480,000	0.25	20.0	1,500
Mat 3	CH	22,530,000	0.20	23.0	2,000
Mat 4	CH	27,260,000	0.20	23.0	2,200
Mat 5	A ~ B	38,270,000	0.20	25.0	2,500
Mat 6	A ~ B	55,100,000	0.20	25.0	3,000

Figure 4.3 shows the time history of the acceleration and the acceleration Fourier spectrum of the input wave used in the numerical experiments. This wave is estimated by adjusting the amplitude of the seismic record at a dam site in the 2004 Niigataken-Chuetsu Earthquake (Masukawa et al. 2014). The acceleration Fourier spectrum is almost constant in the frequency range between 0.03 and 20 Hz. The maximum acceleration (absolute value) is 4.35 m/s^2 and the duration time is 40.96 s. The average value of the acceleration Fourier spectrum is $829 (\text{m/s}^2) \cdot \text{s}$ in the frequency range between 0.03 and 20 Hz. In Figure 4.3, the positive values indicate acceleration in the downstream, while negative values indicate those in the upstream. Hereafter, the signs for the acceleration indicate the direction in the same manner. The wave, shown in Figure 4.3, is inputted at the bottom of the viscous boundary.

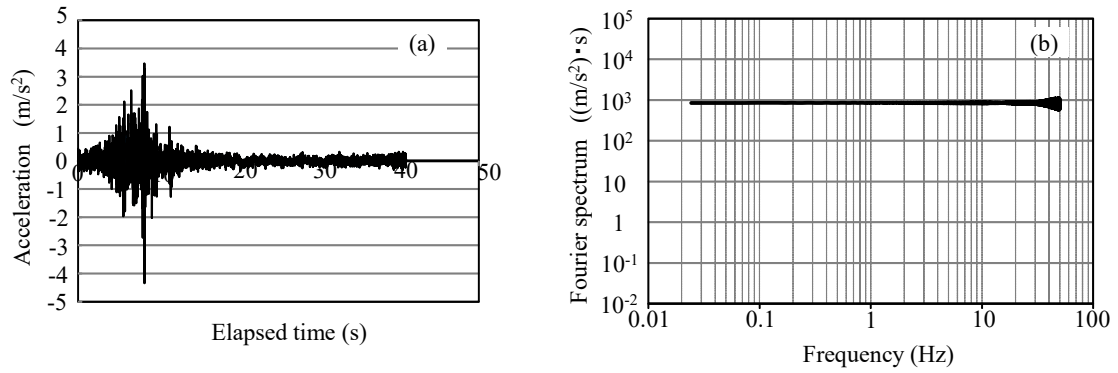


Figure 4.3 Input wave
(a) Acceleration, (b) Acceleration Fourier spectrum

4.3 Results and considerations

4.3.1 Acceleration response

Tables 4.2 and 4.3 show the amplitude ratios of the bedrock and the dam, respectively. The amplitude ratio of the bedrock is the maximum acceleration at the bottom of the dam divided by that of the input wave. Similarly, the amplitude ratio of the dam is the maximum acceleration at the dam's crest divided by that at the bottom of the dam. Comparing the amplitude ratio in the upstream direction (-) and that in the downstream direction (+) in the absolute value, the larger one is shown in these tables. The underlined values represent the amplitude ratios in the upstream direction, while the values that are not underlined represent the ratios in the downstream direction. Figures 4.4 and 4.5 show the time histories of the accelerations at the bottom and the crest of the dam, respectively.

Table 4.2 Comparison of amplitude ratios of bedrock

	Mat 1	Mat 2	Mat 3	Mat 4	Mat 5	Mat 6	Mean
Model A	0.27	0.34	0.44	0.44	0.43	0.48	0.40
Model B	0.26	0.36	0.44	0.44	0.47	0.54	0.42
Model C	0.22	0.29	0.35	0.36	0.37	0.41	0.33
Model D	0.19	0.25	0.30	0.32	<u>0.31</u>	0.39	0.29
Model E	0.14	<u>0.19</u>	0.21	0.22	<u>0.22</u>	<u>0.24</u>	0.21
Model F	0.09	0.12	0.14	0.16	0.17	0.19	0.15
Mean	0.20	0.26	0.31	0.32	0.33	0.37	

Table 4.3 Comparison of amplitude ratios of dam

	Mat 1	Mat 2	Mat 3	Mat 4	Mat 5	Mat 6	Mean
Model A	<u>2.57</u>	4.47	<u>4.94</u>	5.65	7.28	5.20	5.02
Model B	2.65	<u>3.66</u>	<u>5.05</u>	<u>6.07</u>	<u>7.46</u>	<u>7.61</u>	5.42
Model C	2.84	<u>3.82</u>	<u>4.84</u>	<u>5.85</u>	<u>7.79</u>	<u>6.94</u>	5.35
Model D	2.83	<u>3.77</u>	4.60	<u>5.61</u>	<u>6.64</u>	<u>6.18</u>	4.94
Model E	2.46	3.52	5.87	5.59	6.92	7.81	5.36
Model F	2.16	3.14	4.42	4.40	4.90	<u>5.55</u>	4.09
Mean	2.58	3.73	4.95	5.53	6.83	6.55	

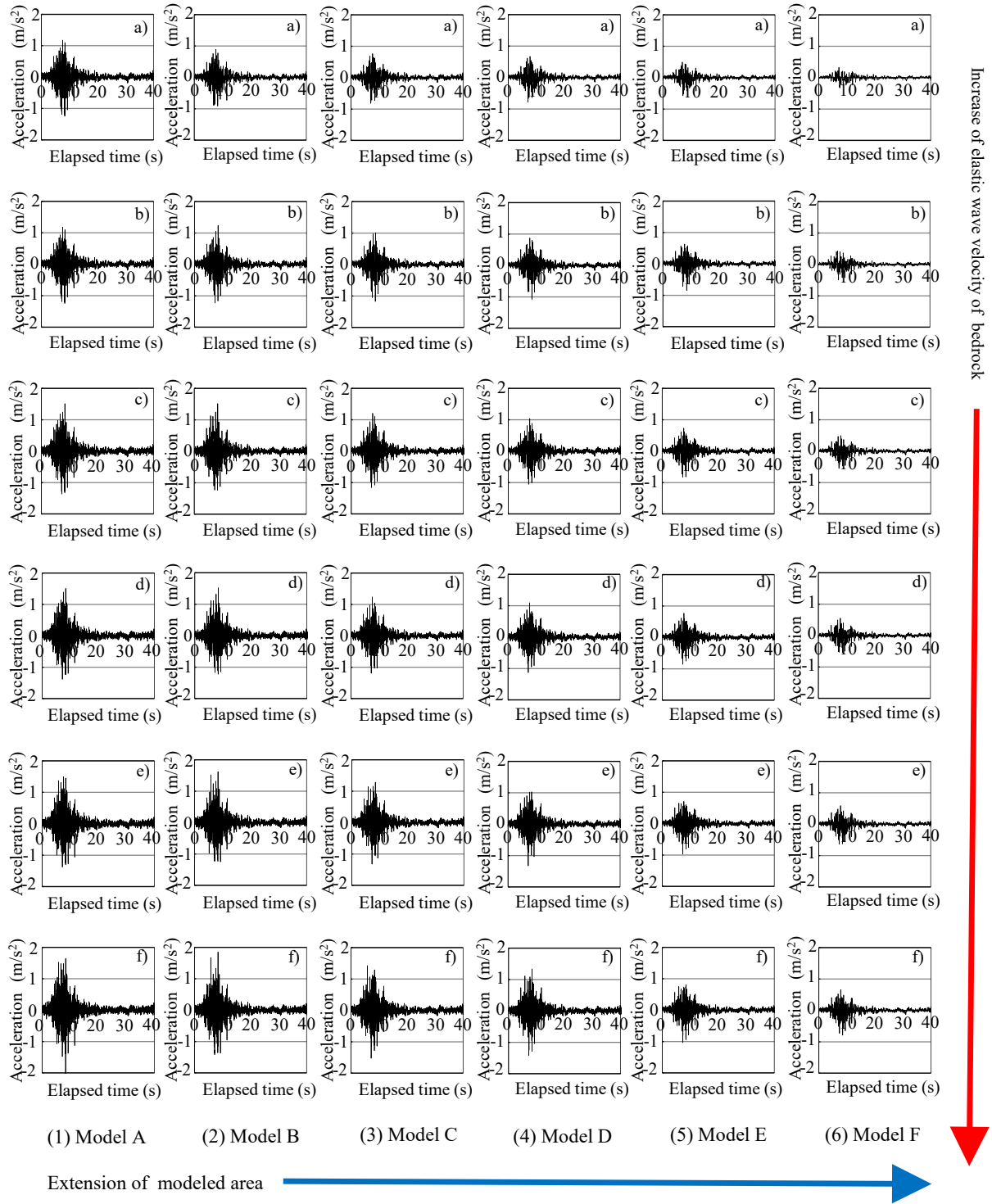


Figure 4.4 Time histories of accelerations at bottom of dam
a) Mat 1, b) Mat 2, c) Mat 3, d) Mat 4, e) Mat 5, f) Mat 6

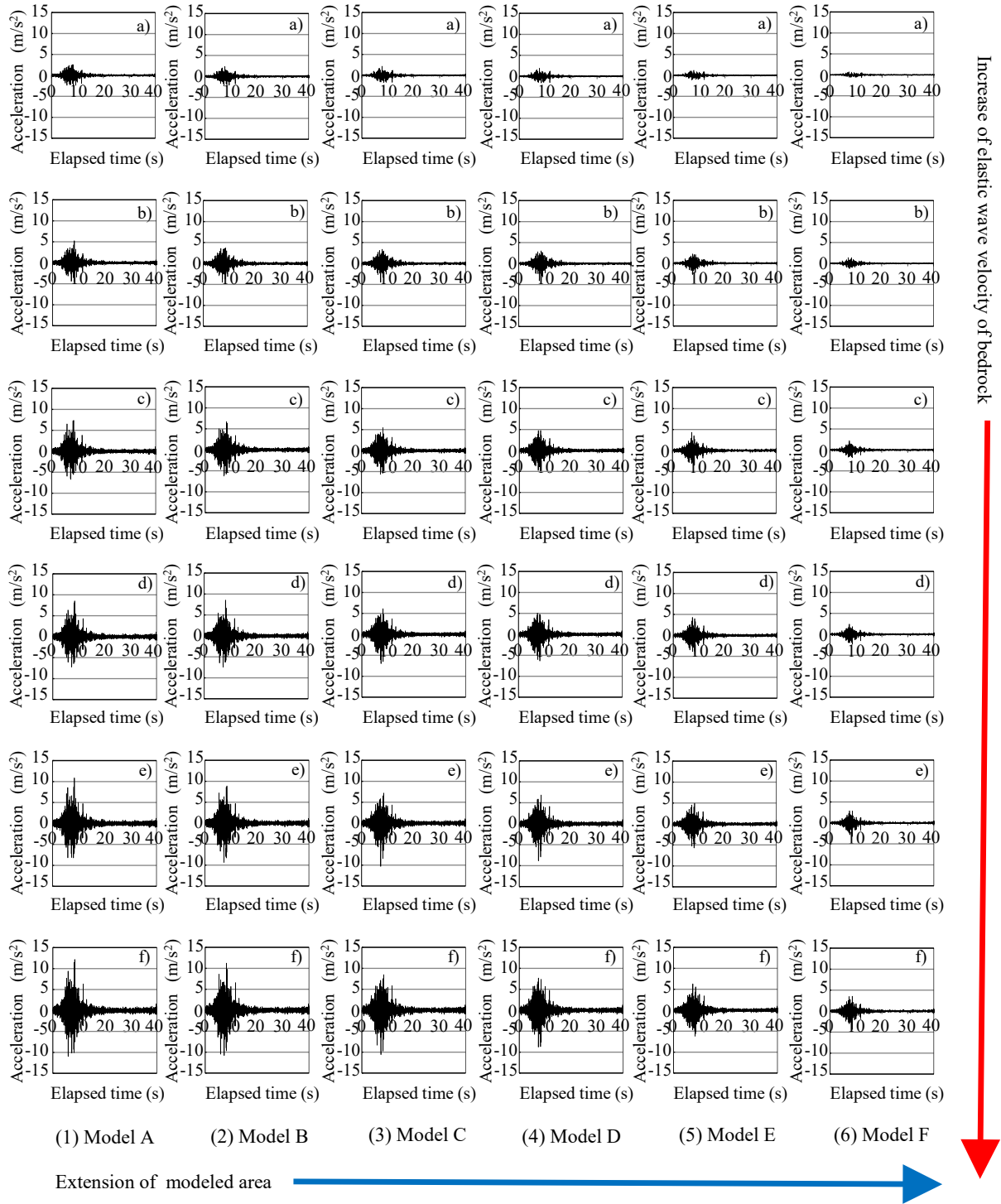


Figure 4.5 Time histories of accelerations at crest
a) Mat 1, b) Mat 2, c) Mat 3, d) Mat 4, e) Mat 5, f) Mat 6

As shown in Table 4.2 and Figure 4.4, the accelerations at the bottom of the dam and the amplitude ratios of the bedrock decrease in accordance with the enlargement of the modeled area and the depth of the bedrock. The amplitude ratios of the bedrock fall below 1.0 in all cases. In the case of a comparably soft deposit ground, the amplitudes of acceleration are an important issue in practice. However, the accelerations decrease according to the distance from the epicenter because of the attenuation of the seismic wave propagating in the stiff bedrock. From the results of the numerical experiments, it is considered that a seismic analysis can properly reproduce the attenuation phenomena according to the distance in the stiff bedrock. The attenuations become large and the maximum acceleration at the bottom of the dam body decreases according to the decrease in the elastic wave velocity of the bedrock. The elastic wave velocity of the bedrock becomes smaller in accordance with the lower quality of the bedrock.

The accelerations at the crest of the dam, shown in Figure 4.5, are amplified by the oscillation of the dam and become larger than those at the bottom of the dam, as shown in Figure 4.4. The accelerations at the crest decrease according to the enlargement of the modeled area, as shown in Figure 4.5, because the acceleration at the bottom of the dam decreases more by the attenuation of the seismic wave propagating in the stiff bedrock when the depth of the modeled area becomes deeper (Table 4.2 and Figure 4.4). From Table 4.3, the amplitude ratios of the dam are remarkably influenced by the material properties of the bedrock, but are less influenced by the region of the modeled area. When the material property of the bedrock is Mat1, the averaged amplitude ratios of the dams are about 2.6. Similarly, the averaged amplitude ratios of the dam for Mat 2, Mat 3, Mat 4, Mat 5, and Mat 6 are almost 3.7, 5.0, 5.5, 6.8, and 6.6, respectively. It is concluded that the amplitude ratios of the dam increase according to the increase in the elastic wave velocity of the bedrock, because the material properties of the dam are constant in all cases.

From these results, it is confirmed that the modeled region affects the results of the response analysis as the attenuation effects the distance in the bedrock, and that the amplitude ratio of the dam is influenced remarkably by the material properties of the bedrock and less by the modeled region.

4.3.2 Predominant frequency of dam

The acceleration Fourier spectrum ratios at the crest and the dam bottom are compared to examine the effects of the modeled region and the material properties of the bedrock on the frequency response characteristics of the dam, and the results are shown in Figure 4.6. Hereinafter, the acceleration Fourier spectrum ratio is referred to as the Fourier spectrum ratio.

The Fourier spectrum ratio generally indicates several peaks; the peak at the lowest frequency is the 1st peak and its frequency is referred to as the 1st predominant frequency. Similarly, the next peak of the Fourier spectrum ratio is the 2nd peak and its frequency is referred to as the 2nd predominant frequency. From Figure 4.6, it is clarified that the 1st predominant frequencies of the dam are almost the same for the same material properties of the bedrock in spite of the difference in the modeled regions. However, the Fourier spectrum ratios at the 1st peak are different and they are influenced by the modeled regions.

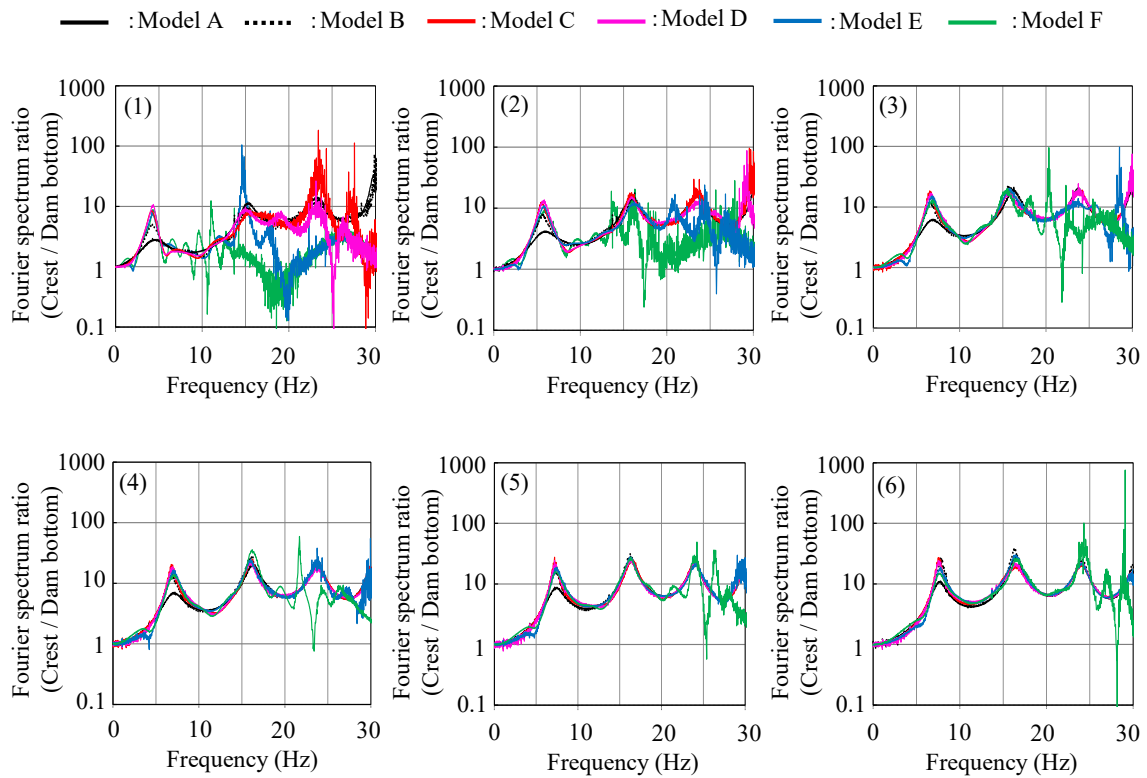


Figure 4.6 Changes in Fourier spectrum ratio by modeled area and material properties of bedrock
(1) Mat 1, (2) Mat 2, (3) Mat 3, (4) Mat 4, (5) Mat 5, (6) Mat 6

Table 4.4 shows the frequencies and the Fourier spectrum ratios at the 1st and the 2nd peaks. The averaged 1st predominant frequency in each model is almost the same when the material properties of the bedrock are the same. The averaged values for the 1st predominant frequency in Mat 1, Mat 2, Mat 3, Mat 4, Mat 5 and Mat 6 are 4.2 Hz, 5.8 Hz, 6.7 Hz, 7.0 Hz, 7.3 Hz, and 7.7 Hz, respectively, and little difference is seen among the models.

On the other hand, the Fourier spectrum ratios at the 1st peak change according to the modeled region and the material properties of the bedrock; the Fourier spectrum ratio becomes larger when the elastic wave velocity becomes higher. As for the effects of the modeled region, the Fourier spectrum ratios show the maximum in Model C and Model D, while that in Model A, which has a shallow bedrock, is comparably lower than the other models.

Table 4.4 Comparison of predominant frequencies and Fourier spectrum ratios of dam

		Mat 1	Mat 2	Mat 3	Mat 4	Mat 5	Mat 6
Model A	Freq.	4.2	5.9	6.9	7.2	7.2	7.8
	(Hz)	(15.5)	(15.8)	(16.3)	(16.3)	(16.3)	(16.3)
	Ratio	2.9	4.2	6.3	7.1	8.8	11.2
		(4.2)	(15.0)	(21.0)	(22.8)	(26.2)	(29.3)
Model B	Freq.	4.3	5.6	6.7	6.9	7.2	7.9
	(Hz)	(15.2)	(16.3)	(15.7)	(16.3)	(16.3)	(16.3)
	Ratio	5.0	8.0	12.1	14.1	18.5	26.6
		(9.8)	(17.8)	(22.8)	(27.4)	(33.3)	(40.4)
Model C	Freq.	4.2	5.6	6.5	6.8	7.2	7.5
	(Hz)	(15.5)	(15.9)	(15.7)	(16.3)	(16.3)	(16.3)
	Ratio	8.5	12.5	18.4	20.4	27.5	27.0
		(8.8)	(18.0)	(18.4)	(25.0)	(27.1)	(20.2)
Model D	Freq.	4.3	5.8	6.7	6.9	7.2	7.5
	(Hz)	(15.0)	(15.9)	(15.6)	(15.9)	(16.3)	(16.3)
	Ratio	10.7	13.6	17.0	18.7	24.3	24.2
		(9.7)	(15.3)	(17.8)	(22.8)	(27.1)	(22.9)
Model E	Freq.	4.2	5.9	6.8	7.1	7.4	7.8
	(Hz)	(14.6)	(16.3)	(15.8)	(16.0)	(16.2)	(16.4)
	Ratio	8.4	10.9	15.3	16.6	18.9	18.8
		(104.4)	(12.6)	(20.3)	(25.0)	(27.9)	(30.0)
Model F	Freq.	4.2	5.8	6.8	7.1	7.4	7.7
	(Hz)	(11.0)	(13.6)	(15.5)	(16.2)	(16.3)	(16.5)
	Ratio	8.8	9.3	13.1	13.8	15.8	15.0
		(12.4)	(18.9)	(20.2)	(35.8)	(27.5)	(26.3)

Ratio: Fourier spectrum ratio (): 2nd predominant peak

As for the whole tendency of the Fourier spectrum ratio until 20 Hz, the effects of the modeled region on the Fourier spectrum ratio (crest/bottom) are small in CM (Mat 3 and Mat 4) and A to B (Mat 5 and Mat 6) class bedrock, which have a comparably high elastic wave velocity. In the case of Mat 1, however, which has low elastic wave velocity, the effects of the modeled region are remarkable. In particular, in model F, which has the deepest bedrock, several peaks arise at the low frequency range between 5 and 10 Hz, and it shows a different tendency from the other models.

From these results, it is clarified that the Fourier spectrum ratios (crest/bottom) at the 1st peak and the 1st predominant frequency are influenced remarkably by the material properties of the bedrock. The 1st predominant frequency of the dam shifts to a higher frequency, and the Fourier spectrum ratio at the 1st predominant frequency becomes larger when the elastic wave velocity becomes higher. These results coincide with those in the studies by Nagayama et al. (1992) and Ueda et al. (1994). The modeled regions have little effect on all the tendencies of the Fourier spectrum ratio when the elastic wave velocity is sufficiently high. On the other hand, the modeled regions influence all the tendencies of the Fourier spectrum ratio when the elastic wave velocity is low. This tendency is especially remarkable in the model with the deeper bedrock. It is clarified that the 1st predominant frequencies of the dam show almost the same value, in spite of the difference in the modeled regions, when the material properties of the bedrock are constant. These results coincide with those of Ueda et al. (1994) and Shiojiri and Ueda (2000). The Fourier spectrum ratios at the 1st peak are influenced by the modeled regions. It is recognized that the Fourier spectrum ratio at the 1st peak becomes the largest when the depth of the bedrock is two or three times the height of the dam for the material properties employed in this study.

4.3.3 Seismic modes of each model

It is clarified that the amplitude ratio and the predominant frequency depend more directly on the material properties of the bedrock than on the modeled regions. Moreover, when the elastic wave velocity of the bedrock becomes higher, the acceleration amplitude ratio of the dam and its Fourier spectrum ratio at the 1st predominant frequency become larger. Then, the primary seismic modes estimated by the eigenvalue analysis are compared, and the mechanism of the revealed seismic behavior is examined. Figure 4.7 shows the primary seismic deformation modes under all conditions. In this figure, the shapes of the primary seismic modes are shown near the dam body, and the scale of the deformation is magnified 20 times for an easy observation of these seismic modes. This figure shows the primary eigenfrequencies at which the primary seismic deformation modes appear.

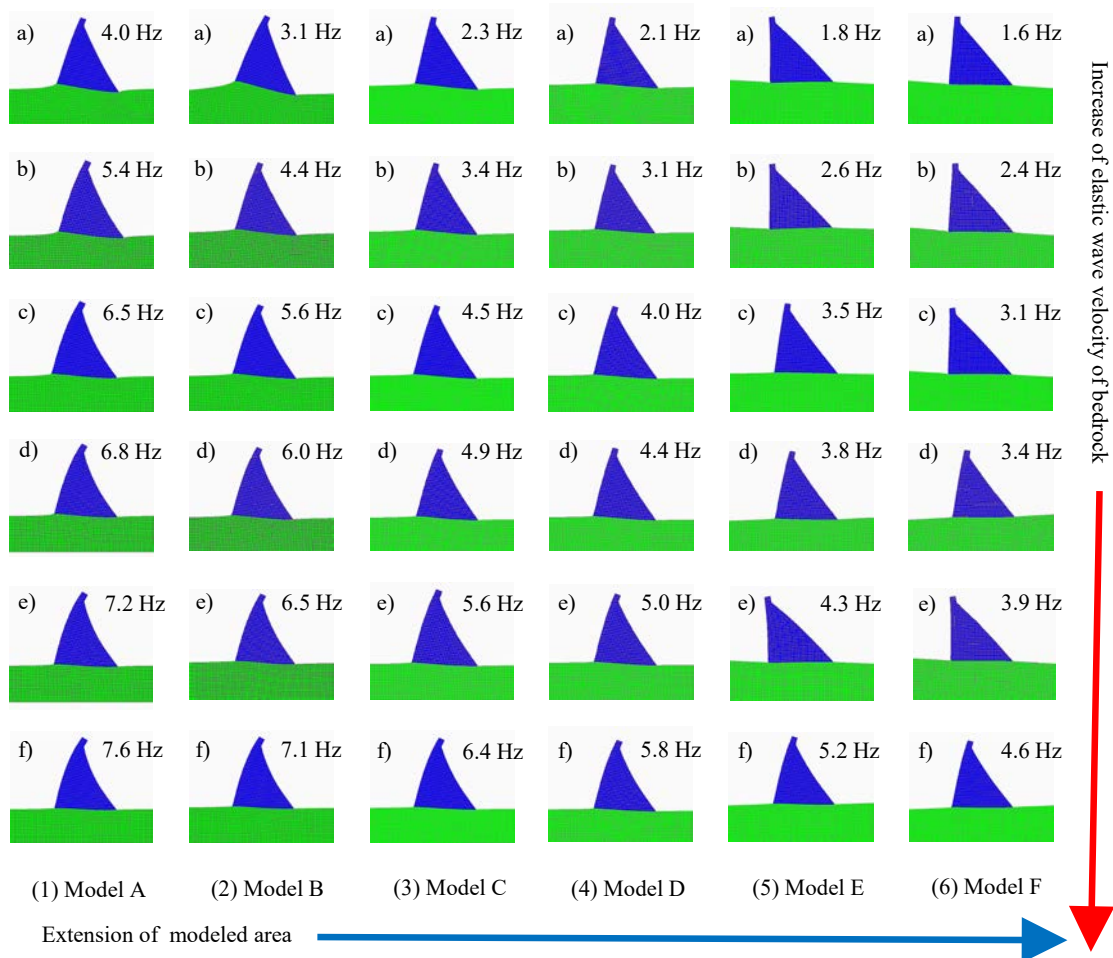


Figure 4.7 Comparison with primary seismic modes in all models

a) Mat 1, b) Mat 2, c) Mat 3, d) Mat 4, e) Mat 5, f) Mat 6

It is recognized that the deformation of the bedrock is remarkable and the dam behaves as a rigid body in the case of Mat 1, whose elastic wave velocity is the lowest. The deformation of the dam becomes more remarkable according to the increase in the elastic wave velocity, and the dam shows the seismic mode as bending. It is clarified that the seismic mode of the dam shows almost the same pattern in spite of the different modeled regions. On the other hand, the primary eigenfrequency is influenced by the modeled region and the material properties of the bedrock, because most of the model is occupied by the bedrock. Thus, the primary eigenfrequency becomes lower according to the expansion of the modeled area, and it becomes higher according to the increase in the elastic wave velocity. However, the shapes of the 1st seismic mode are almost the same, in spite of the different modeled regions, when the material properties of the bedrock are constant. The seismic mode of the dam changes according to the elastic wave velocities of the bedrock. These tendencies coincide with the results of the aforementioned experiments. It is supposed that the seismic behavior of the dam arises by the interaction with the bedrock, and such interaction mainly depends on the material properties of the dam and the bedrock. When the elastic wave velocity of the bedrock is low, the 1st predominant frequency of the dam is lower and its Fourier spectrum ratio is smaller, because the dam shows the seismic mode like a rigid body. And, the elastic wave velocity of the bedrock becomes higher, the 1st predominant frequency of the dam shifts to be higher, and its Fourier spectrum ratio becomes larger, because the seismic mode of the dam body changes, which shows remarkable deformation like bending.

4.3.4 Influence of dam on propagation characteristics of bedrock

The propagation characteristics of the bedrock with the dam (models shown in Figure 4.2) and without the dam (only bedrock) are compared to examine the influence of the dam on the bedrock. The index for the comparison is the ratio of the acceleration Fourier spectrums at the bottom of the bedrock and its surface (bottom of the dam when the dam exists).

Table 4.5 shows the 1st predominant frequencies of the bedrock. From this table, it is revealed that the 1st predominant frequency of the bedrock becomes higher according to the increase in elastic wave velocity in all the models. Its 1st predominant frequency becomes lower according to the extension of the bedrock depth, regardless of the existence of a dam body, when the material properties of the bedrock are constant.

Figures 4.8 to 4.10 show the results of all the models in each material of the bedrock. From Figure 4.8, it is revealed that the dam affects the whole tendency of the Fourier spectrum ratio (surface of the bedrock/its bottom), the frequency at the 1st predominant peak, and the Fourier spectrum ratio at the 1st peak, when the depth of the bedrock is relatively small, like Model A (0.6 times the dam height) and Model B (1.2 times the dam height). On the other hand, in the case of the others (Figures 4.9 and 4.10), the Fourier spectrum ratios show almost the same tendency regardless of the existence of the dam. In particular, the 1st predominant frequency and the Fourier spectrum ratio at the 1st peak are identical.

Table 4.5 Comparison of influences of dam on 1st predominant frequency of bedrock

		Mat 1	Mat 2	Mat 3	Mat 4	Mat 5	Mat 6
Model A	with dam	7.1	9.9	12.2	12.9	13.9	19.6
	no dam	7.7	11.4	14.6	15.9	17.7	20.3
Model B	with dam	3.1	6.1	7.6	8.2	8.9	10.3
	no dam	3.6	5.3	7.2	7.9	8.8	10.4
Model C	with dam	1.9	2.8	3.7	4.2	4.7	5.6
	no dam	2.0	2.9	3.9	4.3	4.9	5.8
Model D	with dam	1.5	2.3	3.1	3.4	3.9	4.5
	no dam	1.6	2.3	3.1	3.4	3.9	4.7
Model E	with dam	1.0	1.4	1.9	2.1	2.4	2.8
	no dam	1.0	1.4	1.9	2.1	2.4	2.8
Model F	with dam	0.5	0.7	1.0	1.1	1.3	1.4
	no dam	0.5	0.7	1.0	1.1	1.3	1.4

Unit: Hz

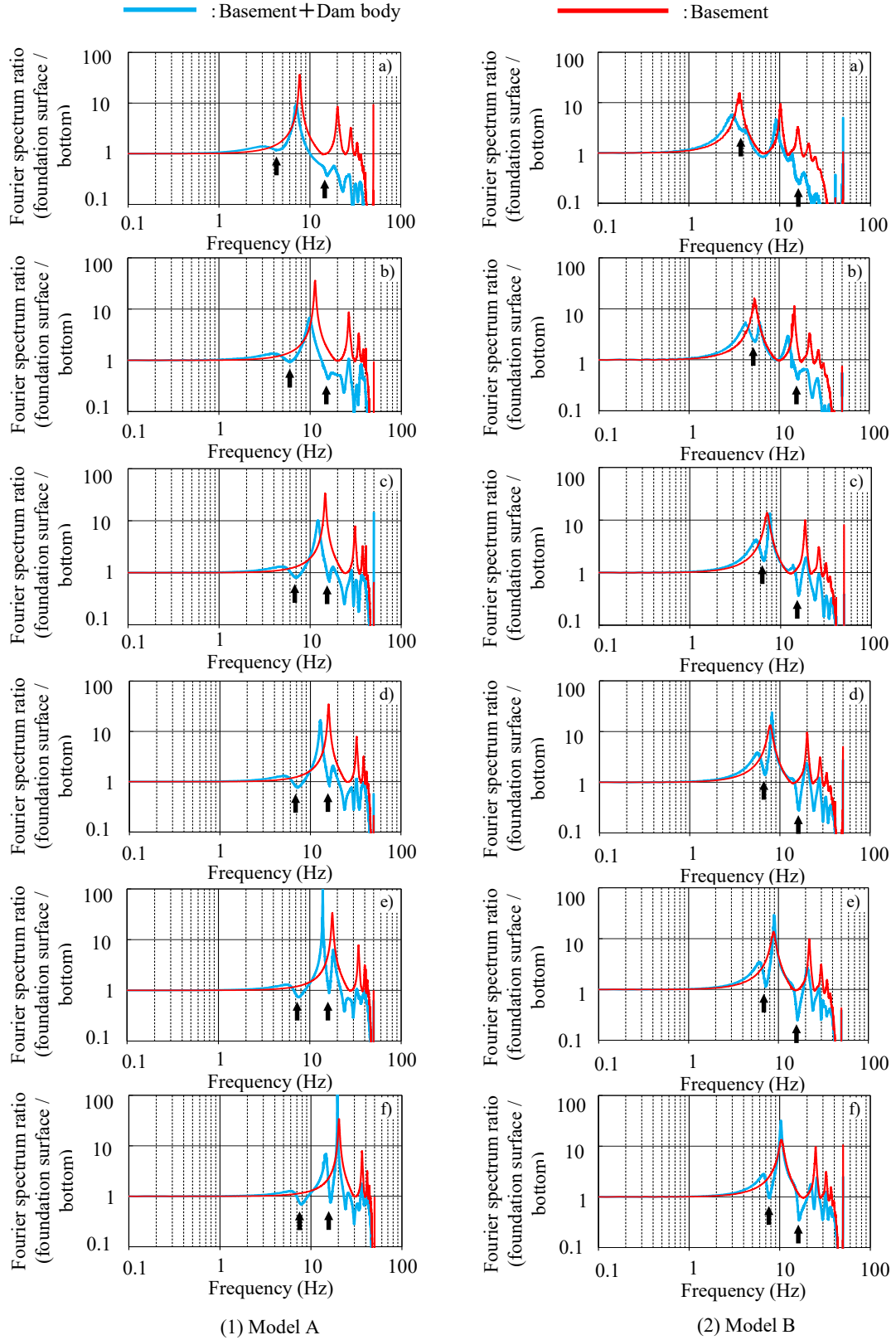


Figure 4.8 Change in Fourier spectrum ratio by existence of dam (Model A and Model B)

a) Mat 1, b) Mat 2, c) Mat 3, d) Mat 4, e) Mat 5, f) Mat 6

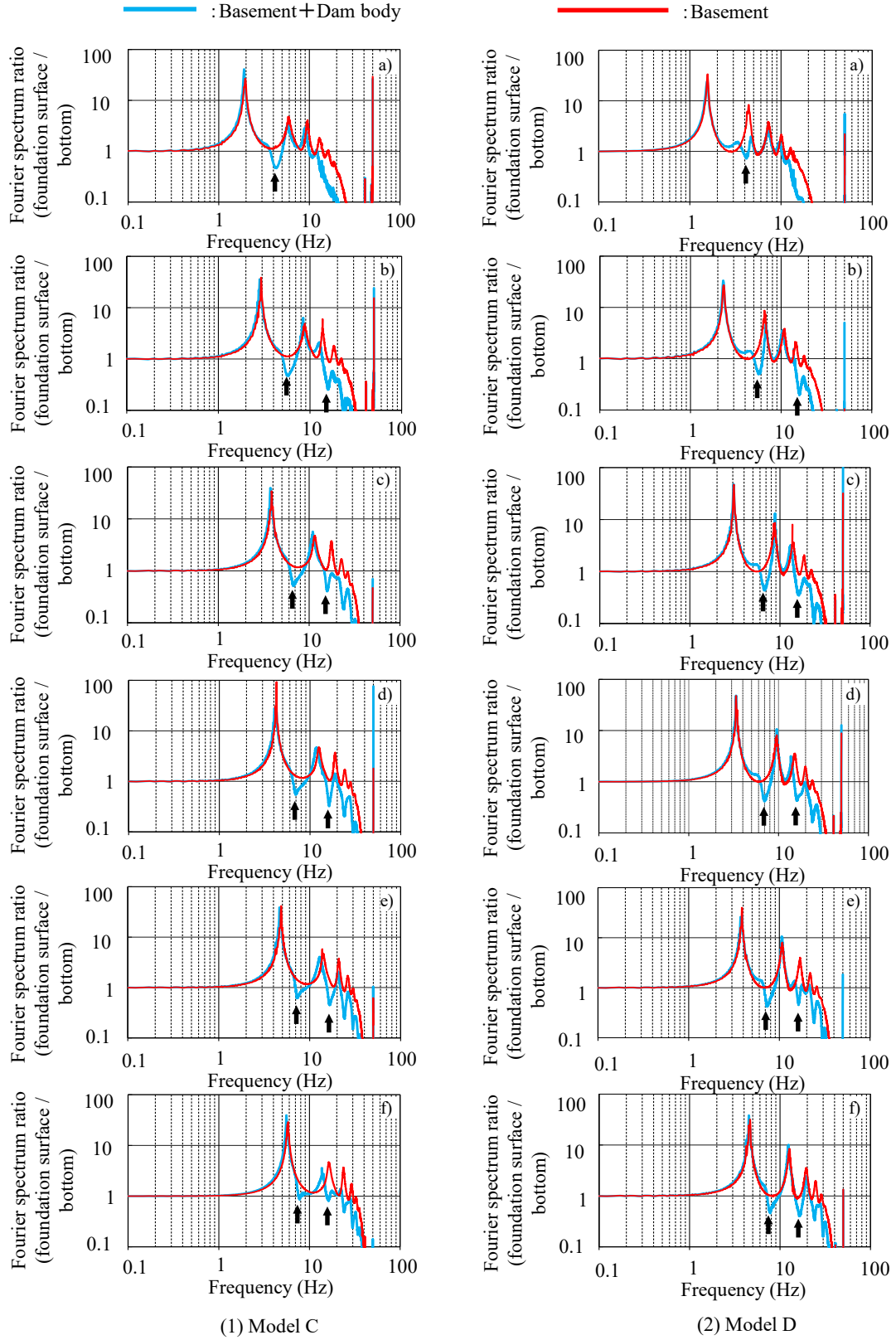


Figure 4.9 Change in Fourier spectrum ratio by existence of dam (Model C and Model D)

a) Mat 1, b) Mat 2, c) Mat 3, d) Mat 4, e) Mat 5, f) Mat 6

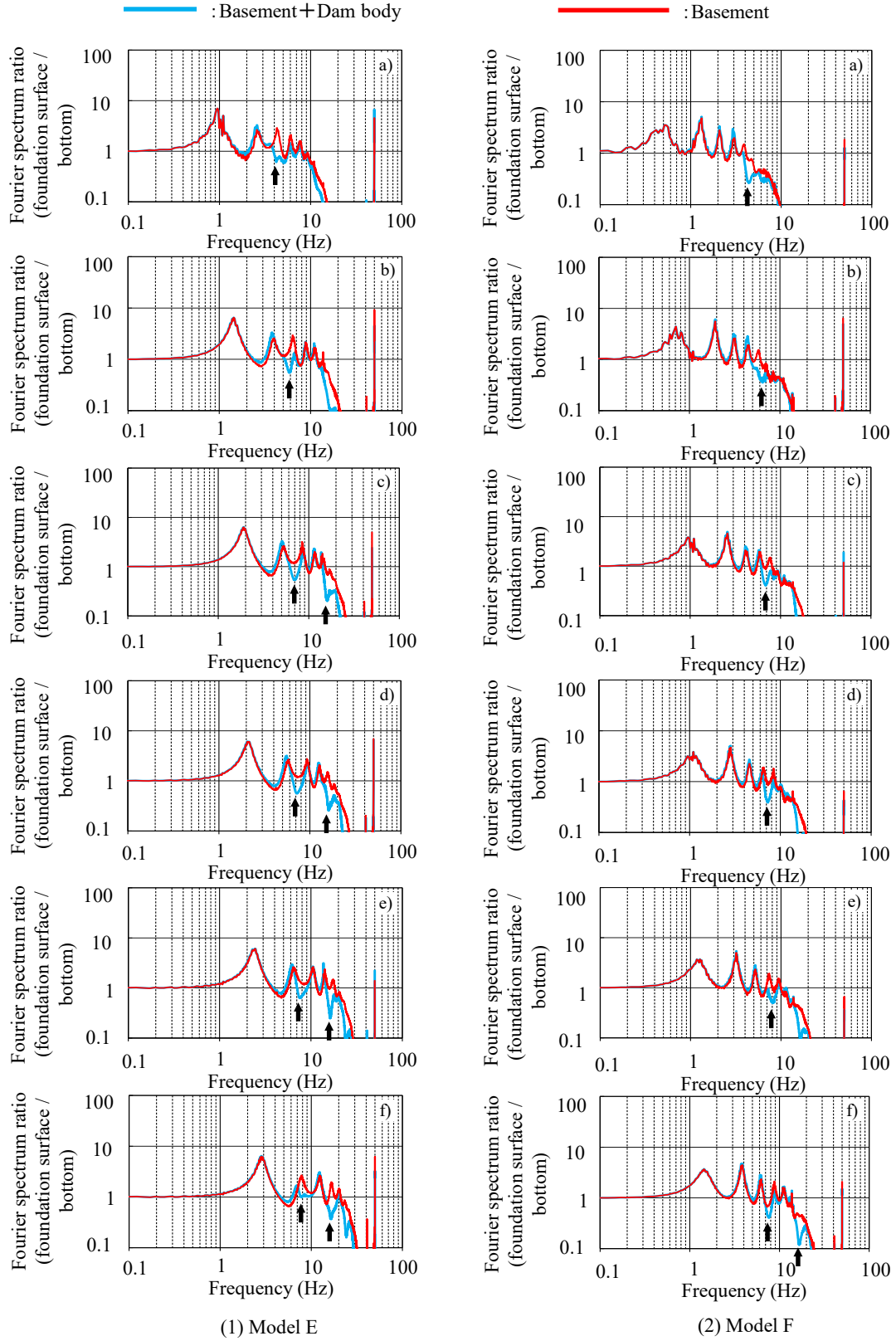


Figure 4.10 Change in Fourier spectrum ratio by existence of dam (Model E and Model F)

a) Mat 1, b) Mat 2, c) Mat 3, d) Mat 4, e) Mat 5, f) Mat 6

However, there are frequency ranges at which the tendency of the Fourier spectrum ratio differs according to the existence of the dam body in Models C, D, E, and F, as shown in Figures 4.9 and 4.10. It is recognized that the Fourier spectrum ratio decreases rapidly and its value drops below 1.0 due to the existence of the dam body at the specified frequency ranges. The dam has a remarkable influence on the Fourier spectrum ratio of the bedrock at these frequencies. Thus, the mutual interaction between the dam and the bedrock is supposed as the main factor which influences the propagation characteristics (Fourier spectrum ratio) of the bedrock. It is considered that the interaction between the dam and the bedrock becomes remarkable at the frequencies of the inflection points, indicated as arrows in Figures 4.8 to 4.10.

Figure 4.11 shows the relation between the frequencies of these inflection points and the peaks of the Fourier spectrum ratio of the dam (crest/dam bottom), shown in Figure 4.6 and Table 4.4. It is revealed that the frequencies, at which the dam more significantly affects the response characteristics of the bedrock, correspond with the 1st or 2nd predominant frequencies of the dam regardless of the modeled region or the material properties of the bedrock. It is concluded that the propagation characteristics of the bedrock vary with the mutual interaction between the dam and the bedrock, and that these effects become the most remarkable at the predominant frequency of the dam.

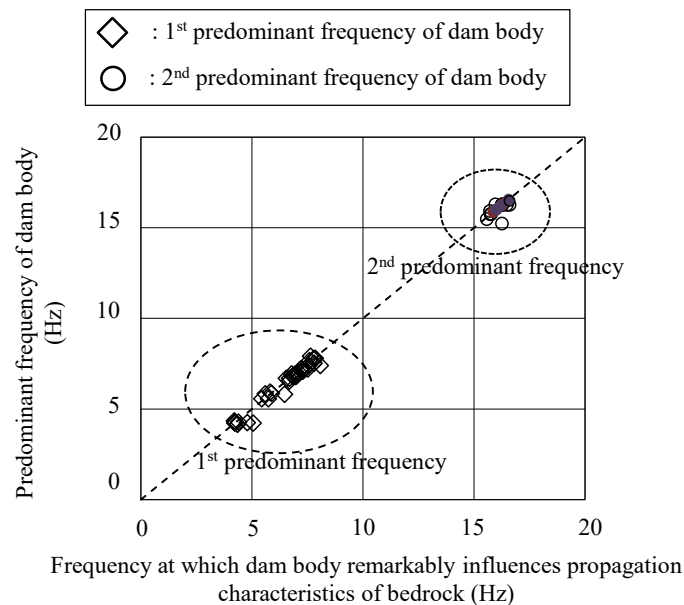


Figure 4.11 Relation between frequencies of inflection points and predominant frequency of dam body

There is little difference in the predominant frequencies of the bedrock according to the existence of a dam body, and the Fourier spectrum ratios show similar shapes around the 1st predominant frequency in Models C, D, E, and F where the depths of the bedrock are above 2.2 times the dam height. However, the difference becomes remarkable in Models A and B where the depths of the bedrock are shallower. It is thought that the predominant frequencies of the bedrock are remarkably influenced by the dam in Models A and B, because the 1st predominant frequencies of the bedrock are close to the 1st or 2nd predominant frequencies of the dam. Thus, the modeled region affects the Fourier spectrum ratio of the bedrock with the dam around the 1st peak.

From these results, it is revealed that the propagation characteristics of the bedrock are influenced by the dam, especially at the predominant frequency of the dam body. Therefore, it is important to set the modeled area to be large enough that the 1st predominant frequency of the bedrock does not approach the predominant frequency of the dam. It is concluded that the depth of the bedrock needs to be more than 2.2 times the dam height in order to satisfy such conditions under the material properties of the dam and the bedrock described in this study.

Chapter 5 Destructive behavior of dam body induced by liquefaction of its foundation

5.1 Centrifugal tests for liquefaction of dam foundation

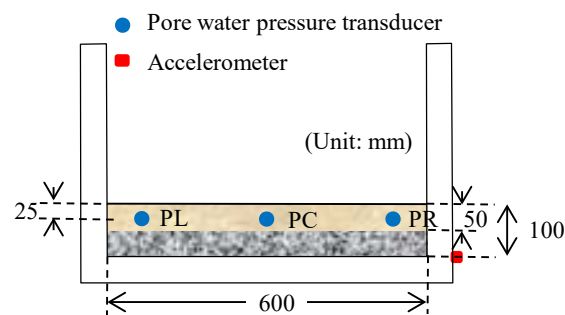
5.1.1 Purpose

In Japan, huge earthquakes have been occurring frequently, for example, the 2011 off the Pacific coast of Tohoku Earthquake, the 2016 Kumamoto Earthquake and so on. There are about 1,800 irrigation fill dams including earth dams that were constructed before 1956 when modern design methods were established for the construction of such dams. Tani and Hori (1998) reported that there are 1,208 fill dams above 15m in height that were constructed before 1960, and they account for almost 70% of the total number of irrigation fill dams. It has now been over 50 years since these dams were constructed and first put into operation. It is supposed that these dams were constructed without the presently available modern design methods or scientific knowledge of liquefaction. Thus, their reliability in terms of seismic safety is understandably lower than that of dams constructed with modern methods. Based on these facts, fundamental scientific knowledge about the seismic behavior of such dams is very important for checking their seismic performance. Moreover, some of the fill dams that have been in operation for a long time are in need of repair. In particular, it is necessary to repair and retrofit the dams which are important to agricultural activities, to protect them against secondary disasters induced by bursting, and to increase their seismic performance.

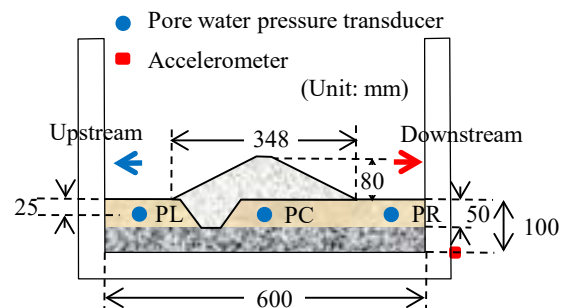
The construction of an impervious zone on the upper slope of the dam body is frequently adopted as a countermeasure against leakage and as reinforcement in the repair of irrigation fill dams (in particular, earth-fill dams), which encounter the problems of leakage and the loss of the cross section by aging (Fukushima et al., 2004; Tani and Fukushima, 2007; Fukushima et al., 2012). The purpose of this study is to acquire fundamental scientific knowledge on the deformation and fracture behavior of dams with inclined core zoning, especially in view of the liquefaction of their foundations, which is one of the principal factors in the large deformations of dams during earthquakes. Then, centrifugal liquefaction experiments are carried out to examine the fracturing mechanism of dams that develops due to the interaction among the liquefiable foundation, the dam body, and the reserved water.

5.1.2 Experimental conditions

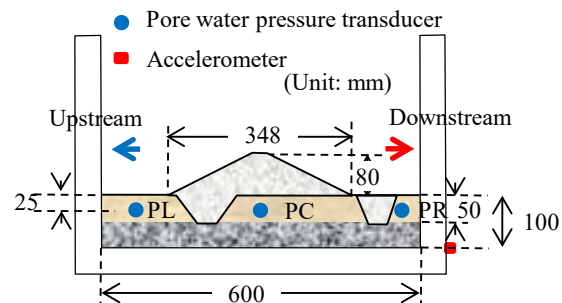
Three models are used in the experiments to compare the behavior of the above-mentioned dams. One is a model that imitates only the liquefiable foundation; it is called Case A. The second is a model that imitates the liquefiable foundation and the dam body with inclined core zoning; it is called Case B. The third is a model that imitates the retrofitting condition, in which the toe of the downstream slope in Case B is dug and refilled with non-liquefiable materials; it is called Case C. The schematics of these models are shown in Figure 5.1.



(a) Case A



(b) Case B



(c) Case C

Figure 5.1 Schematics of models

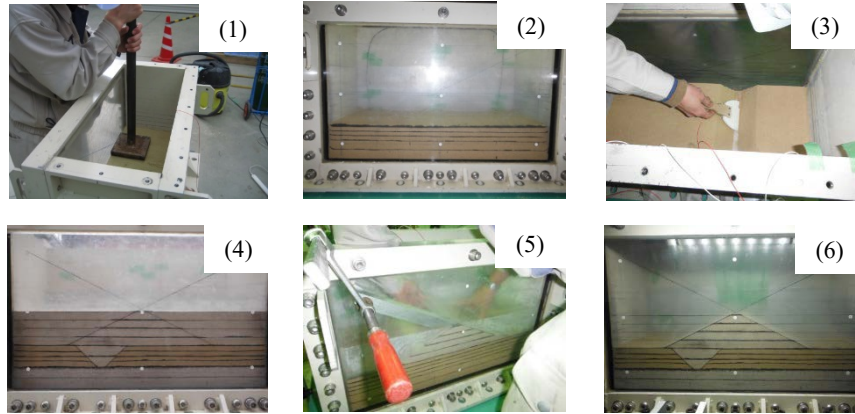


Figure 5.2 Procedures for making each model

The thickness of the liquefiable foundation of each model is 50 mm. A basement is arranged below the liquefiable foundation in order to moderate the influence of the bottom boundary of the soil box on the behavior of the liquefiable foundation. In Cases B and C, the dam bodies are constructed on a liquefiable layer. Their height is 80 mm, the width of their crest is 28 mm, and their slope gradient is 1:2. The slope gradient of the core trench is 1:1. In Case C, the slope gradient of the improved zone at the toe of the downstream slope is 1:0.4. In each model, pore water pressure transducers are set 25 mm below the surface of the liquefiable foundation, and an accelerometer is set at the bottom of the soil box. In the centrifugal tests, the centrifugal acceleration is set at 200 m/s^2 , the height of the estimated prototype of the dam body is 1.6 m, and the depth of the estimated prototype of the liquefiable foundation is 1.0 m.

The procedure for making each model is shown in Figure 5.2. The basement is made of coarse silica sand (Keisya No.4) by tamping under a dry condition. The liquefiable foundation on the basement is made of fine silica sand (Toyoura sand) by tamping with a water content of 5%, and its relative density (D_r) is 50%. The density of the liquefiable foundation is controlled by checking the material weight dropped into the soil box and the height of the compacted soil layer. Figure 5.2 (1) shows the situation of compacting the material by hand, while Figure 5.2 (2) shows an overview of the compacted soil layers as the liquefiable foundation. Before making the dam body, the prescribed area for the liquefiable foundation is dug and shaped into a core trench (Figure 5.2 (3)). The dam body and the core trench are made by tamping the mixture of fine sand (Toyoura sand) and kaolin at a mixing ratio of 8:2. The water content of the mixture is 11%, and the degree of compaction (D_c) of the dam body and the core trench is 92%. The degree of compaction is determined from the results of trial tests in which the density of the liquefiable foundation does not change according to the compaction of the mixture. The density control method, used for the degree of compaction, is the same as that of the liquefiable foundation. Figure 5.2 (4) shows an overview of the compacted

material of the dam body. Here, black colored sand layers are arranged along the inner surface of an acrylic board in the liquefiable foundation and the dam body for easy observation of the deformed model during the experiments. The dam body is shaped by cutting the compacted soil mixture layer, as shown in Figure 5.2 (5). Figure 5.2 (6) presents an overview of the completed model.

The shapes of the completed models are measured by a laser displacement meter in order to compare them with the deformed shapes after the experiments (Figure 5.3). Figure 5.4 shows the procedure for pouring the pore fluid into the liquefiable foundation. The method proposed by Okamura and Inoue (2012) is used as reference for saturating the liquefiable foundation. At first, CO₂ gas is allowed to fully flow into the liquefiable foundation from the bottom of the soil box at the vacuum state. Then, the soil box is kept at the vacuum state and set on the centrifugal equipment. The solution of cellulose ester (viscous fluid) is poured into the liquefiable foundation from the bottom of the soil box under the centrifugal condition by the difference of the hydraulic head between the bottom of the vessel of viscous fluid, set on the soil box, and that of the liquefiable foundation. In general, the viscosity of the pore fluid is arranged as n times the water in the liquefaction experiments under centrifugal acceleration with n times the gravity (Kazama and Inatomi, 1993). A centrifugal acceleration of 200 m/s² is loaded in these liquefaction experiments, and the viscosity of the solution of cellulose ester is then adjusted to 20 cSt. The adding of the viscous fluid is stopped when it is recognized by a camera, set in front of the model, that the viscous fluid has welled up slightly on the surface of the liquefiable foundation. After that, the centrifugal acceleration is unloaded. Then, it is gradually reloaded up to 150 m/s², 170 m/s², and 200 m/s². At each stage, it is confirmed that the pore water pressure increases corresponding to the increase in centrifugal acceleration. From these results, it is supposed that the liquefiable foundation is sufficiently saturated. After unloading the centrifugal acceleration, the vessel, set on the soil box, is removed before the liquefaction test. In the test cases with a dam body (namely, Cases B and C), the solution of cellulose ester with higher viscosity is poured as the reserved water. Its viscosity is adjusted to 2,000 cSt, because the load of the reserved water is focused on, and the effects of the seepage into the dam body and the liquefiable foundation can be inhibited.

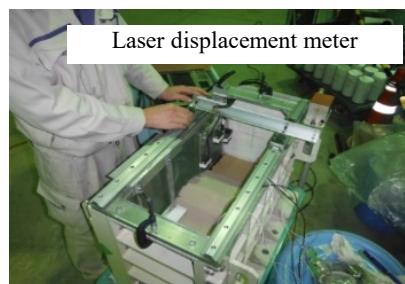


Figure 5.3 Method for measuring shape of each model

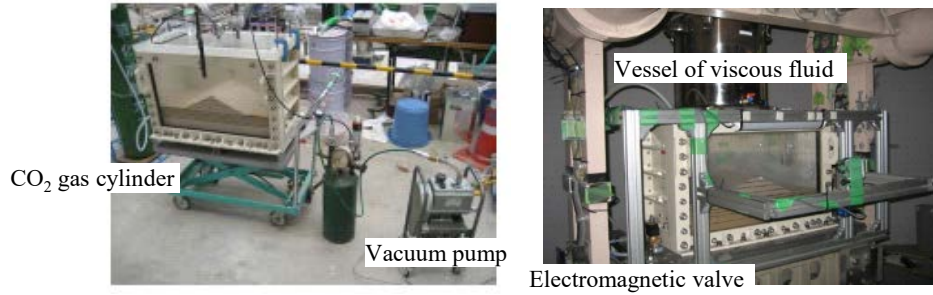


Figure 5.4 Method for pouring pore fluid into liquefiable foundation

Liquefaction experiments are carried out under the centrifugal acceleration of 200 m/s^2 . The duration time and the amplitude of the input wave are chosen to be as long and large, respectively, as possible under the specifications of the shaking device. The input wave is a sine wave, and its amplitude is 3 m/s^2 , its frequency is 3 Hz , and its duration time is 400 s in the prototype scale. Hence, the time converted into the prototype scale is called the “converted time” for a comparison with the real time scale. Figure 5.5 shows a part of the acceleration at the bottom of the soil box. The pore water pressure at each point, shown in Figure 5.1, is measured. Videos of the experiments are recorded by cameras set in front of and on top of the soil box during the experiments. And, the acceleration at the bottom of the soil box is recorded by the accelerometers, presented in Figure 5.1, to confirm that the shaking device can reproduce the input wave properly. The deformed shapes of the models are measured by the laser displacement meter after unloading the centrifugal acceleration (Figure 5.3).

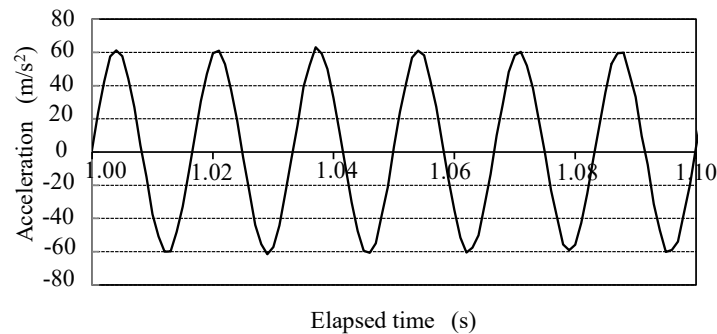


Figure 5.5 Measured acceleration at bottom of soil box

5.1.3 Results and considerations of liquefaction experiments

5.1.3.1 Behavior of pore water pressure

Figure 5.6 shows the time histories of the increase in pore water pressure for the three cases. The elapsed time is expressed as real time and the duration of the shaking is from 0 to 20 s. The legend for each datum in this figure coincides with the pore water pressure transducer shown in Figure 5.1.

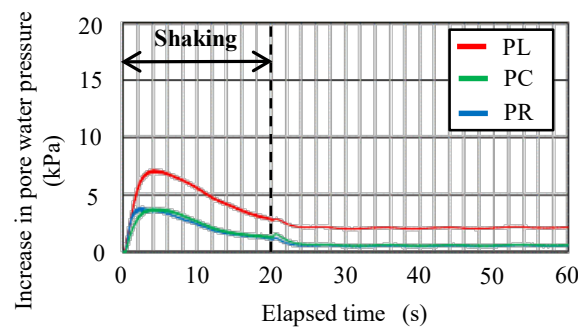
In Case A, which imitates only the liquefiable foundation, the pore water pressure at all points increases immediately at the start of the shaking and decreases after reaching the peak, even during shaking, as shown in Figure 5.6 (1). The behavior of the increase in pore water pressure at PC and that at PR are almost similar, and the maximum value reaches 4 kPa. However, the time required to reach the peak is different. It takes 2.5 s (converted time: 50 s) at PR and 4 s (converted time: 80 s) at PC. The behavior of the increase in pore water pressure at PL shows the same tendency as that at PR and PC. However, the maximum value at PL reaches 7 kPa, which is larger than the others. The time to reach the peak at PL is 4 s (converted time: 80 s) as with PC. Hence, the results vary quantitatively, but they show qualitatively a similar tendency. From these results, it is supposed that the liquefaction phenomena can be reproduced in these experiments.

In Case B, as shown in Figure 5.6 (2), the maximum values for the increase in pore water pressure reach 15 kPa, 16 kPa, and 17 kPa at PL, PC, and PR, respectively. These values show 2.1, 4.0, and 4.3 times compared with those in Case A at PL, PC, and PR, respectively. The behavior of the increase in pore water pressure at PC and that at PR show a similar tendency at first. The pore water pressure increases immediately at the start of the shaking, as all points in Case A, but shows a different tendency after 3 s (converted time: 60 s). The pore water pressure at PC decreases after 3 s (converted time: 60 s) and then increases again after 6 s (converted time: 120 s). It reaches the peak after 11 s (converted time: 220 s) and then decreases. Meanwhile, the pore water pressure at PR does not decrease, but its increase ratio decreases after 3 s (converted time: 60 s). It reaches the peak after 13 s (converted time: 260 s) and then decreases. The pore water pressure at PL continues to increase during shaking, and reaches the peak after 20 s (converted time: 400 s). Thus, the pore water pressure at PL increases gradually until the end of shaking, and its tendency is quite different from that of the others.

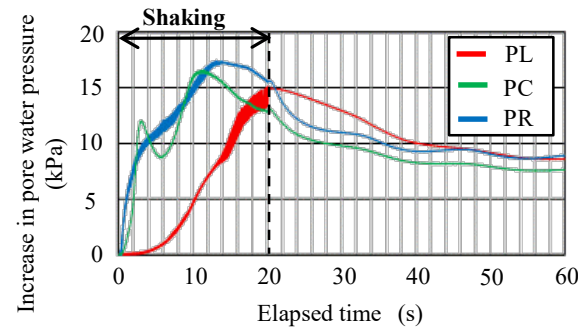
In Case C, as shown in Figure 5.6 (3), the maximum values for the increase in pore water pressure reach 6 kPa at PL, and 7 kPa at PC and PR. These values are almost the same as those in Case A. The pore water pressure at PR increases immediately at the start of shaking and decreases after 3 s (converted time: 60 s). It increases again after 7 s (converted time: 140 s) and reaches the peak after 20 s (converted time: 400 s). While the pore water pressure at PC does not show a rapid increase at the start of the shaking, it continues to increase until 5 s (converted time: 100 s) and then

decreases. It increases again after 12 s (converted time: 240 s) and reaches the peak after 20 s (converted time: 400 s). The pore water pressure at PL continues to increase during shaking and reaches the peak after 20 s (converted time: 400 s). Its behavior is the same as that in Case B.

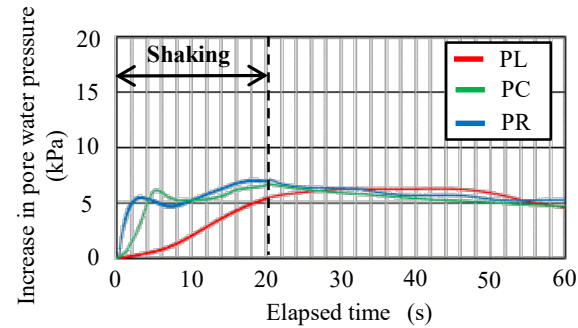
From these results, it is clarified that the dam body affects the behavior of the pore water pressure in the liquefiable foundation. The values of the pore water pressure increase several times when a dam body exists. However, retrofitting at the toe of the downstream slope of the dam body can inhibit the typical increase in pore water pressure. And, the increased value of pore water pressure becomes almost the same as that in the case with only a liquefiable foundation.



(1) Case A



(2) Case B



(3) Case C

Figure 5.6 Time history of increase in pore water pressure for each case

5.1.3.2 Deformed shapes of dam body

Figures 5.7 and 5.8 show the shapes of the dam models before and after shaking in Cases B and C, respectively. In these figures, the shapes of the dam model are shown a) before shaking and (b) after shaking. The dotted line, shown in b) of each figure, expresses the shape of the dam body before shaking.

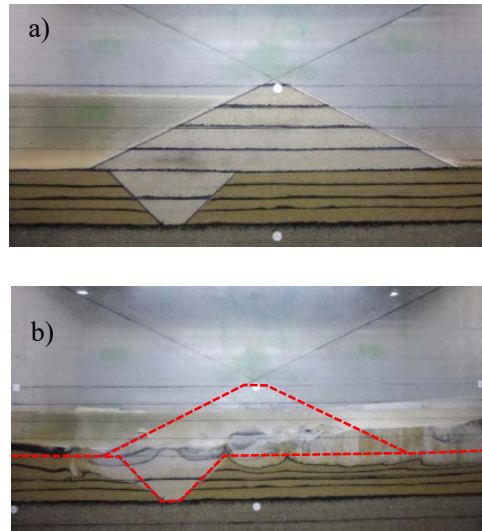


Figure 5.7 Deformed shape of dam body in Case B
a) Before shaking, b) After shaking

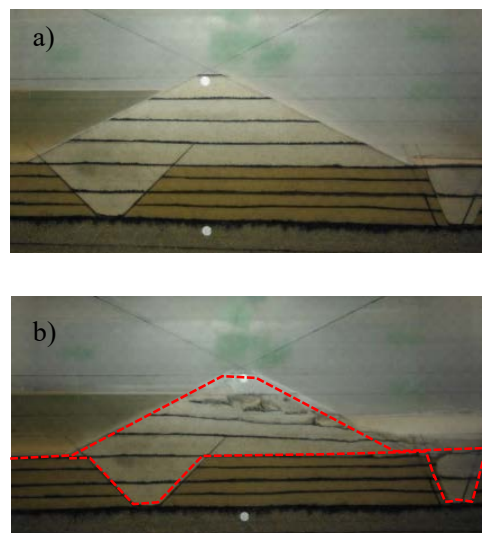


Figure 5.8 Deformed shape of dam body in Case C
a) Before shaking, b) After shaking

From these figures, both models show little deformation under the centrifugal acceleration of 200 m/s^2 , and the water is properly reserved. It takes several hours after pouring the reserved water, but the change in the reserved water level are negligible. From this fact, it is confirmed that the seepage of the reserved water into the dam body and the liquefiable foundation are inhibited.

In Case B, as shown in Figure 5.7 (b), the dam body deforms catastrophically, and the dam body and the foundation become as one. From Figure 5.8 (b), it is recognized that the upper part of the dam body deforms severely, but it can reserve the water until the end of the experiment in Case C. From Figure 5.7 (b), the dam body in Case B settles into the liquefiable foundation and deforms toward the downstream with a large lateral flow. The core trench and the upstream part of the dam body deform toward the upstream with the lateral flow, but the amount of deformation is smaller than its downstream part. It can be recognized from the colored sand layer that the center of the core trench rises at the level of the dam's bottom, but the lower part of the core trench does not deform remarkably. Similarly, the colored sand layer in the liquefiable foundation shows that the upper half of the liquefiable foundation deforms predominantly. From these observations, it is supposed that the remarkable flow of the liquefiable layer arises in its upper part. From Figure 5.8 (b), it is found that the crest and the downstream portion of the dam body in Case C deform severely, but that the large flow toward the downstream is inhibited. The flow is negligible in the core trench and the upstream portion of the dam body. Thus, it is concluded that the deformation of the dam body and the liquefiable foundation, particularly the amount of the flow towards the upstream and the downstream, is remarkably influenced by the retrofitting at the toe of the downstream slope.

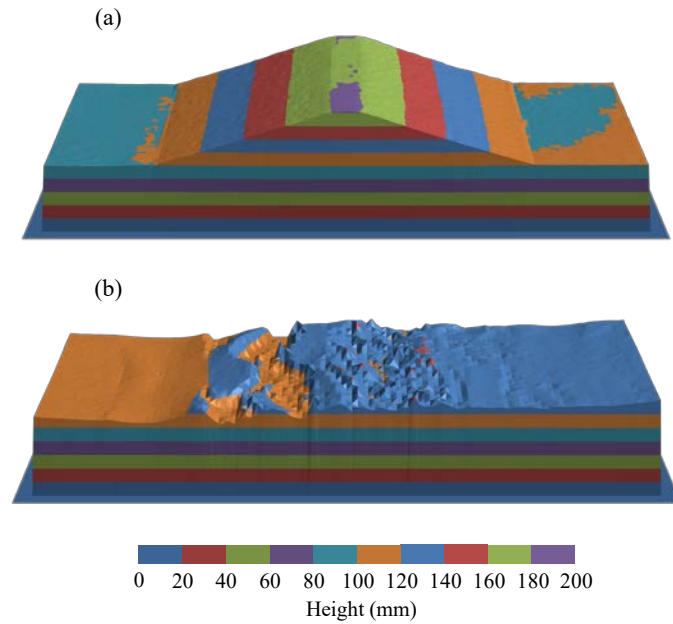


Figure 5.9 Heights of model measured by laser displacement meter in Case B

(a) Before shaking, (b) After shaking

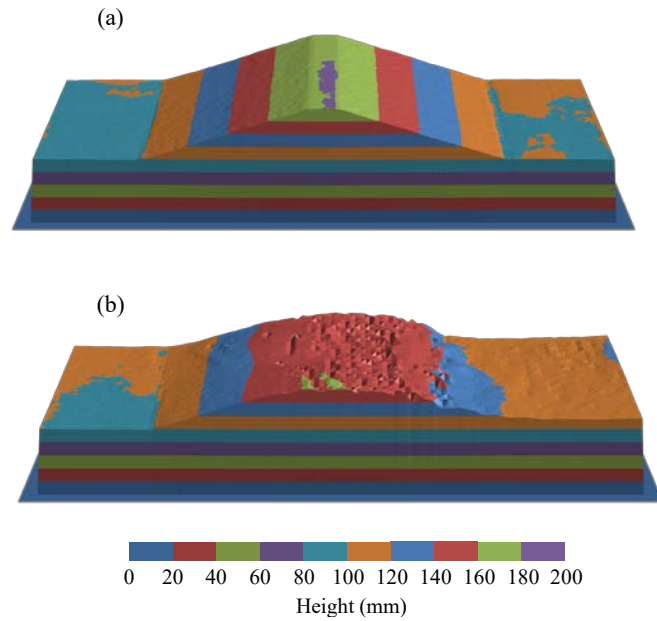


Figure 5.10 Heights of model measured by laser displacement meter in Case C
(a) Before shaking, (b) After shaking

Figures 5.9 and 5.10 show the heights of the models before and after shaking for Cases B and C, respectively. In Model B, as shown in Figure 5.9, the settlement is almost 50 mm at the center of the dam's cross section, and it is 62.5% of the dam's height. The heights on the downstream side of the model increase to over 20 mm, because the materials of the dam body flow into the downstream side and they are deposited on the liquefiable foundation. A large rupture is recognized in the upstream portion of the dam body. The increase in height on the upstream side is smaller than that on the downstream side. In Model C, the settlement is almost 20 mm at the center of the dam's cross section, as shown in Figure 5.10, and it is 25.0% of the dam's height. The heights on the downstream side of the model increase like for Model B, but no ruptures are recognized on the upstream side. The lower part of the upstream slope shows little deformation.

5.1.3.3 Destruction mechanism of dam body

The destruction mechanism of the dam body, induced by the liquefaction of its foundation, is examined by relating the behavior of the pore water pressure in the liquefiable foundation with the deformations of the dam body. It is supposed that the behavior of the liquefiable layer is influenced by the existence of the dam body because of the difference in the initial stress conditions. However, the behavior of the pore water pressure is quite different between Case B and Case C. And, their behavior is more complicated than that in Case A. From these facts, it is supposed that the behavior of the pore water pressure in the liquefiable foundation is closely related not only to the initial stress condition, but also to the deformation behavior of the dam body.

Figures 5.11 and 5.12 show the progress of the destruction of the dam body in Case B, which shows catastrophic deformation because of the lateral flows of the liquefiable foundation and the dam body. Figure 5.11 shows the front view, while Figure 5.12 shows the upper view. From these figures, after 1.0 s (converted time: 20 s), some cracks appear in the crest, but the cross section of the dam body shows little deformation. Then, the downstream portion of the dam body deforms laterally in the downstream direction. After 5.0 s (converted time: 100 s), the shape of the cross section changes remarkably, and many cracks arise on the crest and the downstream slope. Some cracks appear on the upstream slope after 8.0 s (converted time: 160 s) and the reserved water begins to overtop the crest after 9.0 s (converted time: 180 s). One remarkable crack on the upstream is enlarged, and the dam body begins to split in the upstream and downstream directions at the boundary of this crack, after 10.0 s (converted time: 200 s). After 14.0 s (converted time: 280 s), the reserved water perfectly overtops the crest. At the end of the shaking (20.0 s, converted time: 400 s), the dam body catastrophically deforms. Thus, it is concluded that the deformation behavior of the dam body develops progressively, and its tendency changes.

In Case C, the large deformation of the dam body is restricted by the retrofitting at the toe of the downstream slope, and no cracks arise on the upstream slope. The increase in pore water pressure is remarkably lower than that in Case B.

It is revealed that the deformation of the dam body has a remarkable influence on the behavior of the pore water pressure in the liquefiable foundation. The retrofitting, done to restrict the large deformation of the dam body, will prevent the extreme increase in pore water pressure in the liquefiable foundation, and the catastrophic destruction of the dam body can be avoided even though the upper part of the dam body has been severally damaged. In particular, in the case of the inclined core type of zoning, it is supposed that the large deformation in the upstream slope may induce the catastrophic destruction of the dam body. Such deformation should be moderated by restricting the flow of the dam body in the downstream direction.

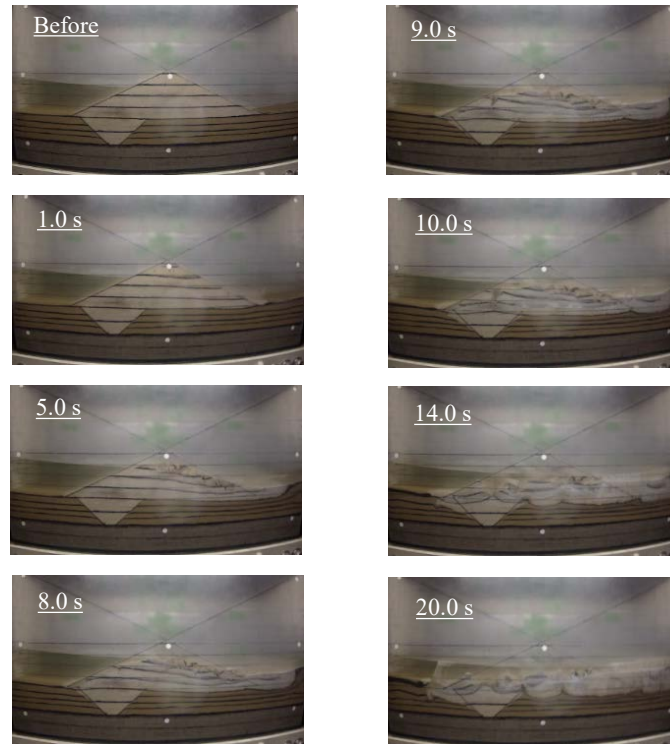


Figure 5.11 Progress of destruction of dam body in Case B (Front view)

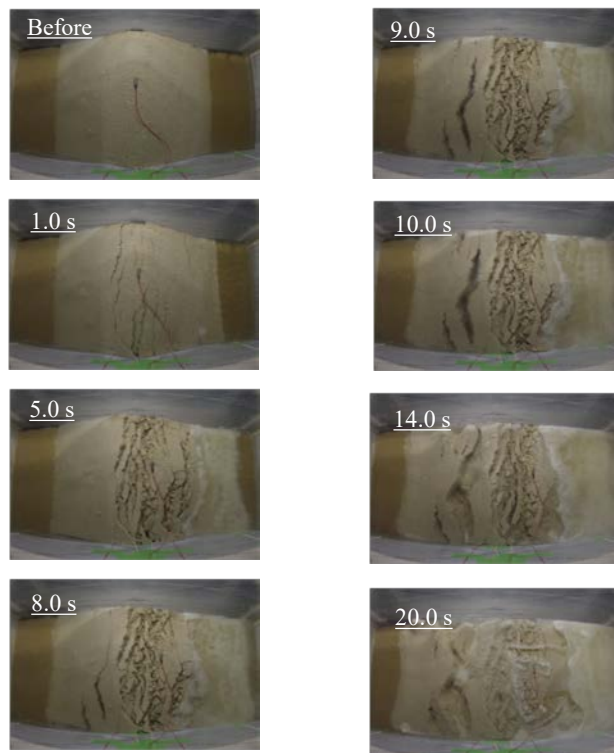


Figure 5.12 Progress of destruction of dam body in Case B (Upper view)

5.2 Numerical experiments

5.2.1 Models and conditions for numerical experiments

There are several zoning types among the fill dams for irrigation. They are roughly divided into three types, namely, the homogeneous type, the inclined core type, and the center core type. The seismic behavior of these zoning types are supposed to be different from each other. Through numerical experiments, the manner in which the deformation behavior of the dam body is influenced by the existence and the position of an impervious zone (core trench) in the liquefiable foundation is examined. Three conditions are considered in the examination, the first is the reserved water condition, the second is the material properties of the liquefiable foundation, and the third is the input waves.

Figures 5.13 to 5.15 present the finite element meshes. Figure 5.13 shows a model of the homogeneous type, Figure 5.14 shows the inclined core type, and Figure 5.15 shows the center core type. Each model has a dam body, with a height of 15 m, a crest width of 4 m, and gradients of the upstream and the downstream slopes of 1:2. The dam bodies are constructed on a liquefiable foundation, but the bodies are made of non-liquefiable materials. The modeled areas of the foundation have a depth that is three times the dam height and a width that is five times the bottom width. The liquefiable foundation is horizontally distributed with a thickness of 8 m on the basement.

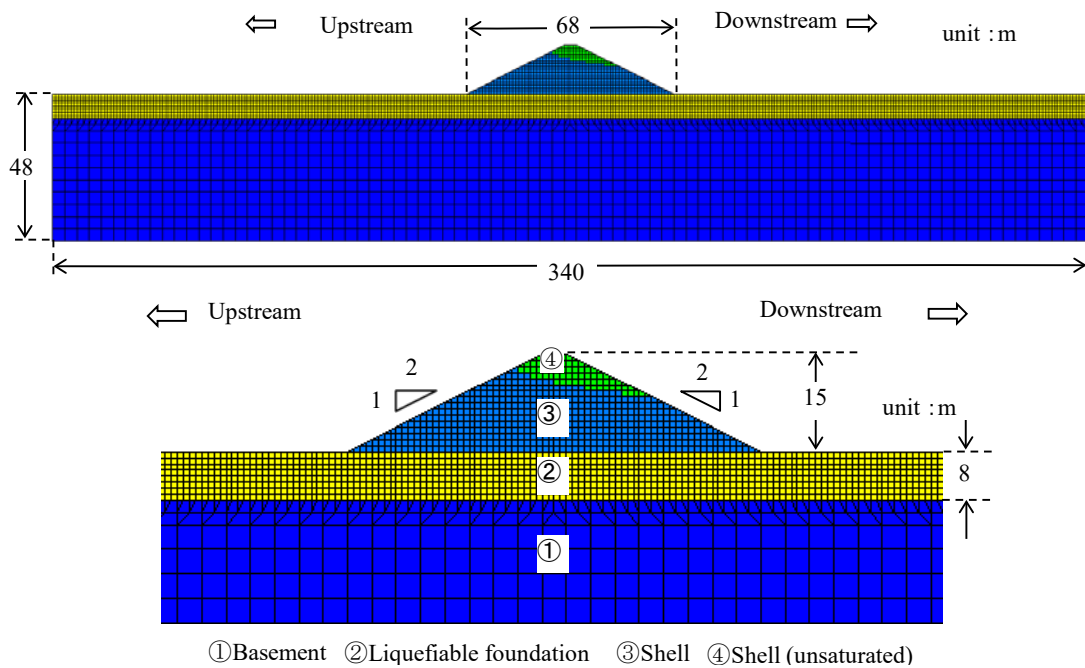
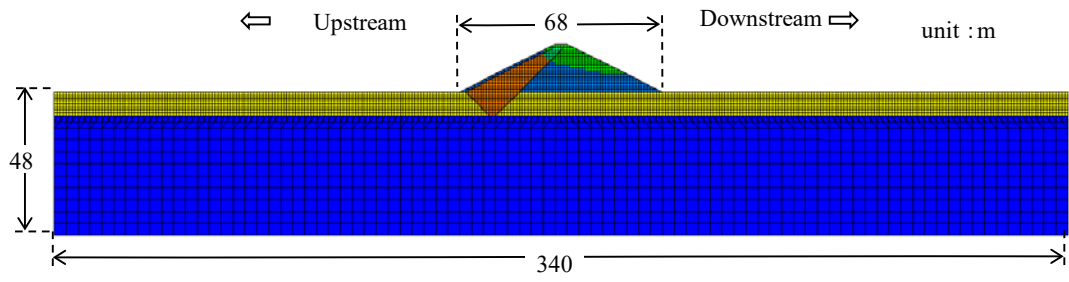
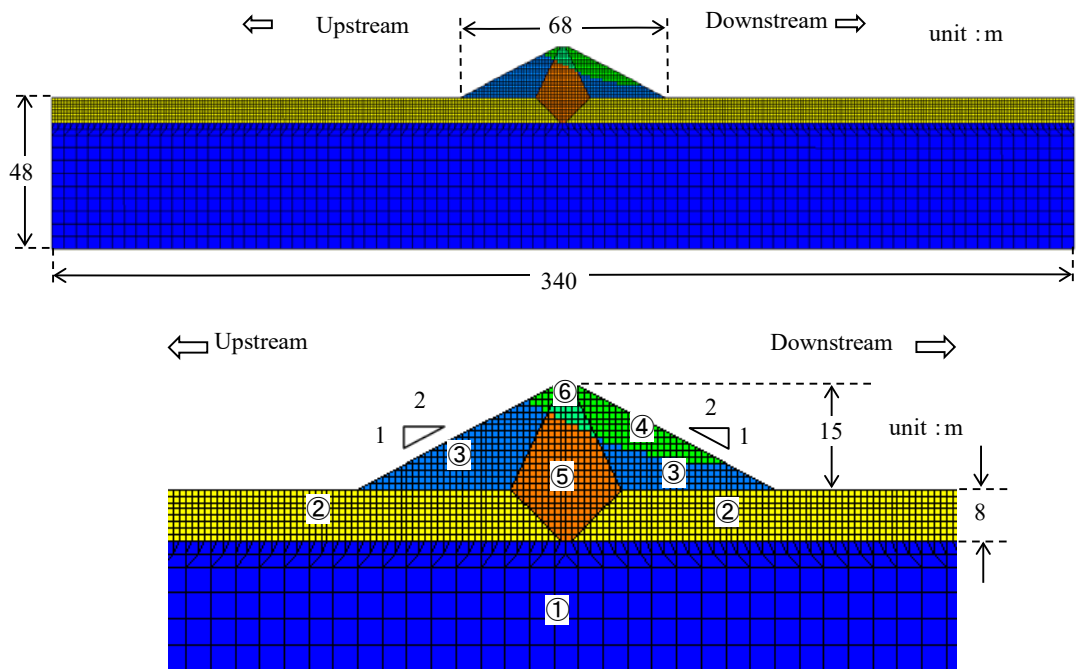


Figure 5.13 Finite element mesh of homogeneous type of zoning



①Basement ②Liquefiable foundation ③Shell ④Shell (unsaturated) ⑤Core ⑥Core (unsaturated)

Figure 5.14 Finite element mesh of inclined core type of zoning



①Basement ②Liquefiable foundation ③Shell ④Shell (unsaturated) ⑤Core ⑥Core (unsaturated)

Figure 5.15 Finite element mesh of center core type of zoning

The material properties of the dam bodies are assumed to be the same, except for the permeability coefficient and the unit weights. Table 5.1 shows the material properties of the basement, the liquefiable foundation, and the dam body. Two materials are selected for the liquefiable foundation. One material, whose N value is 5, shows a rapid increase in axial strain after the stress condition reaches the phase change line; it is referred to as Mat A. The other material, whose N value is 15, shows cyclic mobility behavior during cyclic loading; it is referred to as Mat B.

Two conditions are considered for the reserved water. One is a reserved water depth of 13m, and the other is no reserved water because of the rapid decrease in water level, which is considered to be a dangerous situation in conventional slope stability analyses. The former is called the “reserved condition” and the latter is called the “rapid decrease condition”. In the reserved condition, the wet unit weight is used in the unsaturated zone, and submerged unit weights are used in the saturated zone of the dam body, the basement, and the liquefiable foundation. In the rapid decrease condition, submerged unit weights are applied to the basement, the liquefiable foundation, and the core trench, supposing a free water level located on the surface of the liquefiable foundation. Here, the saturated unit weight is used in the saturated zone of the dam body, except for the core trench, and the wet unit weight is used in the unsaturated zone.

Table 5.1 Material properties

	γ_t (kN/m ³)	γ_{sat} (kN/m ³)	γ_{sub} (kN/m ³)	c (kPa)	ϕ	k (m/s)
Basement	-	27.05	17.27	-	-	4.0×10^{-8}
Liquefiable foundation (Mat A)	-	18.13	8.33	0	25	5.0×10^{-5}
Liquefiable foundation (Mat B)	-	19.21	9.41	0	32	5.0×10^{-5}
Dam body (Shell)	16.82	17.36	7.56	30	25	3.5×10^{-6}
Dam body (Core)	16.82	17.36	7.56	30	25	2.0×10^{-6}

The FEM-FDM coupled liquefaction analysis method “LIQCA” (Oka et al., 1994; The LIQCA Research and Development Group, 2005) is used in the present study. A constitutive model for sand with a generalized flow (Oka et al., 1999) is utilized to estimate the deformation of the liquefiable foundation and the dam body induced by liquefaction. Table 5.2 shows the material parameters for the constitutive model. These material parameters are estimated from the N value and the material properties (shown in Table 5.1) of each material. Here, the dilatancy coefficient D^*_0 of the dam body is zero, assuming that the dam body is made of non-liquefiable materials. It is assumed that the material parameters of the dam body are the same under saturated and unsaturated conditions, except

for the permeability coefficient. The undrained boundary condition is set at the boundary between the saturated and the unsaturated materials.

Table 5.2 Material parameters of constitutive model

	Liquefiable foundation (Mat A)	Liquefiable foundation (Mat B)	Dam body
e_0	1.00	0.77	1.20
λ	0.03	0.02	0.30
κ	0.010	0.002	0.030
OCR*	1.0	1.4	1.0
G_0/σ'_m	390.00	770.00	921.42
M_f^*	1.015	1.100	1.764
M_m^*	0.944	0.900	0.944
B_0^*	2000	8000	3000
B_1^*	10	150	50
C_f	0	0	3000
γ_r^{P*}	0.050	0.005	-
γ_r^{E*}	0.2	0.1	-
D_0^*	3.0	2.5	0.0
n	1.0	3.0	0.0
C_d	2000	2000	2000

Figure 5.16 shows the cyclic strength curves simulated by the parameter sets for Mat A and Mat B, shown in Table 5.2, and the liquefaction strength estimated by the equation proposed in the specifications for highway bridges (Japan Road Association, 2012). From this figure, the simulated cyclic strength curves coincide with the estimated liquefaction strength. Figure 5.17 shows the stress paths and the stress-strain relations simulated by these parameter sets for $\sigma'_{m0} = 100$ kPa and $\eta = 0.20$. Here, σ'_{m0} is the mean effective stress and η is the shear stress ratio. η is defined as τ/σ'_{m0} and τ is the amplitude of cyclic shear stress. Mat A shows a rapid increase in axial strain after the occurrence of liquefaction, while Mat B shows cyclic mobility behavior.

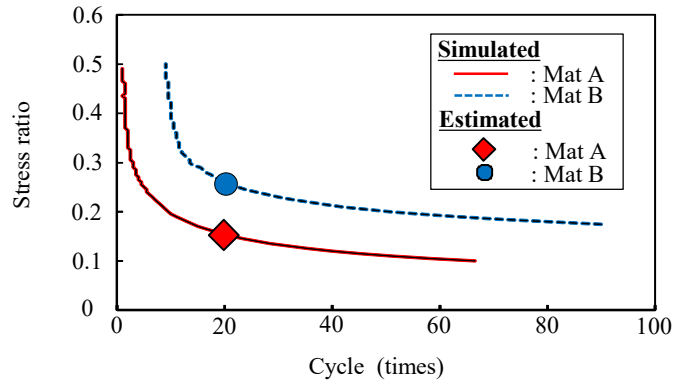


Figure 5.16 Cyclic strength curves of liquefiable foundations

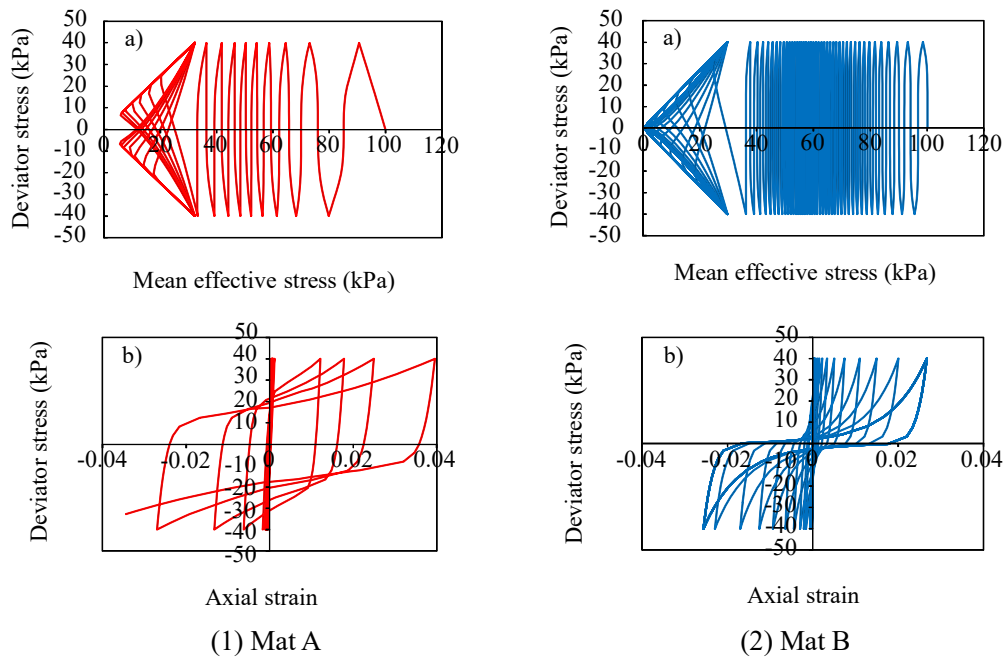


Figure 5.17 Simulation results for liquefiable foundations

a) Stress path, b) Stress-strain relation

The undrained boundary condition is set on the outer edge of the model. The horizontal displacement is fixed and the vertical displacement is free on both sides of the model. Its bottom boundary is set to be the viscous boundary.

Two seismic waves are selected as the input waves for the numerical experiments. Figures 5.18 and 5.19 show the acceleration time history and the Fourier spectrum of each wave. WAVE 1 is the N-S component of the seismic record measured at a site of JMA (Japan Meteorological Agency) in the 1995 Hyogoken-Nanbu Earthquake (JMA). Its duration time is 20 s, the maximum acceleration

is 8.18 m/s^2 , and the predominant frequency is 1.5 Hz . WAVE 2 is the N-S component of the seismic record measured at a dam site in the 2004 Niigataken-Chuetsu Earthquake (Masukawa et al., 2014). Its duration time is 41 s , the maximum acceleration is 4.36 m/s^2 , and the predominant frequency is 2.5 Hz .

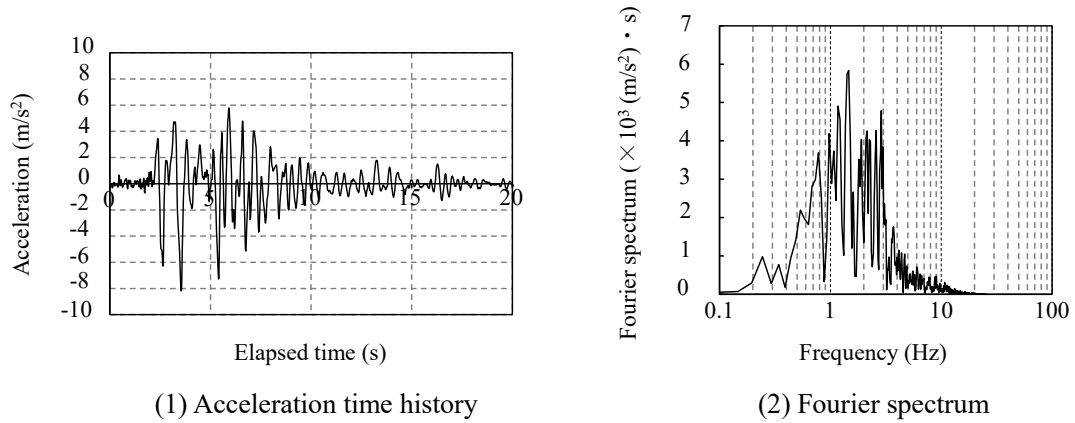


Figure 5.18 Input wave (WAVE 1)

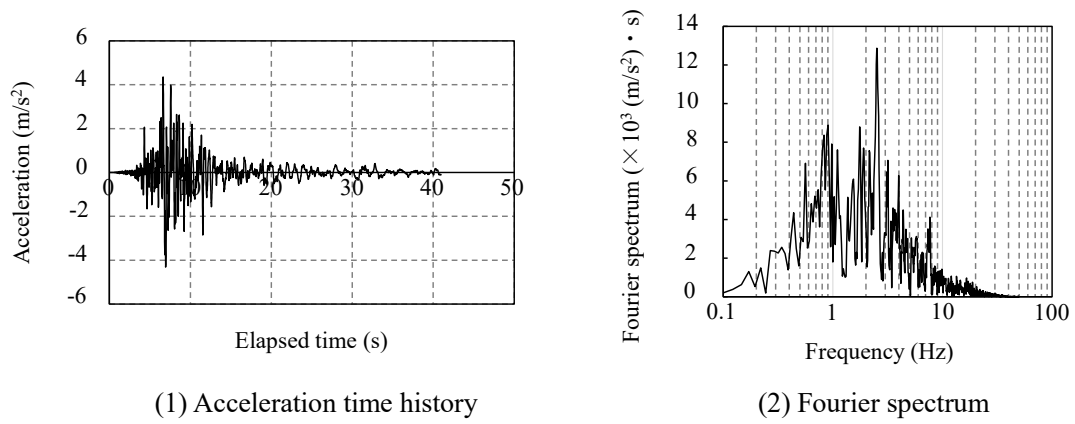


Figure 5.19 Input wave (WAVE 2)

5.2.2 Numerical results and considerations

Figures 5.20 and 5.21 show the deformation and the decreasing ratio of the effective stress ($\Delta\sigma'_m/\sigma'_{m0}$) after shaking for WAVE 1. The results for the liquefiable foundation with Mat A are given in Figure 5.20, while those with Mat B are given in Figure 5.21. The two reserved water conditions are compared. Similarly, Figures 5.22 and 5.23 show the results with Mat A and Mat B for WAVE 2, respectively.

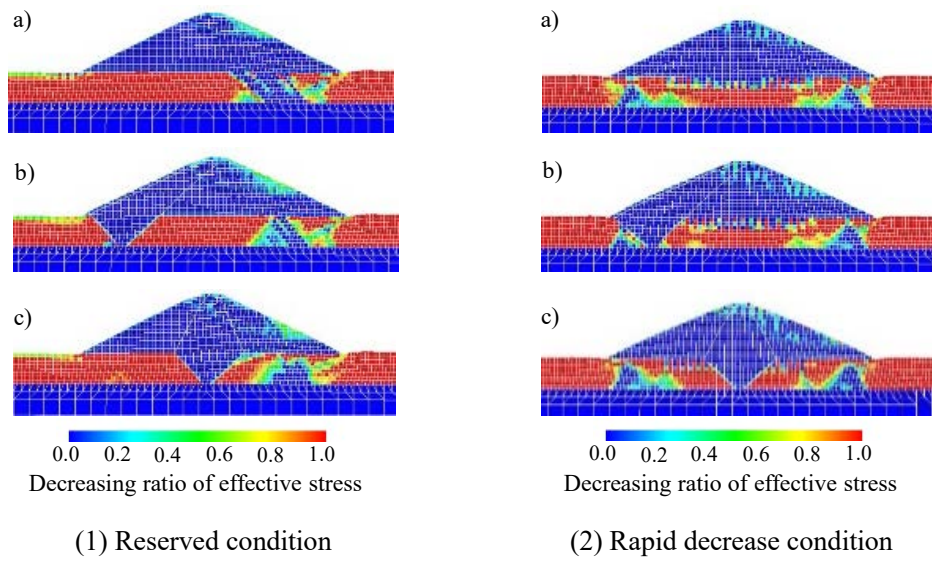


Figure 5.20 Results of simulations when input wave is WAVE 1 (Mat A)

a) Homogeneous type, b) Inclined core type, c) Center core type

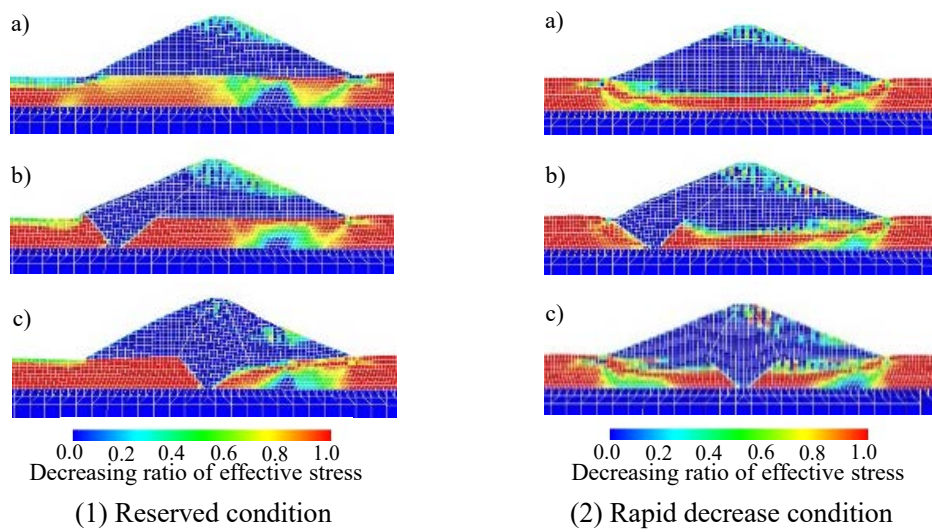


Figure 5.21 Results of simulations when input wave is WAVE 1 (Mat B)

a) Homogeneous type, b) Inclined core type, c) Center core type

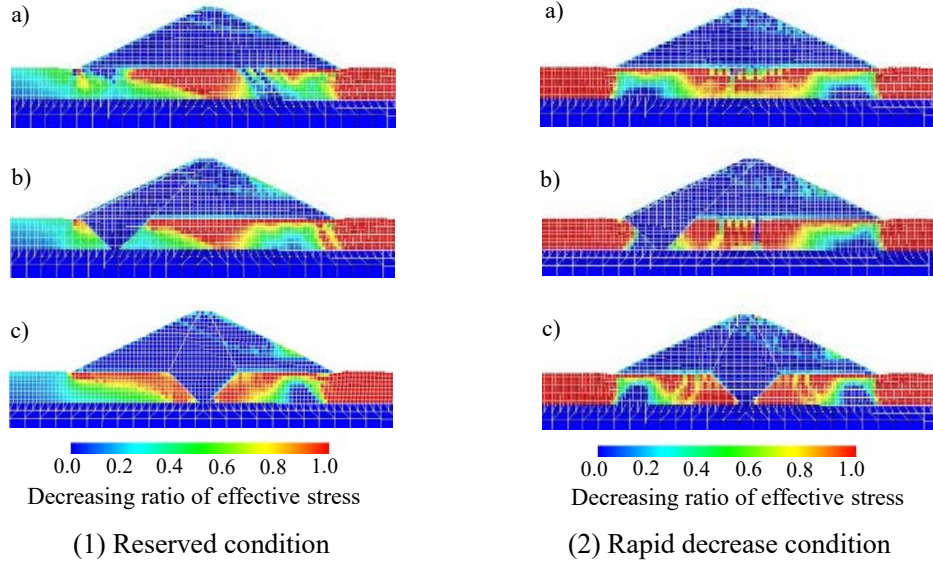


Figure 5.22 Results of simulations when input wave is WAVE 2 (Mat A)

a) Homogeneous type, b) Inclined core type, c) Center core type

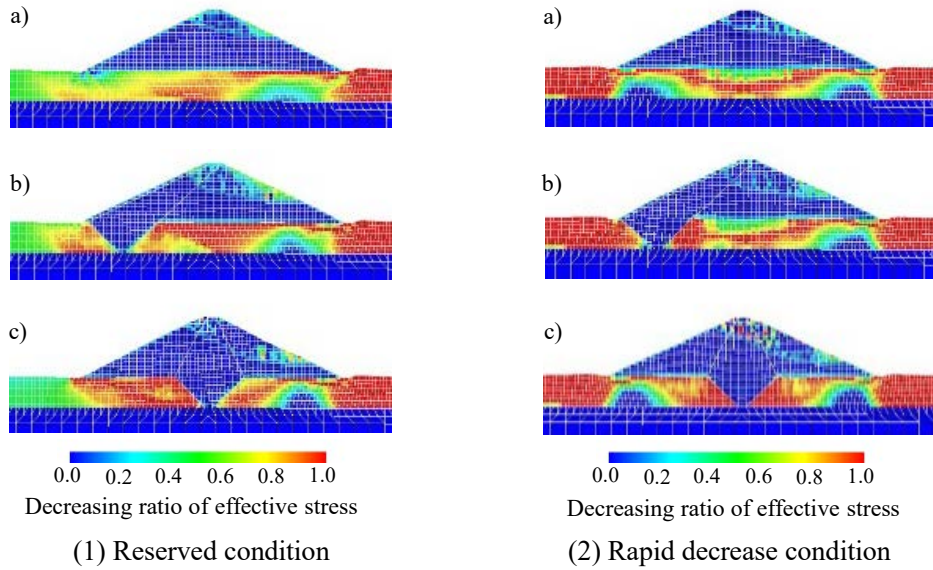


Figure 5.23 Results of simulations when input wave is WAVE 2 (Mat B)

a) Homogeneous type, b) Inclined core type, c) Center core type

From the results, it is found that the deformation mode of each dam type shows the same tendency in spite of the differences in the material properties of the liquefiable foundations and the characteristics of the input waves. On the other hand, the deformation modes change remarkably due to the difference in the zoning type and the reserved water conditions.

Figures 5.24 and 5.25 show a comparison of the displacement loci between the two reserved water conditions in WAVE 1 and WAVE 2, respectively. Tables 5.3 and 5.4 show the final displacements at the center of the crest under the reserved and the rapid decrease conditions, respectively. It is revealed that all zoning types tend to deform in the downstream direction under the reserved condition and settle vertically under the rapid decrease condition. The maximum vertical displacement appears in the inclined core type under the reserved condition despite the difference in the liquefiable foundations and the characteristics of the input waves. The center core type tends to settle more than the others under the rapid decrease condition.

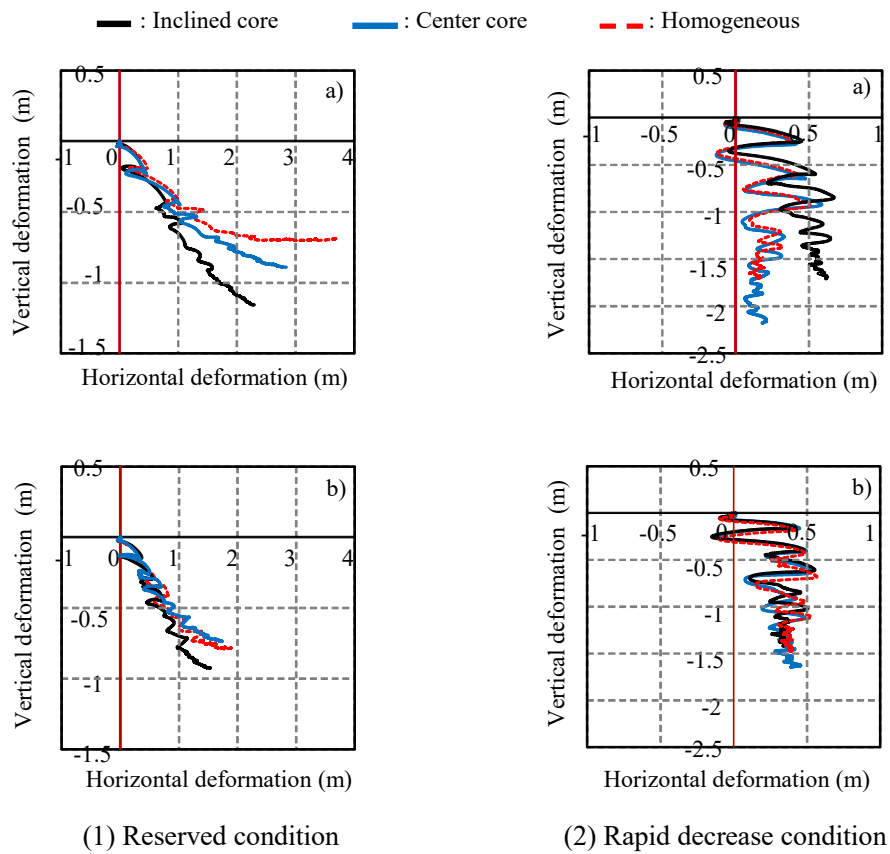
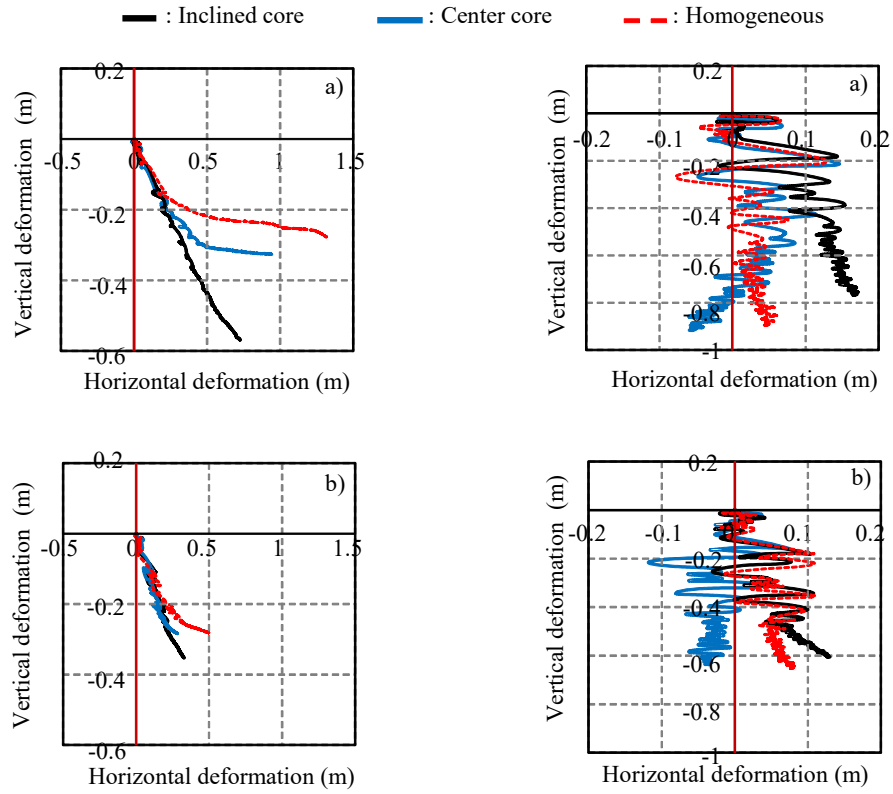


Figure 5.24 Displacement loci at center of crest (WAVE 1)

a) Mat A, b) Mat B



(1) Reserved condition

(2) Rapid decrease condition

Figure 5.25 Displacement loci at center of crest (WAVE 2)

a) Mat A, b) Mat B

Table 5.3 Final displacement at center of crest (Reserved condition)

		WAVE 1		WAVE 2	
		Horizontal	Vertical	Horizontal	Vertical
Mat A $N = 5$	Homogeneous	3.70	-0.69	1.32	-0.28
	Inclined core	2.30	-1.16	0.73	-0.57
	Center core	2.84	-0.89	0.94	-0.33
Mat B $N = 15$	Homogeneous	1.91	-0.78	0.50	-0.28
	Inclined core	1.53	-0.93	0.33	-0.35
	Center core	1.74	-0.74	0.28	-0.28

Table 5.4 Final displacement at center of crest (Rapid decrease condition)

		WAVE 1		WAVE 2	
		Horizontal	Vertical	Horizontal	Vertical
Mat A $N = 5$	Homogeneous	0.15	-1.72	0.04	-0.90
	Inclined core	0.61	-1.71	0.17	-0.77
	Center core	0.18	-2.18	-0.05	-0.92
Mat B $N = 15$	Homogeneous	0.69	-1.48	0.08	-0.65
	Inclined core	0.34	-1.42	0.13	-0.61
	Center core	0.39	-1.65	-0.04	-0.64

The deformation characteristics of each zoning type are summarized as follows. The homogeneous type does not show remarkable settlement compared to the other zoning types in spite of the absence of the core trench. The dam body tends to move in the downstream direction without changing its original shape under the reserved condition. Thus, localized deformation and typical settlement are avoided. Under the rapid decrease condition, the dam body tends to sink into the liquefied foundation, but its settlement is not typically larger than the others. These phenomena cause the attenuation of the seismic wave in the liquefiable foundation during an earthquake. If liquefaction arises, seismic waves will not propagate into the dam body and the decrease in stiffness of the dam body will be reduced. The inclined core type of zoning tends to deform in the downstream direction in spite of the difference in the reserved water conditions, because the core trench is located at the upstream side of the liquefiable foundation. The core trench locally affects the upstream slope, and the deformation mode in the upstream slope is different from the others. The settlement at the center of the crest is smaller than the others under the rapid decrease condition. However, this zoning type shows typically large settlement under the reserved condition. The settlement of the center core type of zoning is almost the same as or larger than that of the homogenous type under the reserved condition, and becomes the largest under the rapid decrease condition. It is supposed that these phenomena are induced by the core trench, because the stiffness of the dam body decreases due to the propagation of seismic waves through the core trench, and then the dam body with low stiffness sinks into the liquefied foundation.

It is revealed that the core trench may restrain the deformation of the dam body induced by the lateral flow of the liquefied foundation, but may affect its deformation mode. The core trench remarkably affects the settlement of the dam body and its localized deformations, because the seismic wave propagates through the core trench and the stiffness of the dam body decreases.

5.3 Numerical verification of damage to small-scale dams induced by liquefaction during past massive earthquakes

5.3.1 Damaged dams and conditions for numerical analysis

Figures 5.26 to 5.28 show the finite element meshes for three dams damaged by past earthquakes. The mesh represents the damaged cross section of each dam. The width of the model is about five times the embankment base, and its depth is about three times the height of the embankment.

Figure 5.26 shows Dam A with a height of 4.0 m and a base width of 36.1 m. It was damaged by the 1983 Nihonkai-Chubu Earthquake, in which the crest settled a maximum of 1.8 m. Figure 5.27 shows Dam B with a height of 6.0 m and a base width of 27.0 m. It was damaged by the 1995 Hyogoken-Nanbu Earthquake, in which the embankment collapsed severely. The settlement of the crest was presumed to be more than 1.25 m. Figure 5.28 shows Dam C with a height of 13.0 m and a base width of 94.2 m. It was damaged by the 1993 Hokkaido Nansei-Oki Earthquake with wide cracks and a maximum height gap of 1 m. In these models, the liquefiable layer in the foundation is set as the liquefaction zone.

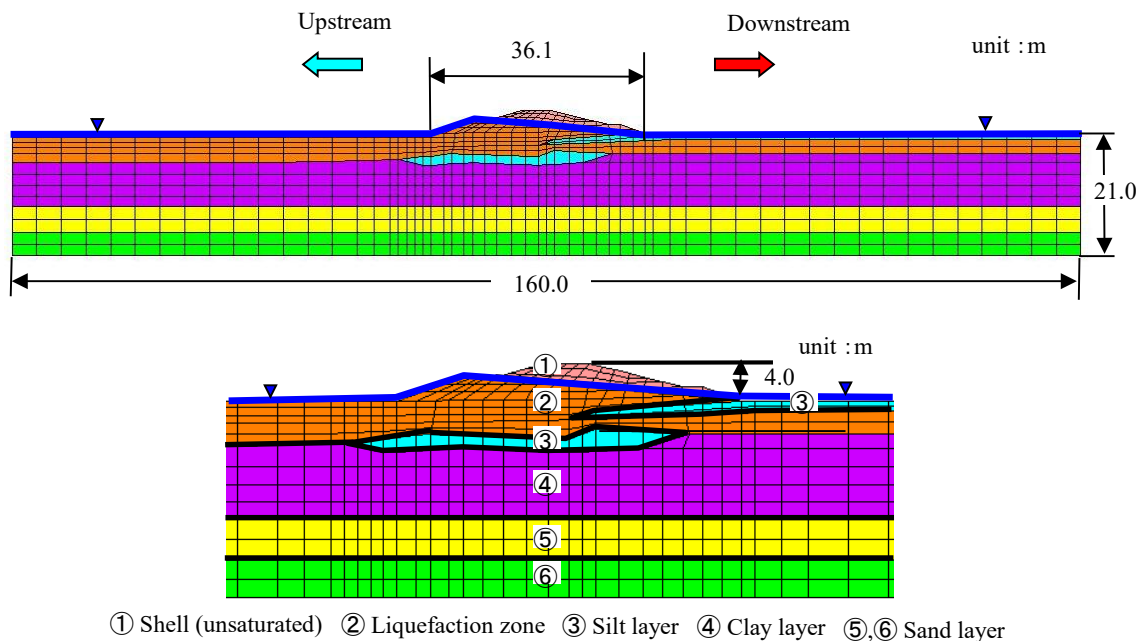


Figure 5.26 Finite element mesh of Dam A

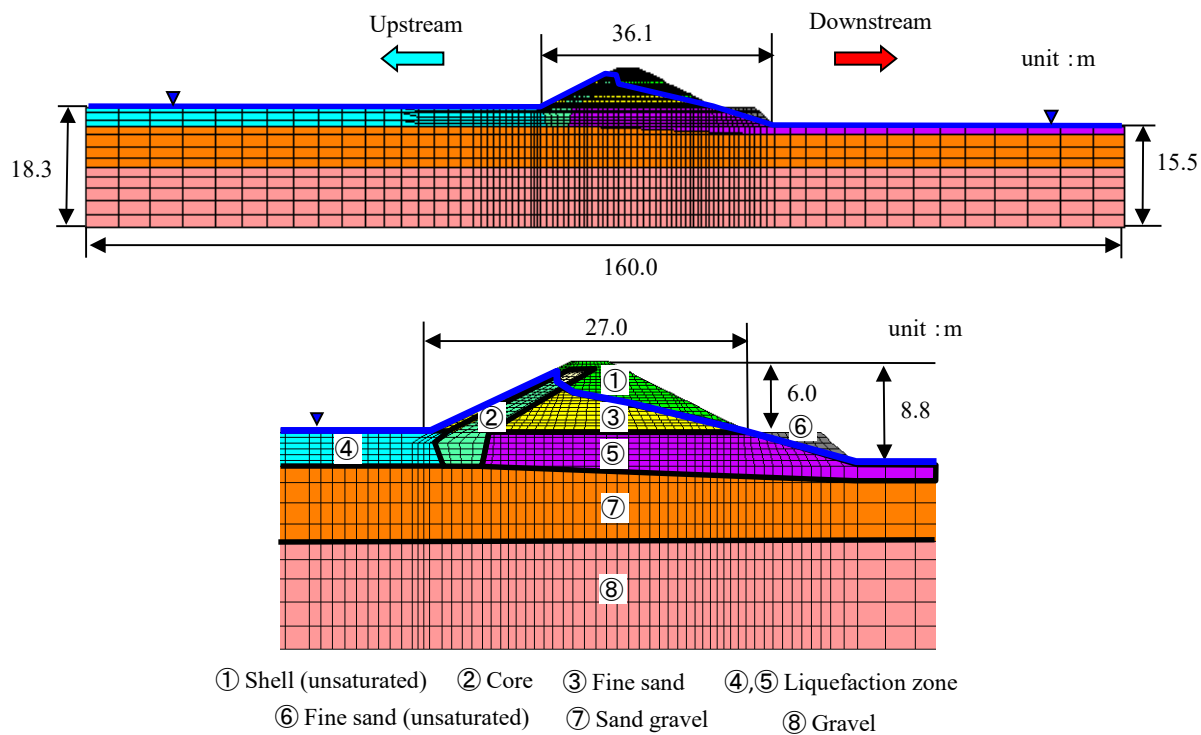


Figure 5.27 Finite element mesh of Dam B

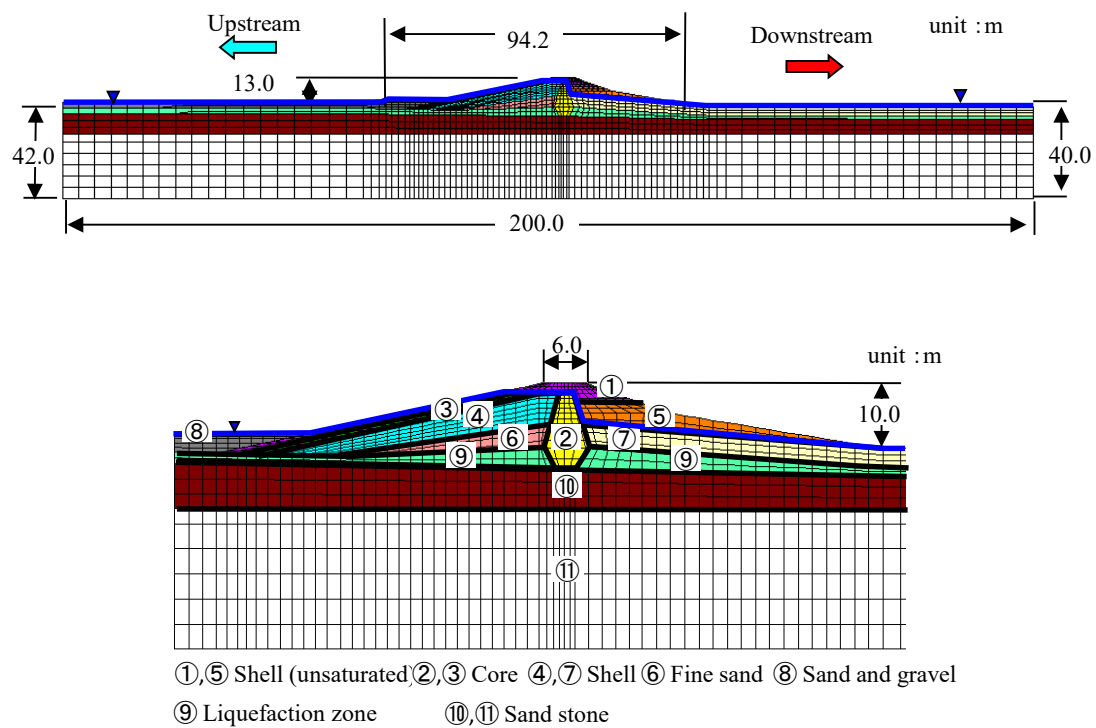


Figure 5.28 Finite element mesh of Dam C

The material parameters for the constitutive model are determined based on the experimental data. For the liquefaction zone, data on the cyclic strength curves were available, but no data were available from the triaxial tests including the stress paths and the stress-strain relations. The parameters are determined to reproduce the cyclic strength curves. Figure 5.29 shows the cyclic strength curves obtained from the experiments and the simulations for the three dams. Figure 5.30 shows the stress paths and the stress-strain relations simulated by the parameters set at $\sigma'_{m0} = 100$ kPa and $\eta = 0.15$. Here, σ'_{m0} is the initial mean effective stress and η is the shear stress ratio. η is defined as τ/σ'_{m0} , and τ is the amplitude of cyclic shear stress.

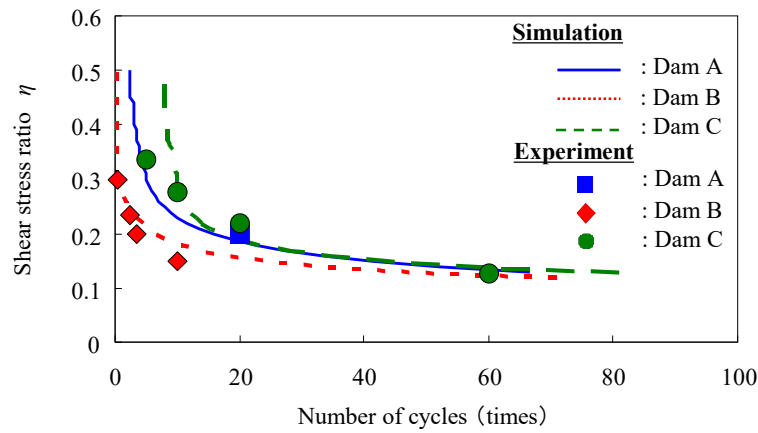


Figure 5.29 Cyclic strength curves obtained from experiments and simulations

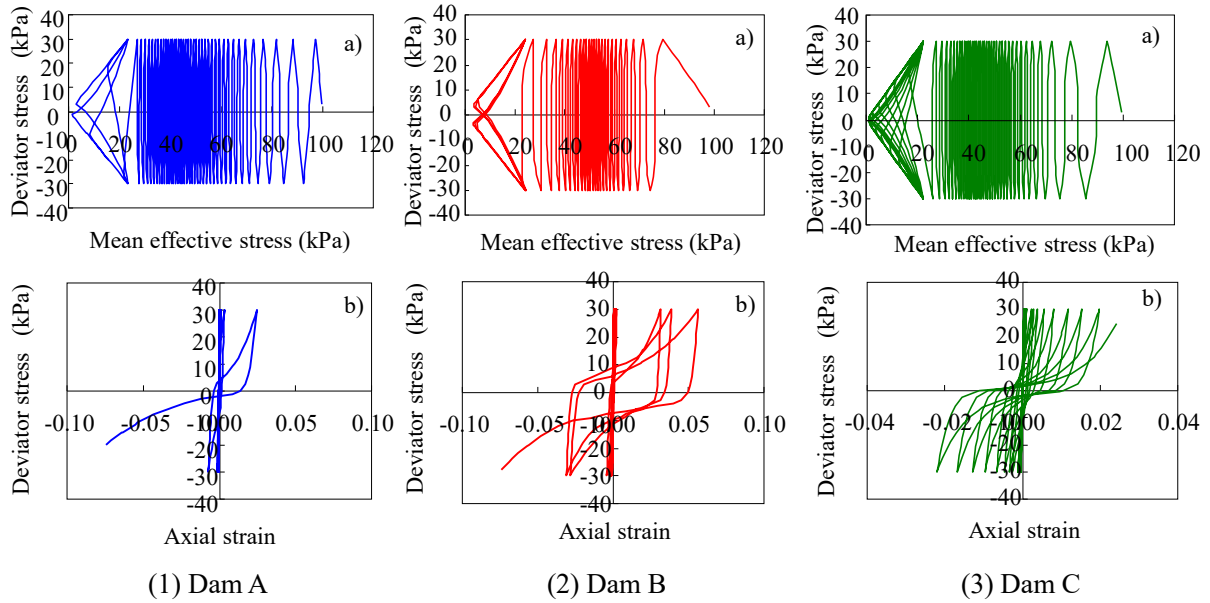


Figure 5.30 Simulation results for liquefaction zone of each dam

a) Stress path, b) Stress-strain relation

From Figure 5.29, it is clear that the simulated cyclic strength curves for Dam A and Dam C are almost the same for 20 cycles. However, the shapes of the stress paths and the stress-strain relationships are different. Figure 5.30 (3) indicates that the stress paths for Dam C show typical cyclic mobility. On the other hand, the stress paths for Dam A in Figure 5.30 (1), show a sudden large deformation.

In these models, it is assumed that the horizontal displacement is fixed, the vertical displacement is free at the side boundary, and the base boundary is set to be the viscous boundary. The surfaces of the saturated material are set to be permeable boundaries. It is assumed that the boundaries between the unsaturated material and the saturated material are impermeable. The reserved water is not considered here.

There is no record of the seismic waves near these dams. Hence, the data monitored by the seismic recorder placed about 30 to 50 km away from these dams are used as the input waves directly. Therefore, the conditions of the geological structure are different between the dam sites and the recorded ones, and the input waves do not necessarily represent the real waves at the dam sites. Figures 5.31 to 5.33 show the input wave for each dam.

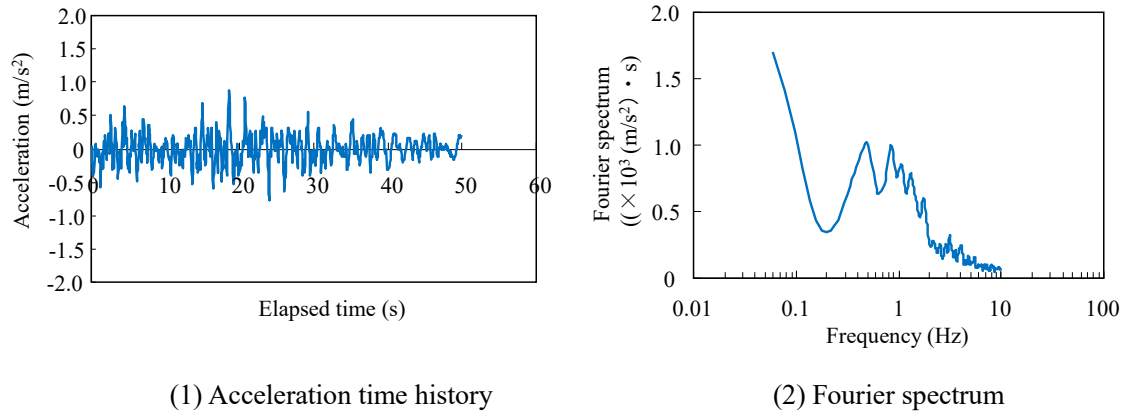


Figure 5.31 Input wave for Dam A

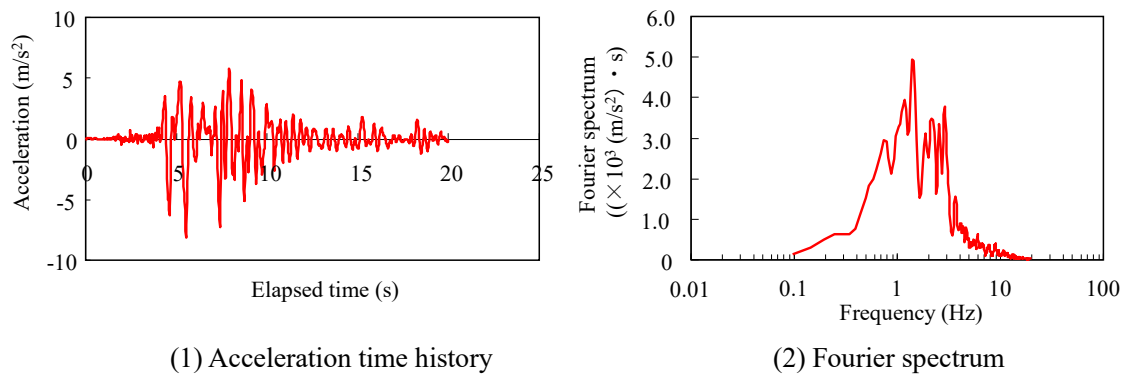
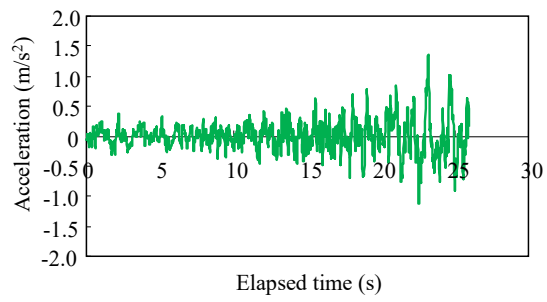
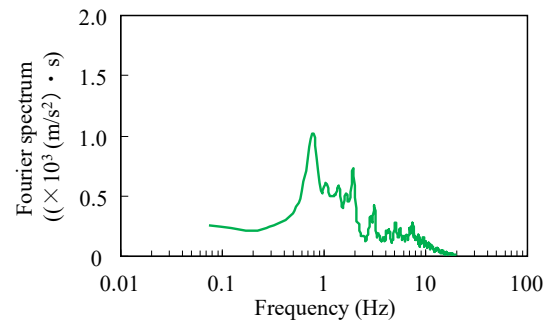


Figure 5.32 Input wave for Dam B



(1) Acceleration time history



(2) Fourier spectrum

Figure 5.33 Input wave for Dam C

5.3.2 Numerical results

Figure 5.34 shows the deformed mesh and the decreasing ratio of the effective stress, while Figure 5.35 shows the deformation loci at the three specific nodes of the crest in Dam A. Similarly, Figures 5.36 and 5.37 show the results for Dam B, and Figures 5.38 and 5.39 show the results for Dam C. The decreasing ratio of the effective stress means $\Delta\sigma'_m/\sigma'_{m0}$; when liquefied perfectly, this value becomes 1.0. These figures show the conditions at the end of the earthquake.

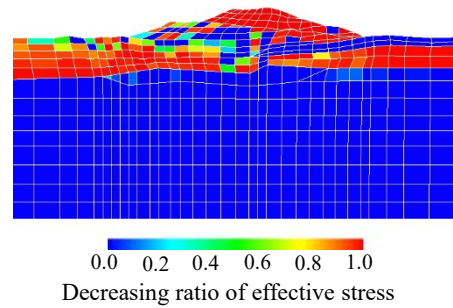


Figure 5.34 Deformed mesh and decreasing ratio of effective stress for Dam A

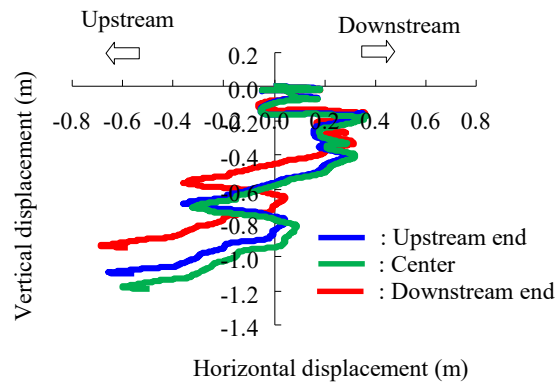


Figure 5.35 Displacement loci at three nodes of crest in Dam A

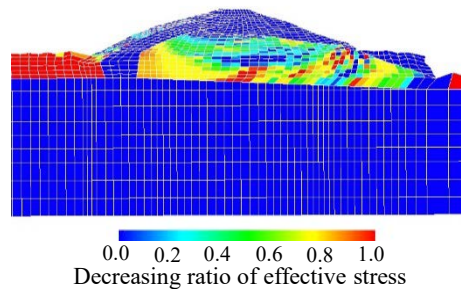


Figure 5.36 Deformed mesh and decreasing ratio of effective stress for Dam B

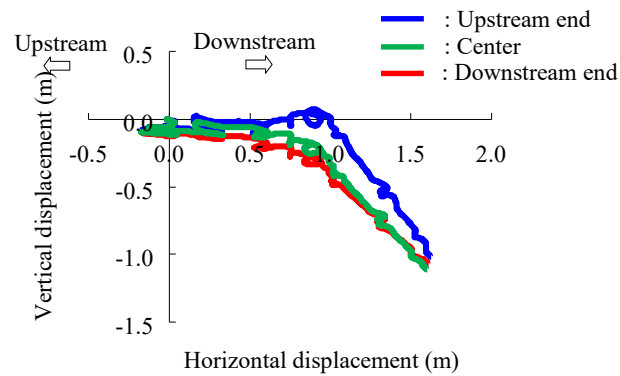


Figure 5.37 Displacement loci at three nodes of crest in Dam B

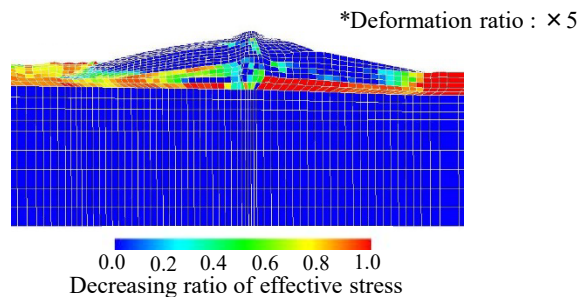


Figure 5.38 Deformed mesh and decreasing ratio of effective stress for Dam C

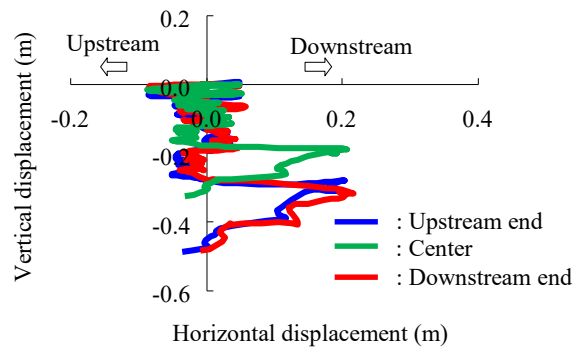


Figure 5.39 Displacement loci at three nodes of crest in Dam C

Figures 5.34, 5.36, and 5.38 show the liquefaction zones for the three dams, where the decreasing ratio of the effective stress is approximately 1.0. From these figures, the deformation modes for the three types of dams are different. Figure 5.34, for Dam A, it is seen that the upstream slope and the crest of the embankment moved to the upstream side, while the lower part of the downstream slope moved to the downstream side. In Figures 5.36 and 5.38 for Dam B and Dam C, respectively, the final deformation modes of the embankments are different on the upstream side and the downstream side, because the embankment is divided into two parts by the core zone. In Figure 5.36, the value of the decreasing ratio of the effective stress increased in almost all parts of the embankment with the liquefaction of the embankment and the foundation. The entire embankment moved toward the downstream side, and the core zone leaned in the downstream direction, because most of the embankment is on the downstream side in the inclined core type. In Figure 5.38, for Dam C, the upstream portion of the embankment moved to the upstream side, while the downstream portion moved to the downstream side. Due to the fine sand layer on the upstream side of the embankment and the asymmetry of the shape of the embankment, the deformation did not occur in a symmetrical mode and the core bent slightly. The crest settled almost vertically, and the upstream and downstream slopes moved to the upstream and downstream sides, respectively. Discontinuous deformation occurred at the upper part of both slopes. Such behavior may cause vertical gaps on the slopes. These results represent the actual situation.

In Figure 5.35, for Dam A, the crest of the embankment settled and gradually moved to the downstream side, and then suddenly moved to the upstream side. Finally, the center of the crest settled about 1.2 m, and then moved about 0.5 m to the upstream side. In Figure 5.37, for Dam B, the embankment did not settle, but moved to the downstream side at first. Then, it was horizontally displaced approximately 1.0 m to the downstream side, where crest settlement began to occur. Finally, the center of the crest settled about 1.1 m and moved about 1.6 m to the downstream side. From Figure 5.39, for Dam C, the crest of the embankment settled almost vertically, and then suddenly moved to the downstream side. Finally, the center of the crest settled almost vertically approximately 0.3 m, and both ends of the crest settled almost vertically approximately 0.5 m. These results show that the final deformation modes of the embankments are different due to the existence and the position of the cores.

The analyzed maximum values of the settlement of the crest are compared with the measured ones. In Dam A, the simulated settlement is 1.2 m and the measured one is 1.8 m. In Dam B, the simulated settlement is 1.1 m and the estimated one is 1.25 m. In Dam C, the simulated gap is 0.2 m and the measured one is 1 m. The simulated results are almost consistent with the measured ones.

Chapter 6 Influence of dam shape and input wave direction on three-dimensional seismic response behavior of fill dams

6.1 Shaking table tests

6.1.1 Purpose

Since the 1995 Hyogoken-Nanbu Earthquake, other huge earthquakes have occurred frequently in Japan, for example, the 2004 Niigataken-Chuetsu Earthquake, the 2008 Iwate-Miyagi Nairiku Earthquake, the 2011 off the Pacific coast of Tohoku Earthquake and so on.

No fill dams designed by the pseudo static (seismic coefficient) method have been utterly destroyed by the motion of such huge earthquakes. However, damage, such as settlement and longitudinal cracks at the crest, have been observed (Kohgo, 2006; Masukawa et al., 2012; Masukawa et al., 2014). It is supposed that these incidents of damage were caused by the complicated three-dimensional seismic behavior of the dam body in a higher seismic mode and that such behavior will be an important factor for managing the safety of existing dams.

Studies on the changes in the shaking mode and the predominant periods of a dam body have been enumerated and expressed as the three-dimensional behavior of fill dams according to the shape of the valley. In these studies (Aoyama, 1979; Makdisi et al., 1982; Mejia and Seed, 1983; Dakoulas and Gazetas, 1986; Gazetas and Dakoulas, 1992; Watanabe and Kawakami, 1995), the results of numerical analyses under two- and three-dimensional conditions were compared by considering the valley shapes. It was concluded that the behavior of a dam body is remarkably influenced by the abutment when the shape of the valley is steep and the crest/height ratio of the dam body is small. Therefore, it is recommended that the three-dimensional condition be considered when the shape of the valley is steep and the crest/height ratio is small. In addition, physical modeling experiments were performed to examine the influence of the abutment on the seismic behavior of the dam body by varying the shape and the crest/height ratio of the dam models made of silicon rubber. These results (Yasunaka, 1986; Watanabe and Kawakami, 1995; Masukawa et al., 2000) reported the same conclusions as the numerical studies.

The above-mentioned studies were mainly focused on the seismic response in the stream direction, because the maximum acceleration response of fill dams rises in the stream direction. The pseudo static method is used for the design of the cross section of fill dams. However, few studies have focused on the seismic response in the dam-axis direction. It is important to consider the behavior of a dam body on the seismic mode, which shows the maximum seismic response, when the stability of the dam body is verified in the design phase.

A higher seismic mode, showing the complicated seismic response, is essential for verifying the

complicated seismic behavior of a dam body in a real earthquake. This is important the same as the seismic mode which shows the maximum seismic response for the safety management of existing dams.

In this chapter, the characteristics of the seismic behavior of fill-type dams are examined by shaking table tests, in which three dam models with different shapes are shaken separately in the stream and dam-axis directions, and then the effects of the dam shape and the direction of the input wave on the seismic response behavior are verified.

6.1.2 Properties of dam models

Three types of dam models, with different shapes, are tested to evaluate the effect of the dam shape on the seismic behavior. The models consist of a dam body made of silicon rubber and an abutment made of a metal frame and a wooden board. The metal frame and the shaking table are rigidly connected by bolts.

Silicon rubber, with a low elastic modulus, is selected as the material for the dam body because of the ease of observing the changes in seismic mode according to the shape of the dam model and the shaking direction. The silicon rubber used in this study is a two-component type of rubber which hardens at ordinary temperatures after mixing the main material and the stiffening agent. The density of the silicon rubber after hardening is 0.98 g/cm^3 , the elastic modulus is 39.2 kN/m^2 , and Poisson's ratio is 0.5.

The overview of a constructed model is shown in Figure 6.1. A mold is used in the construction of the dam body to divide the slopes into three layers. The procedure for the construction of the dam body is given as follows: 1) Set the mold on the abutment frame; 2) Pour the silicon rubber, in a liquid state, into the mold; 3) Leave the silicon rubber at rest until it hardens perfectly. This cycle of three steps is repeated three times for the 1st to 3rd layers. The silicon rubber permeates into the surface of the wooden abutment and hardens. The dam body and the abutment are integrated to avoid the separation of these two parts.

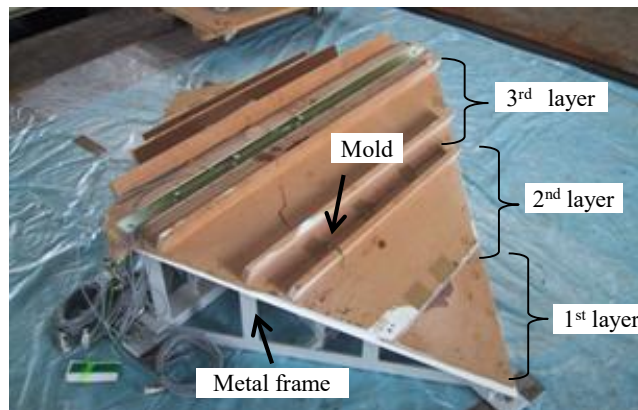


Figure 6.1 Overview of dam model under construction (Model C)

The dam models are schematically shown in Figure 6.2. Model A has a symmetric shape with respect to both the maximum cross section and the dam-axis. The symmetric axis of Model B is the dam-axis. Model C has an asymmetric shape. All three models have the same height, length of crest, and slope gradient. Their height is 300 mm, their length of crest is 1,200 mm, their crest length/height ratio is 4, and their slope gradient is 1:2.5. Accelerometers are located at the appointed sites, as shown in Figure 6.2, to record the acceleration during the tests. In Models A and C, the accelerometers are set on the points which divide the length between the center of the crest (A1 and C1) and the abutment into three. In Model B, the accelerometers are arranged at even intervals on the crest and divide the crest length into four sections (B1: right above the deepest valley and B2: center of the crest). The arrows in this figure indicate the positive acceleration directions.

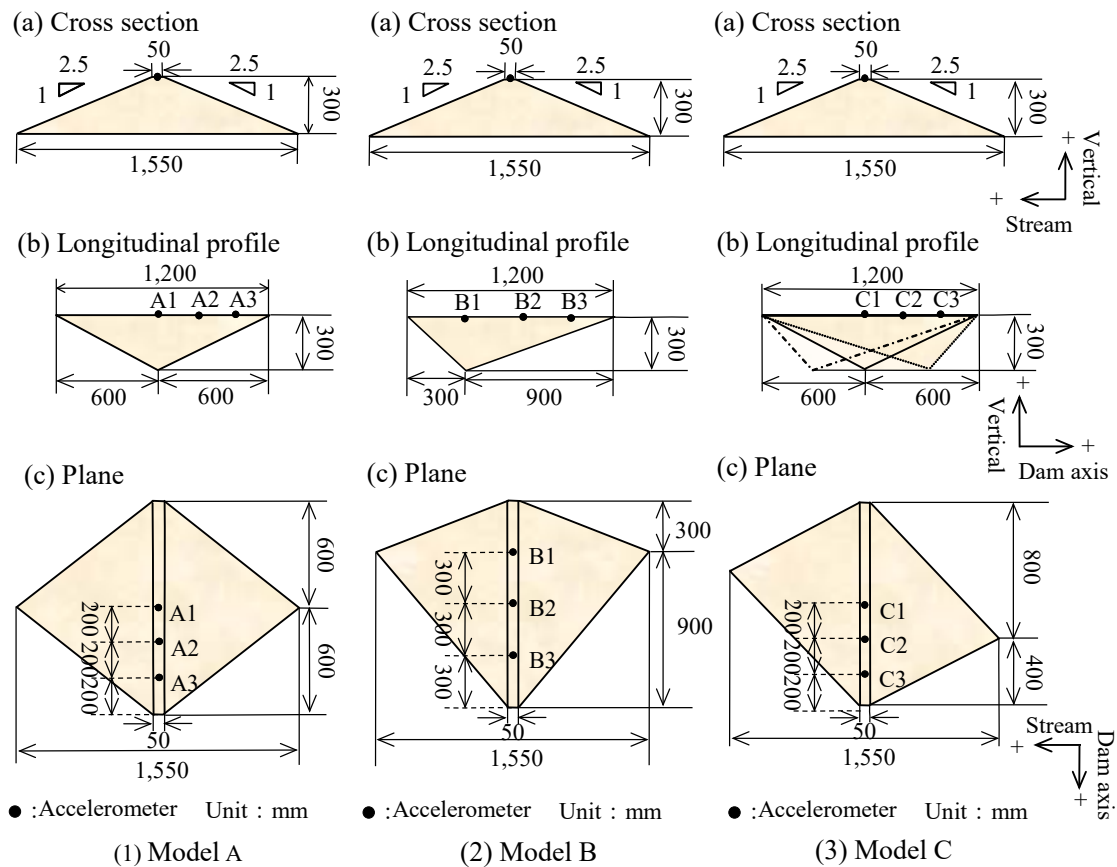


Figure 6.2 Schematics of models

6.1.3 Test conditions

Three accelerometers (strain-gage type) are set on the shaking table and each of them measures a component of the three-dimensional accelerations (stream, dam-axis, and vertical directions). Piezoelectric-type accelerometers are set at the prescribed sites on the crest of the dam models; they measure the acceleration responses of the dam bodies during the experiments. Piezoelectric-type accelerometers are small (11 mm × 11 mm × 11 mm), but the three components of the accelerations can be measured with just this one device. Before setting the accelerometers on the crest of the dam bodies, they are set on the shaking table and calibrated by a comparison with strain gage-type accelerometers under shaking. Figure 6.3 shows an overview of the shaking table tests. The dam models are set on the shaking table, and the stream and dam-axis directions of these models are seen to coincide. Therefore, the seismic responses of the three models can be measured simultaneously by one shaking procedure. From the trial shaking, it is found that the accelerometer at the top of the abutment shows almost the same accelerations as those on the shaking table. Then, it is considered that the abutment behaves rigidly in the same way as the shaking table.

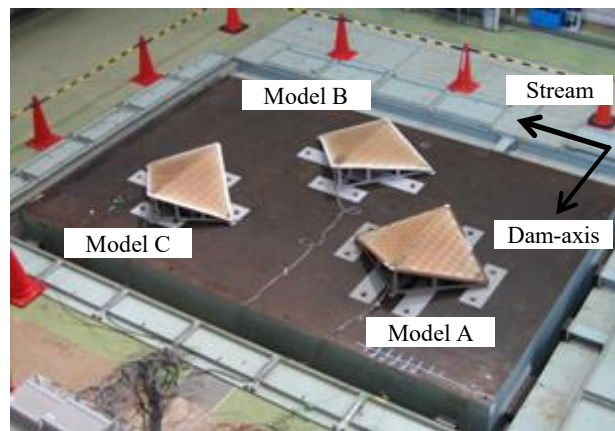


Figure 6.3 Overview of shaking table tests

Table 6.1 shows the specifications of the experiment cases. At first, the experiments are carried out under the frequency sweep condition in order to examine the effects of the shaking direction and the frequency of the input wave on the seismic behavior of the dam body. The designed wave is a sine wave, with amplitude of 1.0 m/s^2 , whose frequency is swept from 0.6 to 14.0 Hz. However, the shaking table device cannot sweep the frequency from 0.6 to 14.0 Hz thoroughly because of the limited amount of data. Thus, it is divided into two waves, Wave 1 and Wave 2. The frequency of Wave 1 is swept from 0.6 to 9.3 Hz, while that of Wave 2 is swept from 4.7 to 14.0 Hz. The experiment cases using these waves are Case 1 and Case 2. The shaking directions of Cases 1 and 2 are the stream and the dam-axis, respectively.

It is shown from the frequency sweep tests that the dam bodies tend to show the typical responses in the direction orthogonal to the shaking direction at some frequencies when the shaking direction is the dam-axis. In addition, therefore, supplementary experiments are conducted under the constant mono frequency condition; these cases are Cases 3 and 4. In these two cases, the frequencies of the input sine waves are selected to be 6.5 Hz and 7.6 Hz, respectively. Then, the shaking direction is the dam-axis in Cases 3 and 4, and the amplitude of the sine wave is 1.0 m/s^2 , the same as for Cases 1 and 2.

Table 6.1 Specifications of experiment cases

	Experiment case	Frequency (Hz)	Amplitude (m/s^2)	Shaking direction
Frequency sweep	Case 1	0.6 ~ 9.3 Hz (Wave 1)	1.0	Stream
		4.7 ~ 14.0 Hz (Wave 2)		
	Case 2	0.6 ~ 9.3 Hz (Wave 1)	1.0	Dam-axis
		4.7 ~ 14.0 Hz (Wave 2)		
Mono frequency	Case 3	6.5 Hz	1.0	Dam-axis
	Case 4	7.6 Hz	1.0	Dam-axis

Figure 6.4 shows the results of the accelerations measured on the shaking table in the frequency sweep tests, as an example. It is obvious that the amplitudes of the input waves are almost 1.0m/s^2 and the shaking table device represents the prescribed amplitude value. Figure 6.5 shows the relations between the frequencies of the waves measured on the shaking table and the elapsed time. Here, the frequencies are calculated by the zero-crossing method. The frequencies of both waves increase lineally among the prescribed frequency ranges for Wave 1 at a rate of 0.33 Hz/s and for Wave 2 at a rate of 0.36 Hz/s . It is confirmed that the shaking table device can reproduce the required amplitude and frequency of the input waves.

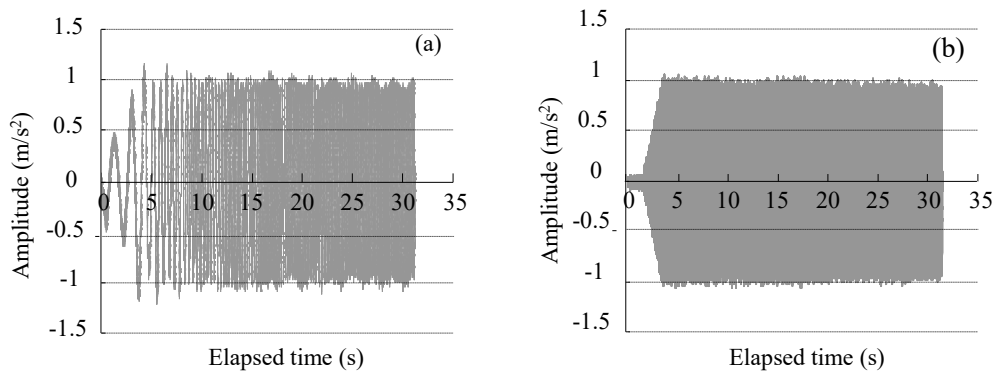


Figure 6.4 Measured input waves
(a) Wave 1, (b) Wave 2

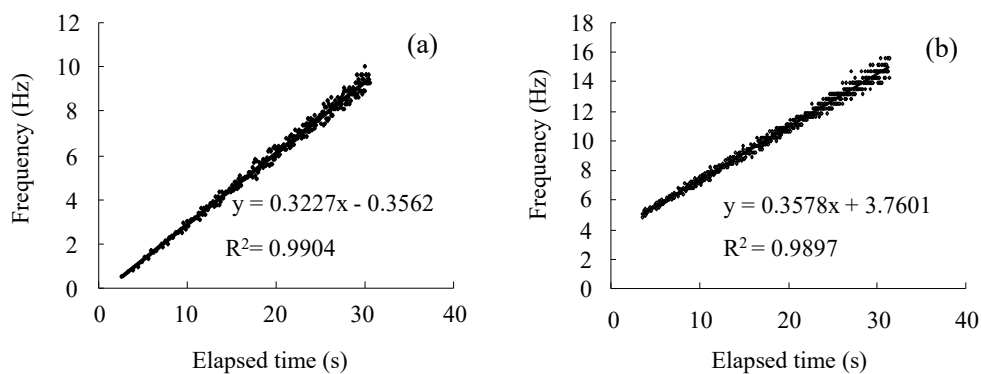


Figure 6.5 Frequencies of measured input waves changing with elapsed time
(a) Wave 1, (b) Wave 2

6.2 Results and consideration of shaking tests under sweep frequency condition

6.2.1 Acceleration response

6.2.1.1 Time histories

Table 6.2 shows the maximum values of accelerations (absolute values) measured at each site of the models. In Case 1, the maximum response of Model A is 11.8 m/s^2 at A1 and its direction is stream. The maximum response of Model B is 8.6 m/s^2 at B1, the maximum response of Model C is 10.3 m/s^2 , and their directions are stream. In Case 2, the maximum accelerations of Models A, B, and C are 9.7 m/s^2 at A1, 6.9 m/s^2 at B1, and 9.3 m/s^2 at C1, respectively, and their directions are all dam-axis. In both cases, the direction of the maximum response coincides with that of the input wave. The maximum response arises on the right above the deepest valley (A1, B1, and C1) in all models.

Table 6.2 Measured maximum acceleration values (absolute values)

	Case 1 (Stream direction)			Case 2 (Dam-axis direction)		
	Stream	Dam-axis	Vertical	Stream	Dam-axis	Vertical
	(m/s^2)	(m/s^2)	(m/s^2)	(m/s^2)	(m/s^2)	(m/s^2)
A1	11.8	0.7	1.1	0.6	9.7	2.4
A2	5.5	1.4	0.6	0.5	5.3	3.9
A3	3.1	0.3	0.3	0.5	2.9	1.4
B1	8.6	0.7	0.6	3.0	6.9	4.4
B2	5.7	1.0	0.7	3.7	6.5	4.1
B3	4.2	0.7	0.3	0.6	2.3	1.2
C1	10.3	2.0	0.9	2.1	9.3	0.8
C2	4.3	1.3	0.9	2.4	5.8	3.8
C3	2.4	0.5	0.4	0.7	2.5	0.9

The time series data measured in Cases 1 and 2 are shown in Figures 6.6 to 6.8 and Figures 6.9 to 6.11, respectively. Figures 6.6 and 6.9 are the results of Model A, Figures 6.7 and 6.10 are those of Model B, and Figures 6.8 and 6.11 are those of Model C. The amplitudes of the accelerations change remarkably according to the changes in frequency with time. The maximum acceleration response value and the shape of the acceleration wave at each site of the models are quite different. In Case 1 (shaking to the stream direction), the response of all the models is mainly to the stream. In Case 2 (shaking to the dam-axis direction), however, a remarkable vertical response arises in all the models, and Models B and C show three-dimensional response behavior. Thus, the response behavior of the dam body changes remarkably according to the direction of the input wave.

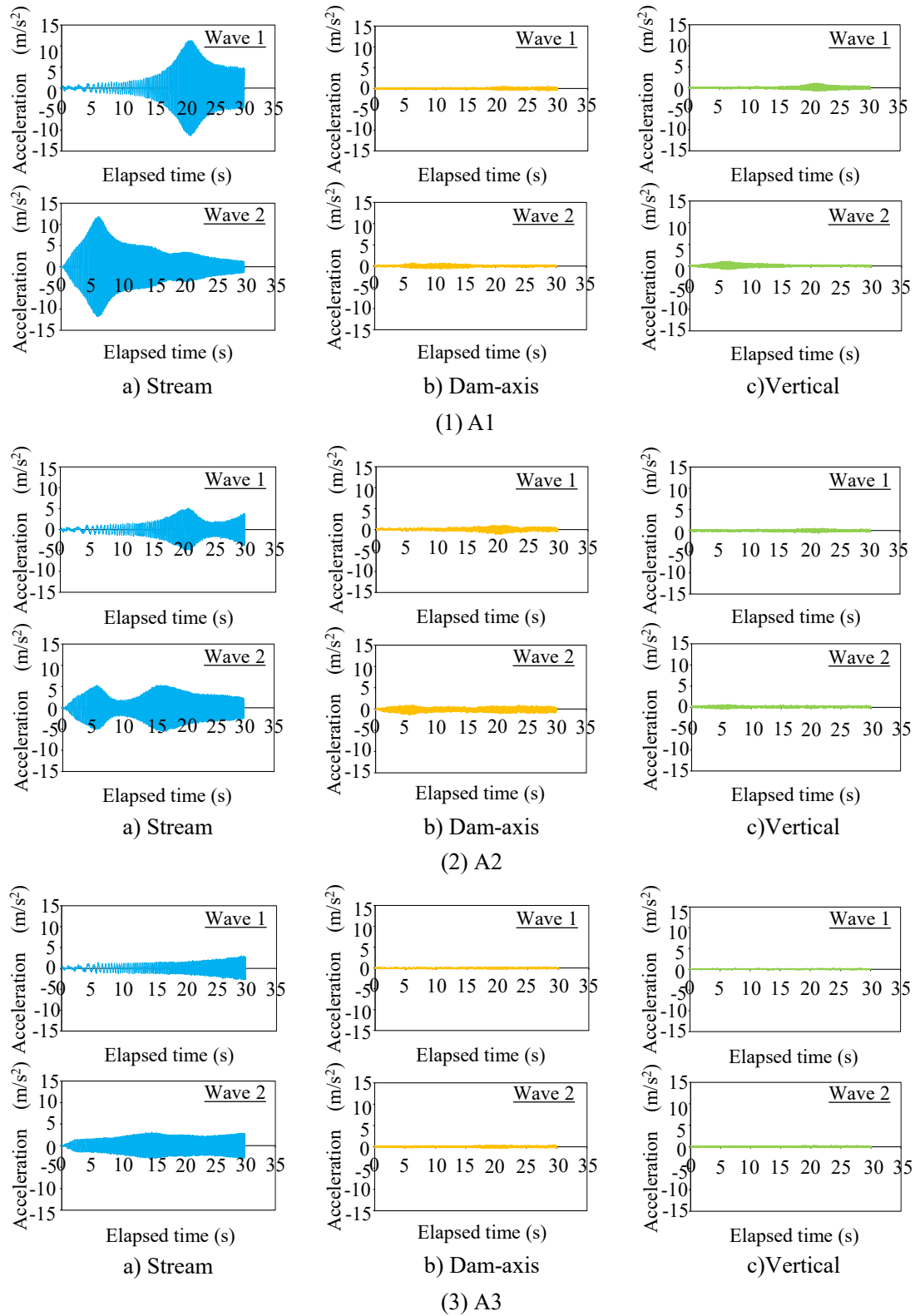


Figure 6.6 Time histories of acceleration responses of Model A
in Case 1 (Shaking in stream direction)

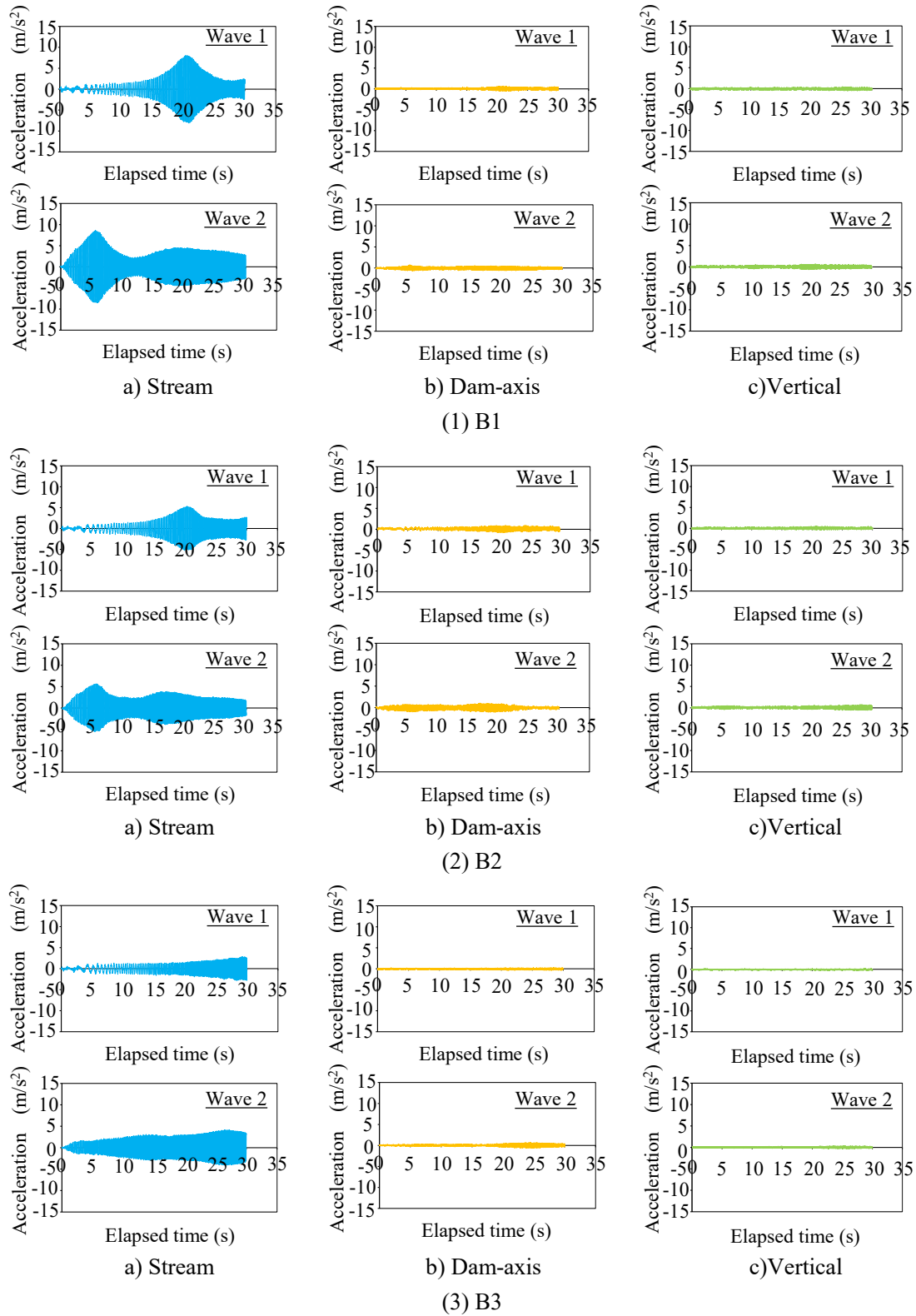


Figure 6.7 Time histories of acceleration responses of Model B in Case 1 (Shaking in stream direction)

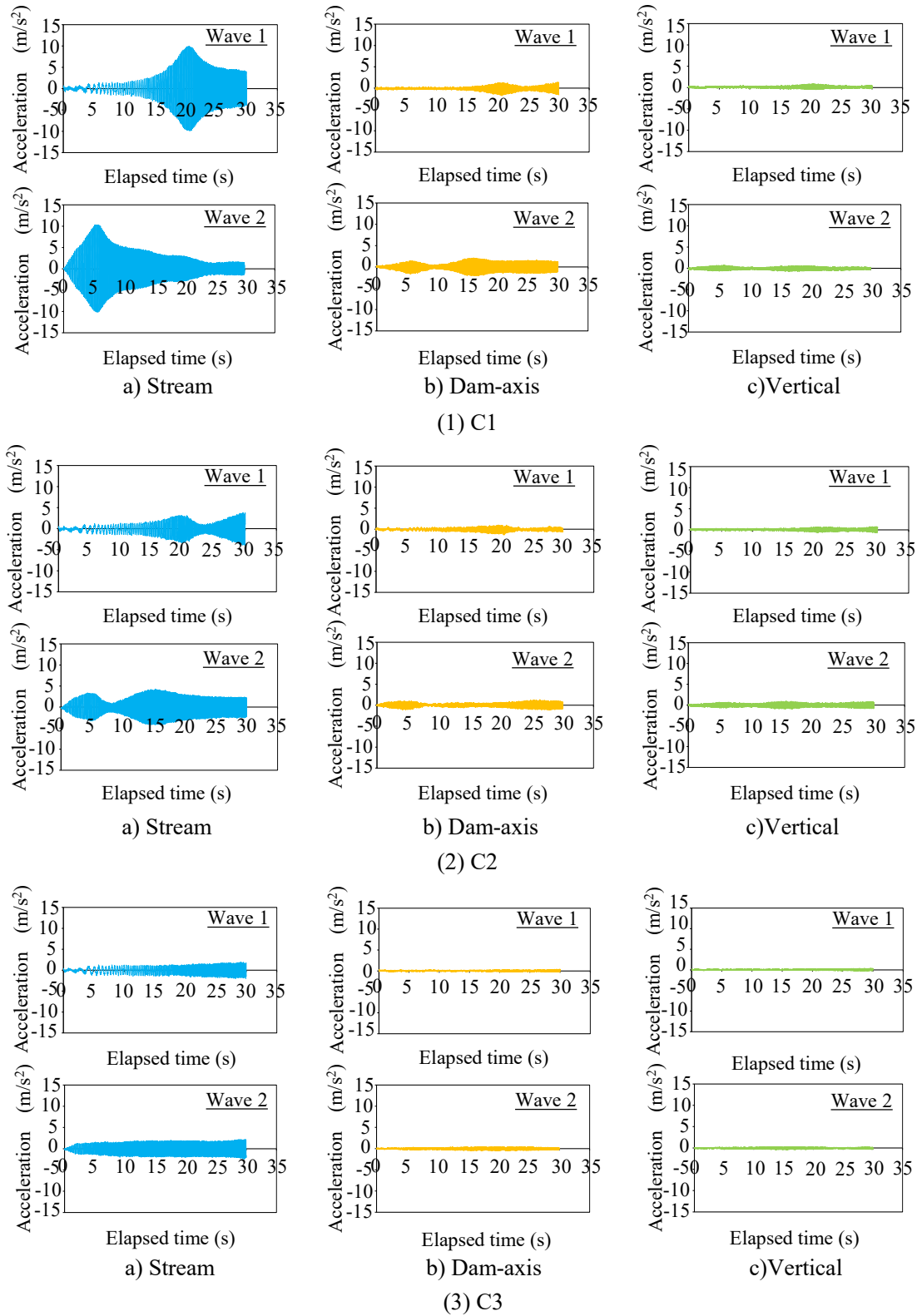


Figure 6.8 Time histories of acceleration responses of Model C
in Case 1 (Shaking in stream direction)

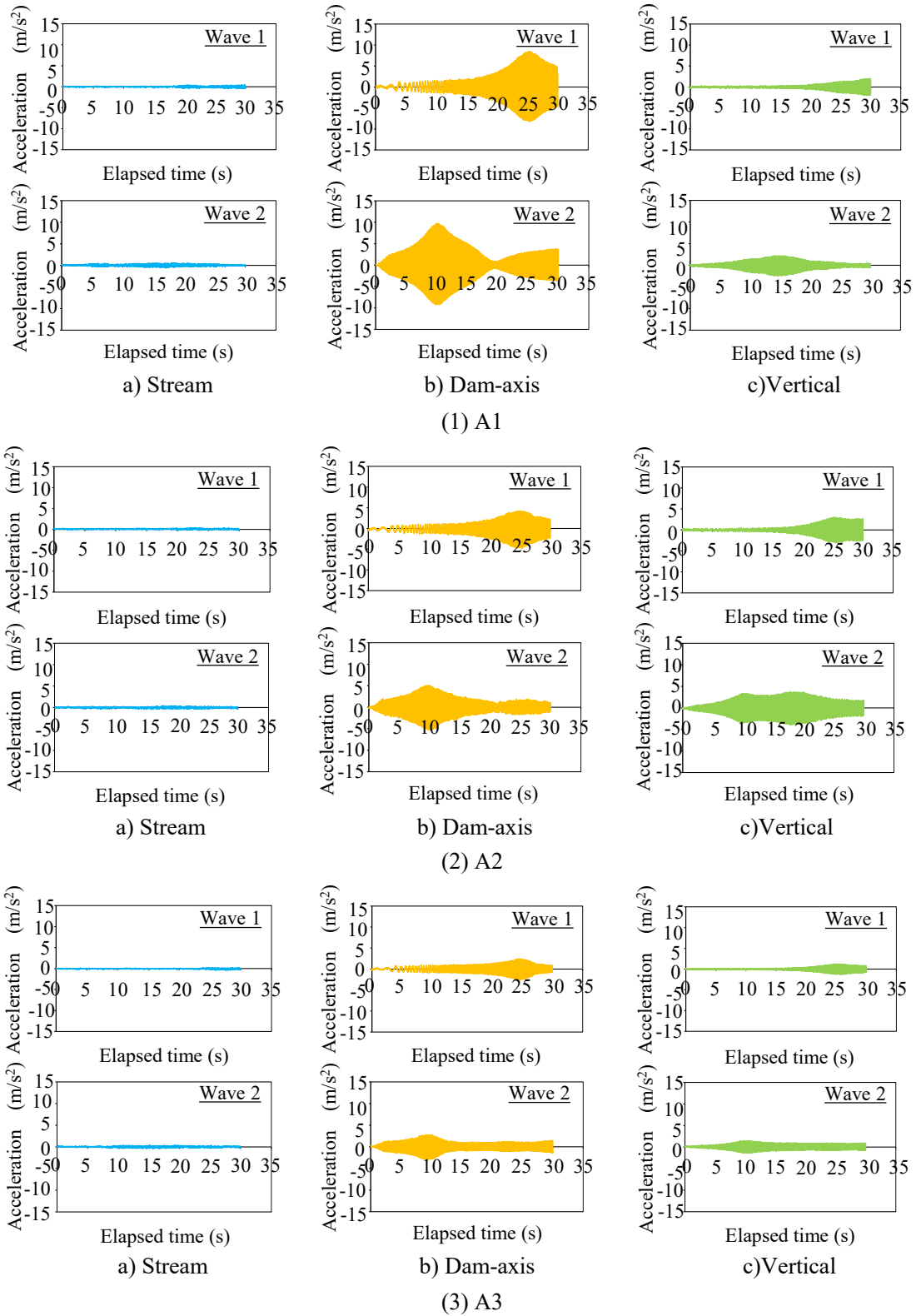


Figure 6.9 Time histories of acceleration responses of Model A in Case 2 (Shaking in dam-axis direction)

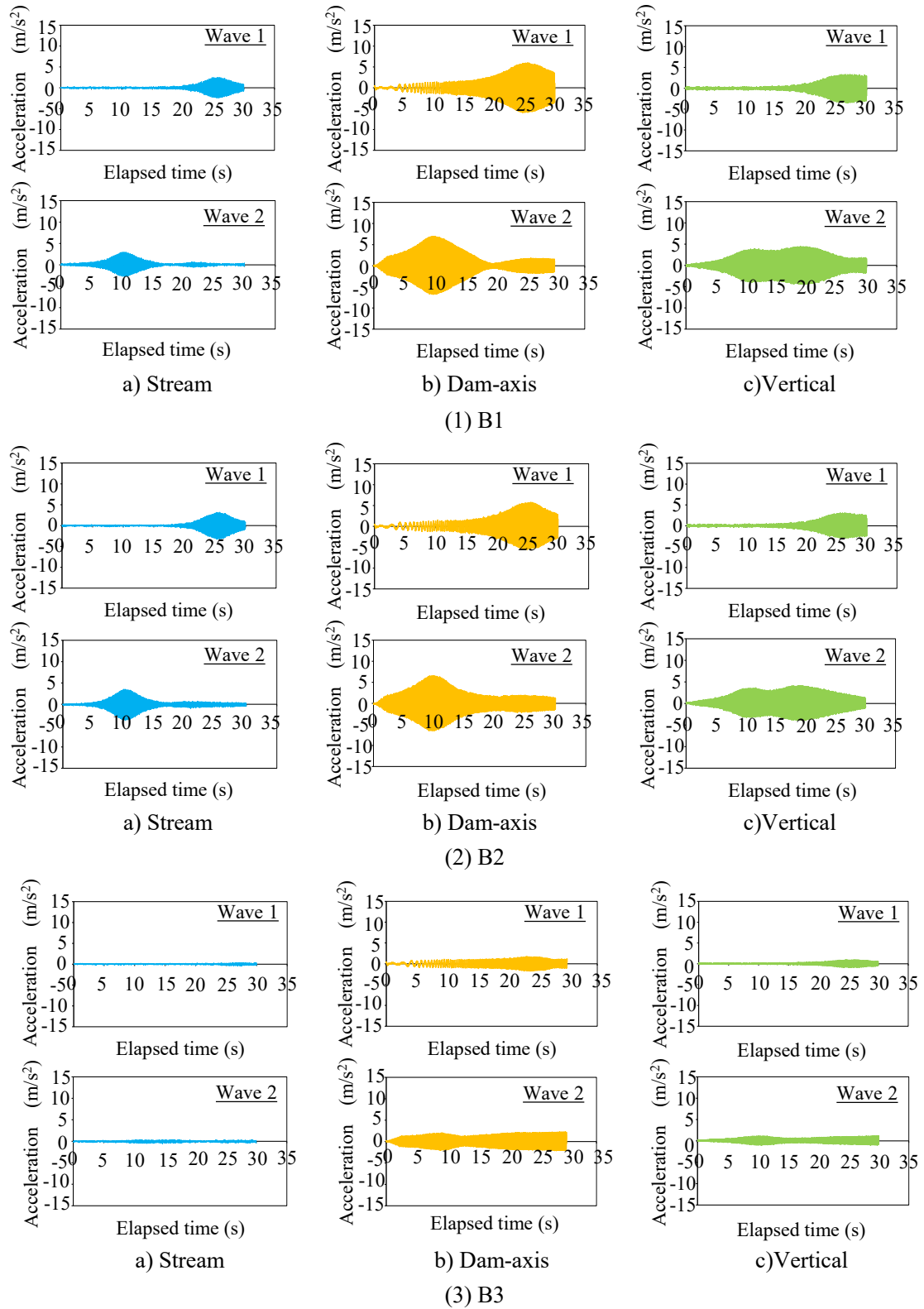


Figure 6.10 Time histories of acceleration response of Model B in Case 2 (Shaking in dam-axis direction)

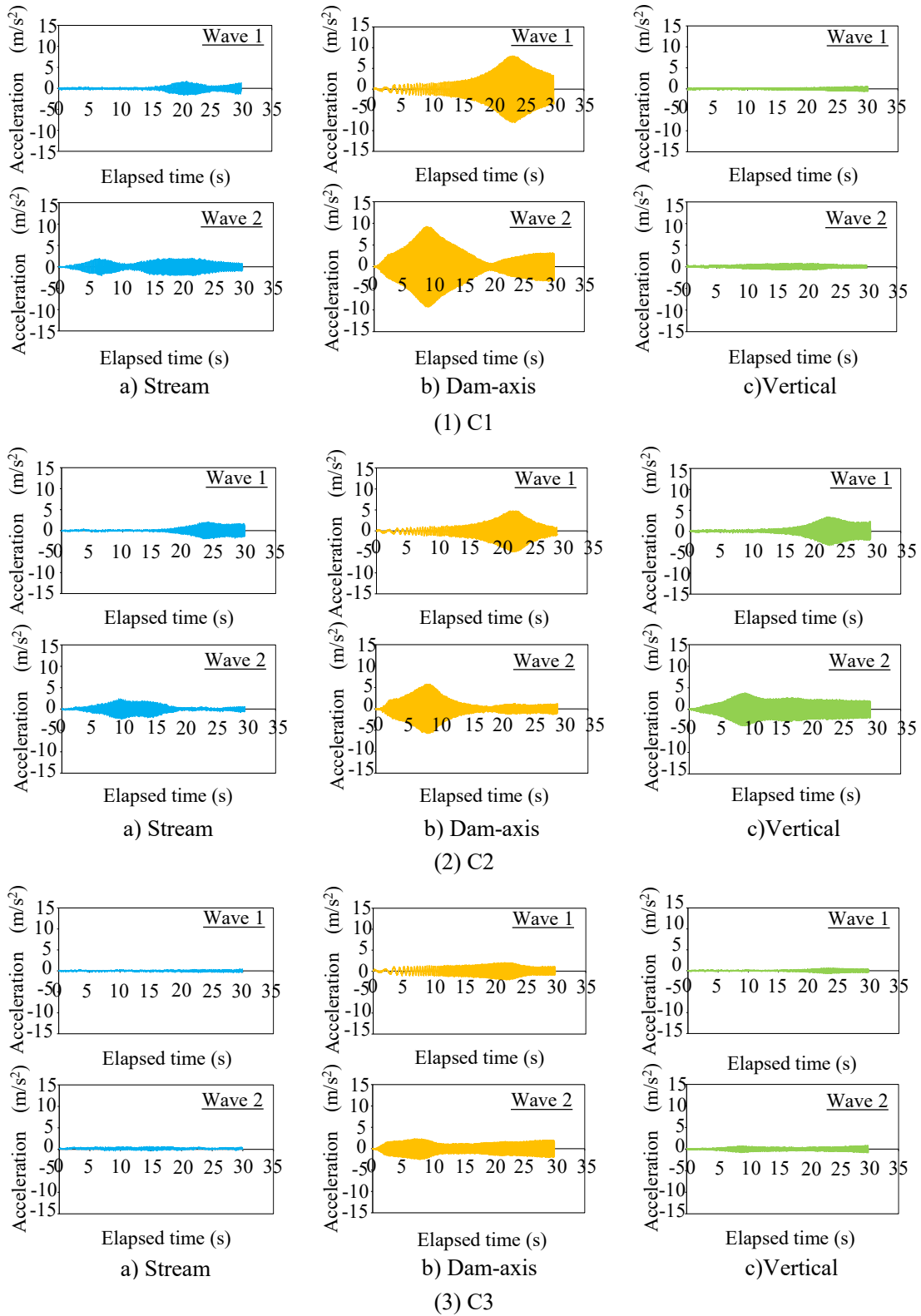


Figure 6.11 Time histories of acceleration response of Model C in Case 2 (Shaking in dam-axis direction)

6.2.1.2 Acceleration loci

Figures 6.12 to 6.14 show the acceleration loci at each point of the models in Case 1. Herein, the results of Wave 1 (frequency is from 0.6 to 9.3 Hz) and Wave 2 (frequency is from 4.7 to 14.0 Hz) are merged onto graphs, so that each graph shows the acceleration loci at the frequency sweep from 0.6 to 14.0 Hz. The two direction arrows in each graph show the shaking direction. In the graphs without arrows, the shaking direction is perpendicular to the plane. As shown in Figures 6.12 and 6.13, the responses of all points of Model A and Model B are only in the shaking direction. On the other hand, C1, which is right above the deepest part of the valley and at the center of the crest, shows behavior different from the other sites of Model A and Model B, as shown in Figure 6.14. The maximum acceleration in the shaking direction (stream) is 10.3 m/s^2 , and the acceleration in its orthogonal direction (dam-axis) shows -1.2 m/s^2 simultaneously. Similarly, the minimum acceleration in its orthogonal direction (dam-axis) is -2.0 m/s^2 , and the acceleration in the shaking direction shows 3.1 m/s^2 simultaneously. Thus, the C1 responses are not only in the shaking direction, but also in the direction orthogonal to the input wave. All points of the three models show few responses in the vertical direction.

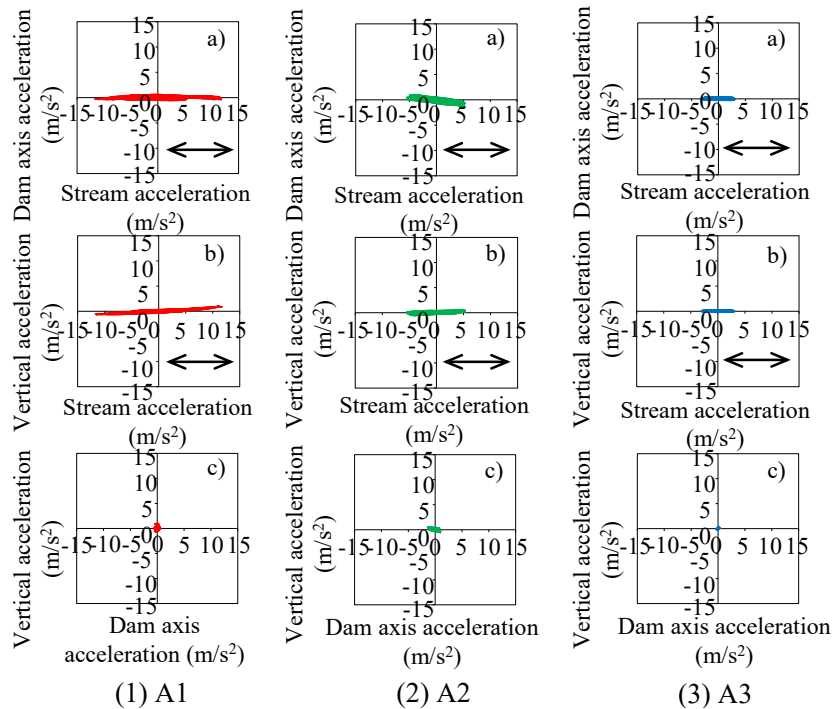


Figure 6.12 Loci of measured acceleration of Model A in Case 1

a) Stream and dam-axis plane, b) Stream and vertical plane, c) Dam-axis and vertical plane

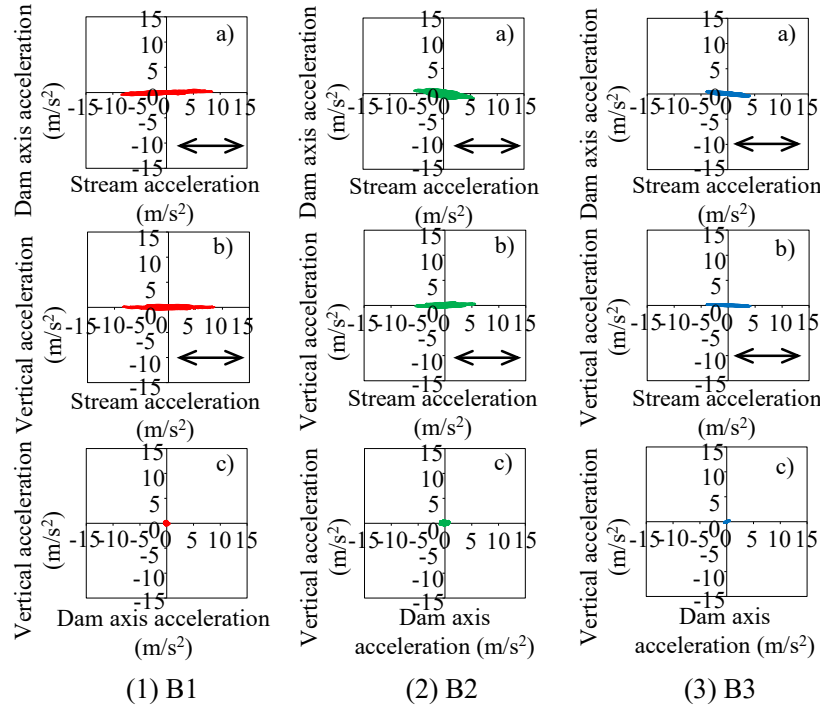


Figure 6.13 Loci of measured acceleration of Model B in Case 1

a) Stream and dam-axis plane, b) Stream and vertical plane, c) Dam-axis and vertical plane

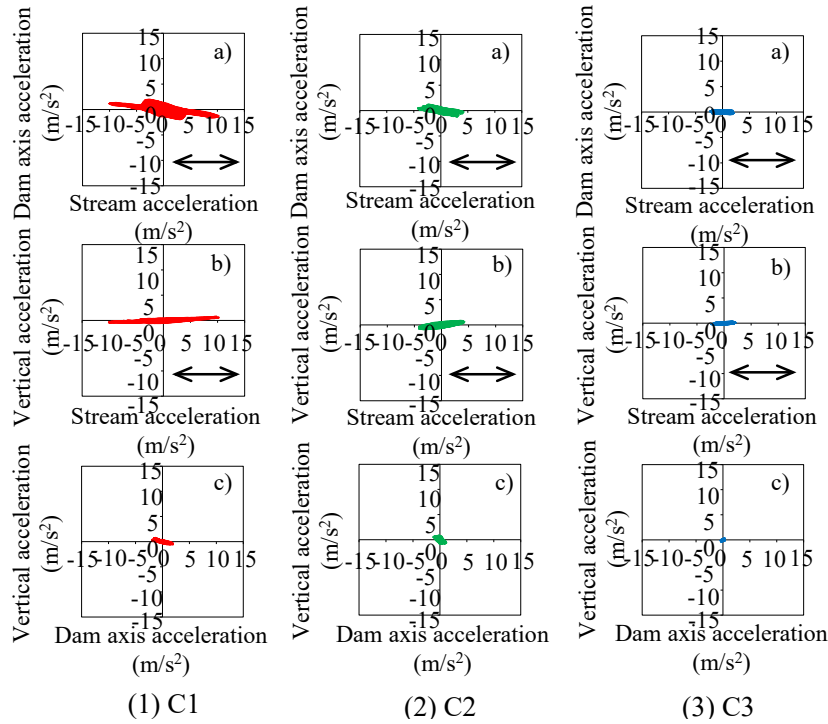


Figure 6.14 Loci of measured acceleration of Model C in Case 1

a) Stream and dam-axis plane, b) Stream and vertical plane, c) Dam-axis and vertical plane

Figures 6.15 to 6.17 show the acceleration loci at each site of the models in Case 2, as was seen in Figures 6.12 to 6.14. In Figure 6.15, the responses of all sites of Model A are only in the shaking direction on the horizontal plane. However, the maximum vertical accelerations are 2.4 m/s^2 at A1, right above the deepest part of the valley and the center of the crest, 3.9 m/s^2 at A2, and 1.4 m/s^2 at A3. From Figure 6.16, the B1 and B2 responses are in the shaking direction (dam-axis) and its orthogonal direction (stream) like the drawing of an ellipse on the horizontal plane. The dominant response directions of B1 and B2 are opposite in the dam-axis and vertical plane. At B1, right above the deepest part of the valley, the maximum acceleration (absolute value) in the shaking direction (dam-axis) is 6.9 m/s^2 , and that in its orthogonal direction (stream) is 3.0 m/s^2 . Similarly, at B2, the maximum acceleration (absolute value) in the shaking direction (dam-axis) is 6.5 m/s^2 and that in its orthogonal direction (stream) is 3.7 m/s^2 , which is the center of the crest. The maximum vertical accelerations (absolute values) are 4.4 m/s^2 at B1 and 4.1 m/s^2 at B2. C1 and C2 show the acceleration responses orthogonal to the shaking direction (stream), as shown in Figure 6.17. At C1, the minimum acceleration in the shaking direction (dam-axis) is -9.3 m/s^2 , and the acceleration in its orthogonal direction shows -1.2 m/s^2 simultaneously. At C2, the maximum acceleration in the shaking direction is 5.8 m/s^2 , and the acceleration in its orthogonal direction shows -1.1 m/s^2 simultaneously. Thus, the dominant response directions at C1 and C2 are opposite in the horizontal plane. The vertical accelerations at C1 and C3 are not remarkable, but the maximum vertical acceleration (absolute value) at C2 is 3.8 m/s^2 .

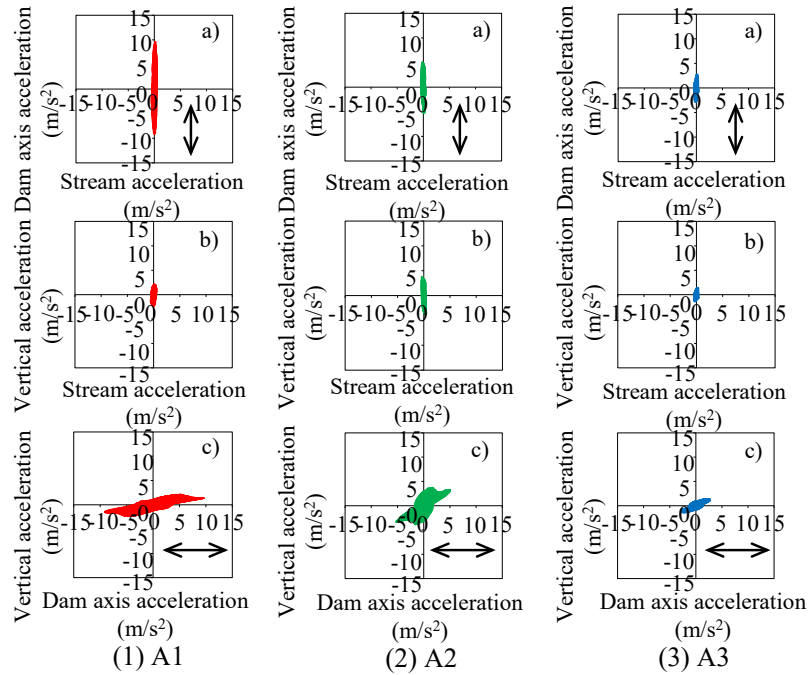


Figure 6.15 Loci of measured acceleration of Model A in Case 2

a) Stream and dam-axis plane, b) Stream and vertical plane, c) Dam-axis and vertical plane

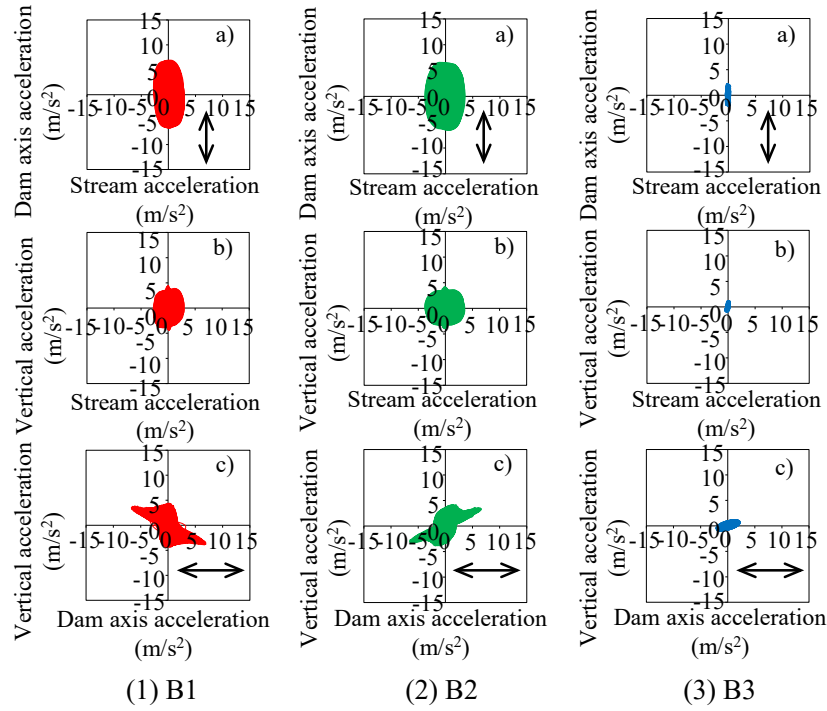


Figure 6.16 Loci of measured acceleration of Model B in Case 2

a) Stream and dam-axis plane, b) Stream and vertical plane, c) Dam-axis and vertical plane

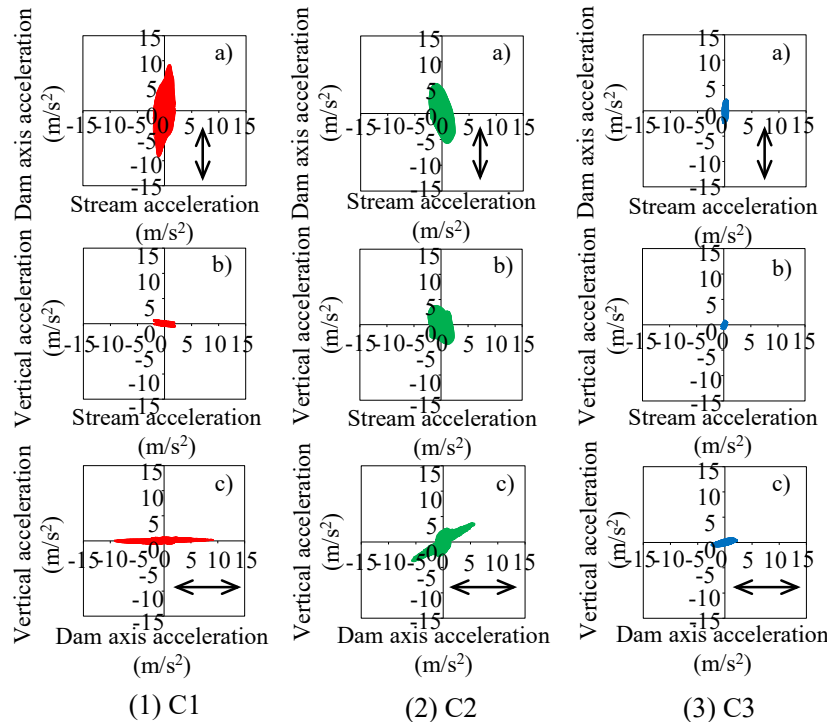


Figure 6.17 Loci of measured acceleration of Model C in Case 2

a) Stream and dam-axis plane, b) Stream and vertical plane, c) Dam-axis and vertical plane

6.2.1.3 Characteristics of acceleration response in each model

It is revealed that the tendencies of the acceleration responses are different at each site on the crest. When the shaking direction is stream, the sites which are right above the deepest part of the valley (A1, B1, and C1), show the maximum value of the acceleration response. Here, the response ratio is the acceleration at each site divided by the amplitude of the input wave (1.0 m/s^2). The maximum response ratios at A1, B1, and C1 are 11.8, 8.6 and 10.3, respectively (see Table 6.2). The center of the crest corresponds to the sites, which are right above the deepest part of the valley in Models A and C. However, in Model B, the site which is right above the deepest part of the valley is near the abutment. Thus, it is supposed that the value of the response ratio at B1 is smaller than that at either A1 or C1, because B1 is more strongly influenced by the abutment than A1 or C1.

The maximum accelerations are measured at the sites which are right above the deepest part of the valley (A1, B1, and C1) when the shaking direction is dam-axis (Case 2). This tendency is the same as in Case 1. The maximum response ratios at A1, B1, and C1 are 9.7, 6.9, and 9.3, respectively. B2, which is the center of the crest, shows a comparatively high response ratio of 6.5 which is almost the same as that at B1. The acceleration response arises in the direction orthogonal to the shaking direction in all models. In particular, the acceleration loci of Model B take an ellipse shape in the horizontal plane. The sites of all the models show remarkable responses in the vertical direction. It is supposed that the dam body shows complicated three-dimensional behavior because its deformation in the dam-axis is restricted by the abutment.

It is concluded that the sites at which the maximum acceleration values were recorded during the shaking in the stream and dam-axis directions, are right above the deepest part of the valley. Moreover, responses orthogonal to the shaking direction could arise depending on the shape of the dam and the direction of the input wave. The dam body shows especially complicated three-dimensional behavior when the shaking direction is dam-axis.

6.2.2 Relationships between acceleration response and frequency

Figures 6.18 and 6.19 show the Fourier spectra analyzed from the three components of the acceleration response at each point of the models by Fast Fourier Transform (FFT). Here, the Fourier spectra analyzed from the data of Wave 1 and Wave 2 are merged onto a graph. In these figures, the solid arrows show the 1st predominant frequencies and the broken arrows show the 2nd predominant frequencies. The gray arrows show the inflection points at which the Fourier spectra change in tendency from decreasing to increasing. The frequency values at those points are also shown. The predominant frequencies at each point are shown in Table 6.3.

Figure 6.18 shows that all points, except for C1, respond only in the stream direction through the targeted frequency range in Case 1. The predominant frequencies at A1, B1, and C1 are 6.3 Hz, 6.1 Hz, and 6.0 Hz, respectively, as shown in Table 6.3. The shapes of the Fourier spectra at A1 and C1 seem to be almost the same and show a remarkable single peak. However, the shape of the Fourier spectrum at B1 is different from the others. It shows the double peaks among the targeted range. The 1st predominant frequency is 6.1 Hz, and the 2nd predominant frequency is 11.5 Hz. The frequency of the inflection point at which the Fourier spectrum changes in tendency from decreasing to increasing is 8.6 Hz, as shown in Figure 6.18 (2).

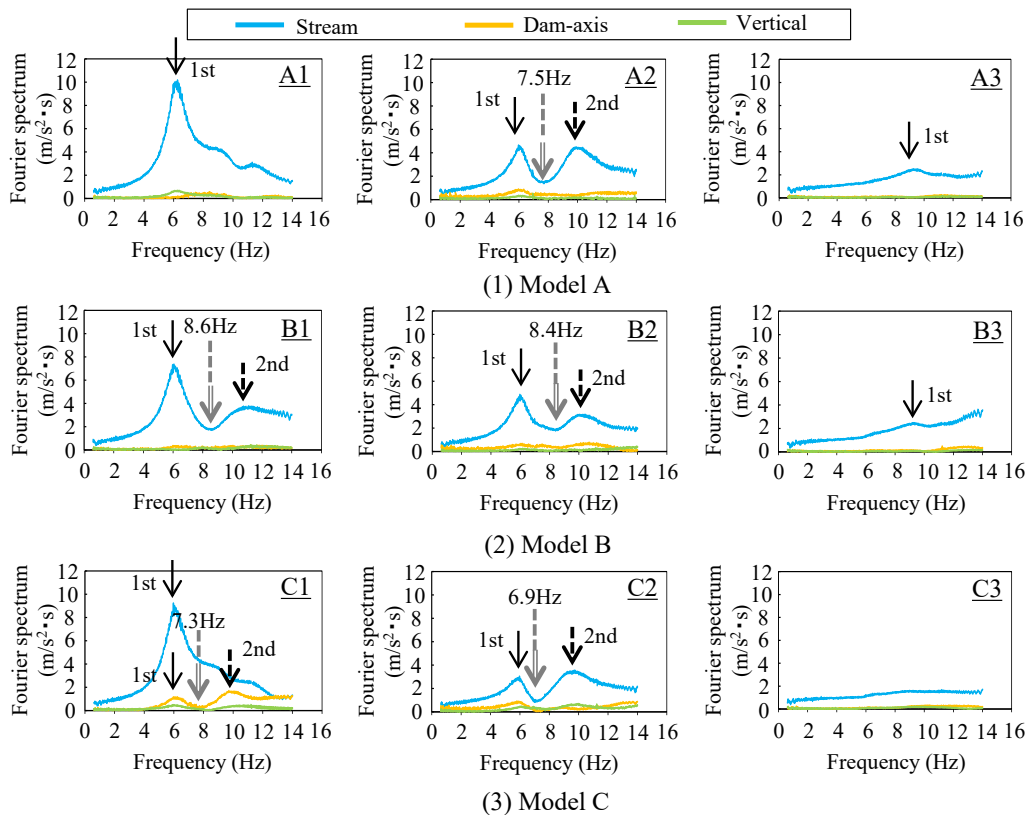


Figure 6.18 Fourier spectrum of amplitudes at each point in Case 1

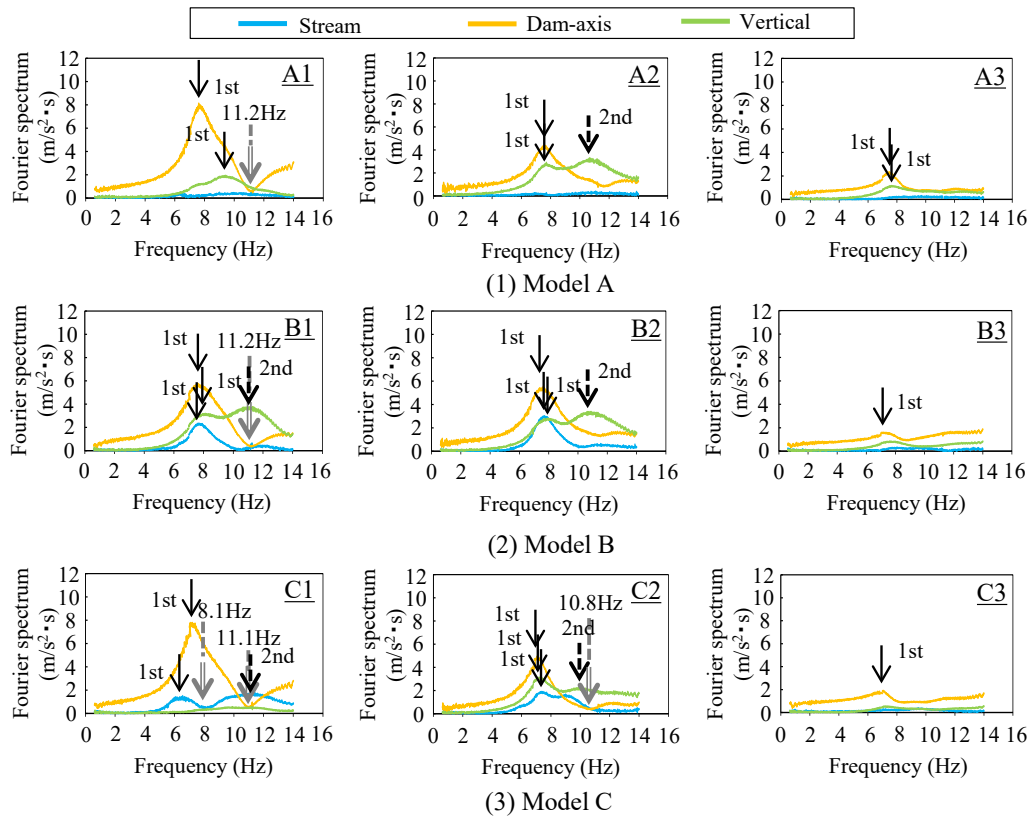


Figure 6.19 Fourier spectrum of amplitudes at each point in Case 2

Table 6.3 Measured dominant frequencies of each model

	Case 1			Case 2		
	Stream	Dam-axis	Vertical	Stream	Dam-axis	Vertical
A1	6.3	-	-	-	7.6	9.4
A2	6.0 (9.9)	-	-	-	7.6	7.7 (10.7)
A3	9.2	-	-	-	7.5	7.6
B1	6.1 (11.5)	-	-	7.6	7.6	8.2 (11.3)
B2	6.0 (10.0)	-	-	7.7	7.6	8.0 (10.8)
B3	9.3	-	-	-	7.2	-
C1	6.0	6.2 (9.7)	-	6.5 (11.1)	7.8	-
C2	6.0 (9.7)	-	-	7.4	7.0	7.1 (10.0)
C3	-	-	-	-	7.0	-

Unit: Hz, (): 2nd predominant frequency, - : undetected

A1 and C1 are just above the deepest part of the valley and at the center of the crest, so they are less influenced by the abutment. On the other hand, B1 is just above the deepest part of the valley, but near the abutment. B2 is at the center of the crest, but the depth is lower. These points are affected by the restriction of the abutment. The shapes of the Fourier spectra at A2, B2, and C2 show the double peaks, the same as B1 and B2. It is considered that the shape of the Fourier spectrum at the site which is less influenced by the abutment shows a remarkable single peak, while the shape of the Fourier spectrum at the site which is influenced by the restriction of the abutment shows double peaks or no remarkable peak.

Figure 6.19 shows the maximum Fourier spectrum recognized at each site in Case 2; they are all right above the deepest part of the valley (A1, B1, and C1). The 1st predominant frequencies at A1, B1, and C1 are 7.6 Hz, 7.6 Hz, and 7.8 Hz, respectively (see Table 6.3). These sites show no response in the shaking direction (dam-axis) at a frequency of around 11.2 Hz, as shown by the double arrows in Figure 6.19. In addition, at A1 and B1, the responses in the stream direction are not recognized. These sites are considered as the nodes of the horizontal vibration at this frequency. The maximum responses in the stream direction (orthogonal to the input wave) are recognized at B2 and C2 in Models B and C. Their predominant frequencies are 7.7 Hz and 7.4 Hz, respectively (see Table 6.3). In Model A, no remarkable response in the stream direction is detected. Thus, the response property at a site on the crest will change according to the frequency of the input wave, and the seismic behavior of a dam body will show complicated three-dimensional behavior at specific frequencies. In Case 2, vertical responses arise remarkably at A2, B1, B2, and C2. The vertical responses at these sites exceed the horizontal responses at specific frequencies, which are the 2nd predominant frequencies in the vertical direction (A2: 10.7 Hz, B1: 11.3 Hz, B2: 10.8 Hz, and C2: 10.0 Hz; see Table 6.3).

It is revealed that the maximum responses in all the models arose at the sites which are right above the deepest part of the valley, in spite of the difference in the input wave direction. The predominant frequencies at these sites are similar for each input wave direction. A complex shape of the dam body, such as Model C, can cause a response in the direction orthogonal to the shaking direction when the shaking direction is stream. However, complicated three-dimensional responses can arise not only when the shape of the dam body is complex, like Model B and Model C, but also when the shape is simple, like Model A at a specific frequency, when the shaking direction is the dam-axis.

6.2.3 Seismic response behavior on crest of dam body at a specific frequency

6.2.3.1 Model A

The dam models show the complicated response behavior at specific frequencies according to their shape and the shaking direction. Therefore, the acceleration vector in each model is verified according to the specific conditions (shaking direction and specific frequency of the input wave). In particular, specific conditions are selected for which a site on the crest of the dam body shows no response during shaking.

For Model A, little response arises at either A1 or A3, while the maximum vertical response arises at A2 when the shaking direction is the dam-axis and the frequency of the input wave is 10.7 Hz (Figure 6.19 (1) and Table 6.3). Figure 6.20 shows the acceleration vectors under these conditions. The solid black arrows indicate the vectors when the vertical acceleration (absolute value) becomes the maximum at A2. The dotted gray arrows indicate the vectors when the vertical acceleration at A2 reaches the opposite peak after a half wave. From Figure 6.20, it is revealed that little response arises at either A1 or A3 and that the responses in the dam-axis and vertical directions arise at A2. The vertical component is especially dominant in the response at A2. This behavior shows that the crest of Model A will heave under specific conditions of the input wave.

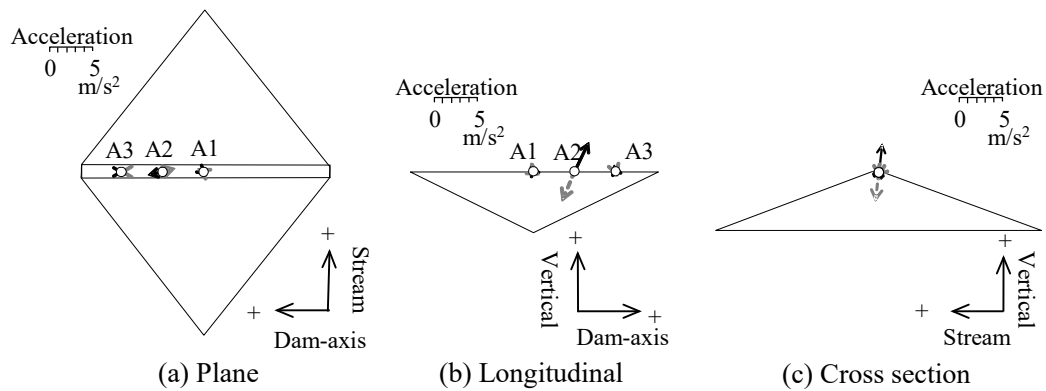


Figure 6.20 Acceleration vectors of Model A (Case 2, 10.7 Hz)

6.2.3.2 Model B

For Model B, the responses at B1 and B2 show the opposite directions in the dam-axis and vertical planes, as shown in Figure 6.16, when the shaking direction is dam-axis (Case 2). And, the responses in the dam-axis direction become the maximum at B1 and B2 when the frequency of the input waves is 7.6 Hz (Table 6.3). Figure 6.21 shows the acceleration vectors when the shaking direction is the dam-axis and the specific frequency of the input wave is 7.6 Hz. The solid black arrows show the vectors when the acceleration (absolute value) in the dam-axis becomes the maximum at B2. The dotted gray arrows show the vectors when the dam-axis acceleration at B2 reaches the opposite peak after a half wave. From Figure 6.21 (b), it is revealed that the signs of the responses in the vertical direction are opposite at B1 and B2. And, the signs of the responses in the stream direction are opposite at B1 and B2, as shown in Figure 6.21 (a). Hence, the responses at B1 and B2 show opposite signs not only in the vertical direction, but also in the stream direction. This behavior indicates that the crest of Model B will show a complicated three-dimensional response under specific conditions of the input wave.

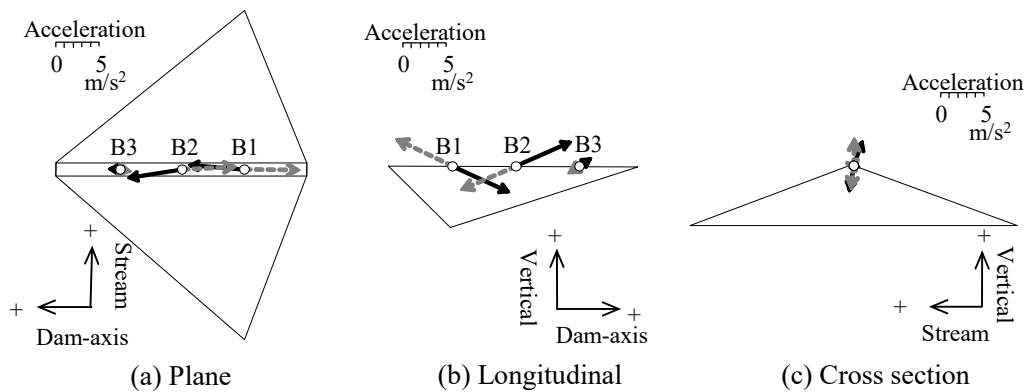


Figure 6.21 Acceleration vectors of Model B (Case 2, 7.6 Hz)

6.2.3.3 Model C

For Model C, the responses at C1 and C2 show the opposite directions in the stream and dam-axis planes, as shown in Figure 6.17, when the shaking direction is the dam-axis (Case 2). The responses in the dam-axis direction become the maximum at C1 and C2 when the frequencies of the input waves are 7.8 Hz and 7.0 Hz, respectively (Table 6.3). Figure 6.22 shows the acceleration vectors when the shaking direction is the dam-axis and the specific frequency of the input wave is 7.8 Hz. Figure 6.23 shows the acceleration vectors when the shaking direction is the dam-axis and the specific frequency of the input wave is 7.0 Hz. The solid black arrows in Figures 6.22 and 6.23 show the vectors when the acceleration (absolute values) in the dam-axis become the maximum at C1 and C2, respectively. The dotted gray arrows in Figures 6.22 and 6.23 show the vectors when the dam-axis accelerations at C1 and C2, respectively, reach the opposite peak after a half wave. The acceleration vectors in both cases show the same tendency, namely, that little response arises in the vertical direction at either C1 or C3, but that the vertical response at C2 is remarkable. From Figures 6.22 (a) and 6.23 (a), it is seen that the signs of the responses in the stream direction are opposite at C1 and C2. These results indicate that the crest of Model C will show complicated three-dimensional behavior under specific conditions of the input wave.

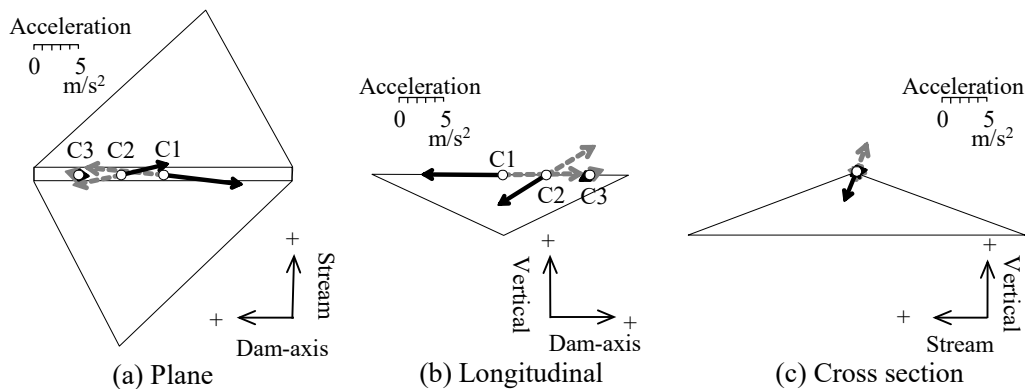


Figure 6.22 Acceleration vectors of Model C (Case 2, 7.8 Hz)

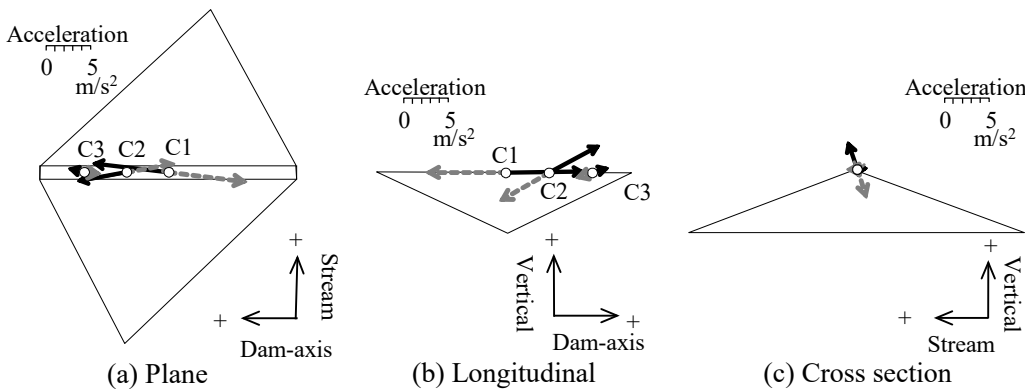


Figure 6.23 Acceleration vectors of Model C (Case 2, 7.0 Hz)

6.3 Results and considerations of shaking tests under constant mono frequency

From shaking tests under the sweep frequency condition, it is revealed that the predominant frequencies in the stream are 7.6 Hz and 6.5 Hz for Models B and C, respectively (see Table 6.3), when the shaking direction is the dam-axis. At these frequencies, the crest of the models show complicated three-dimensional behavior, because the responses arise in the direction orthogonal to the input. Therefore, shaking tests are carried out under the condition that the direction of the input wave is the dam-axis and its frequency is constant. The input waves are sine waves with amplitude of 1.0 m/s^2 , and their frequencies are 6.5 Hz (Case 3) and 7.6 Hz (Case 4).

Figures 6.24 and 6.25 show the time series data, while Figures 6.26 and 6.27 show the acceleration loci in Cases 3 and 4, respectively. In both cases, Model A responds only in the dam-axis in the horizontal plane. On the other hand, the shapes of the acceleration loci in Models B and C are different from that in Model A. Both models show the stream response, which is orthogonal to the input wave. In Model B, the acceleration loci show eclipse shapes in the horizontal plane at B1 and B2, and their response directions are opposite. Similarly, in Model C, the acceleration loci show eclipse shapes in the horizontal plane at C1 and C2, and their response directions are opposite. These results indicate that vibration nodes in the stream and dam-axis planes exist between these points, namely B1 and B2 in Model B and C1 and C2 in model C.

A vertical response arises in all models. In particular, the vertical responses at A2, B1, B2, and C2 are remarkable, and the acceleration loci at these points show eclipse shapes in the stream and vertical planes. And, it is revealed that no vertical response arises at either A1 or C1 for either case. It is thought that A1 and C1 are nodes of vertical vibration at these frequencies. The response directions are opposite in the dam-axis and vertical planes at B1 and B2, as shown in Figures 6.26 (2) c) and 6.27 (2) c). These results indicate that vibration nodes exist in the dam-axis and vertical planes between B1 and B2.

It is clarified that the crest of the models will show complicated three-dimensional behavior at specific frequencies when the direction of the input wave is the dam-axis. In such situations, vibration nodes will appear on the crest. At the vibration nodes, no response will arise against the input wave. Therefore, care must be taken when the data for real dams, involving measurement by seismometers, are analyzed. Attention must also be paid to the locations of the seismometers.

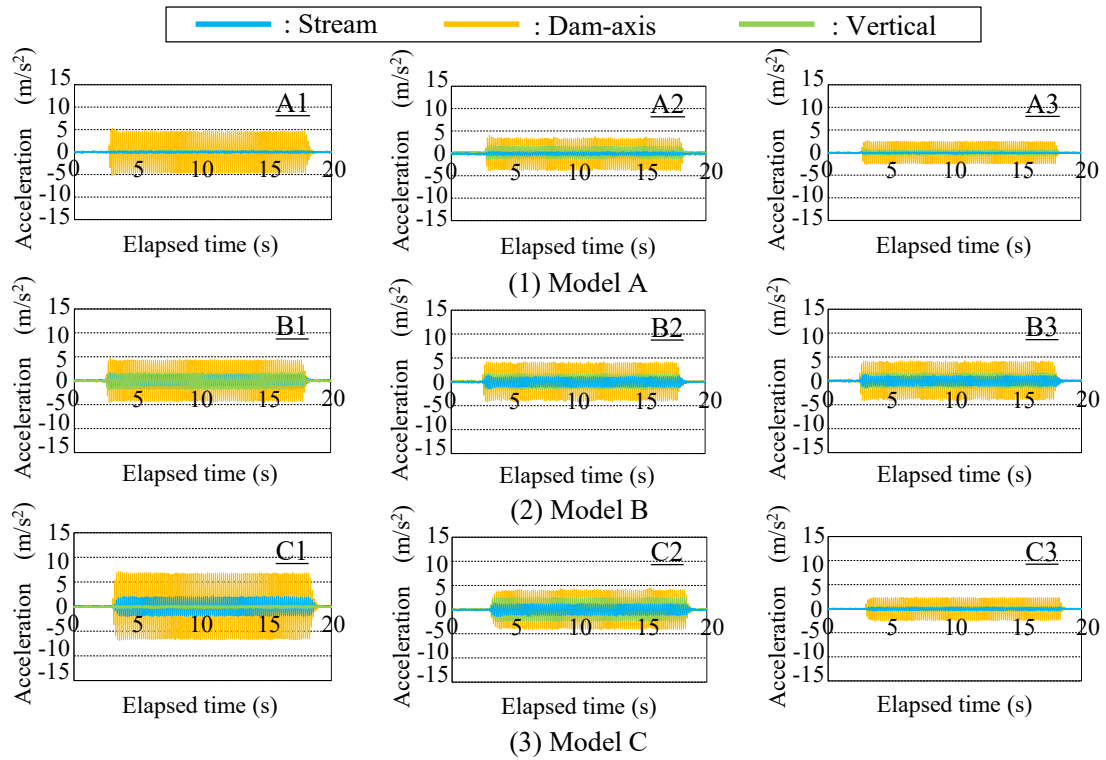


Figure 6.24 Time histories of acceleration response in Case 3

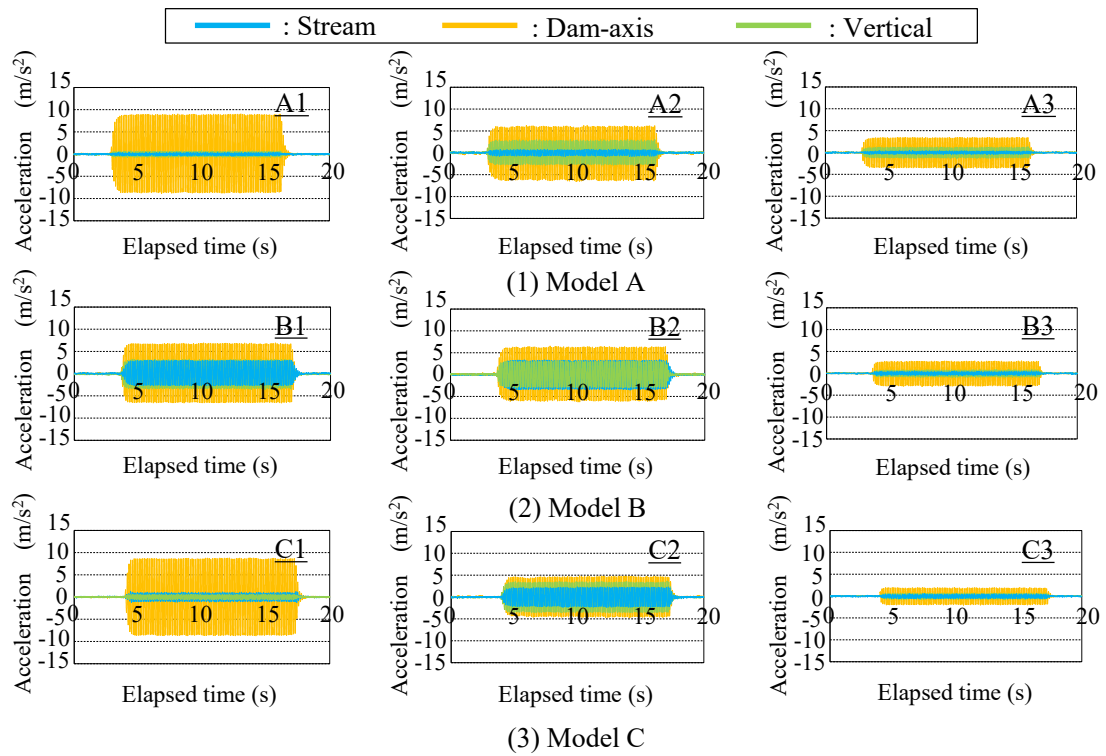


Figure 6.25 Time histories of acceleration response in Case 4

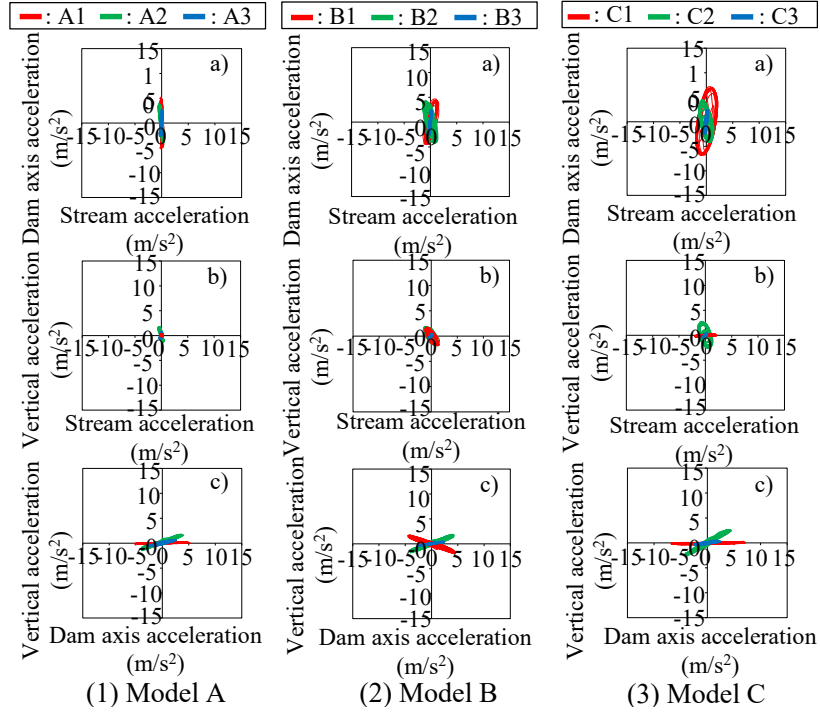


Figure 6.26 Loci of measured accelerations in Case 3

a) Stream and dam-axis plane, b) Stream and vertical plane, c) Dam-axis and vertical plane

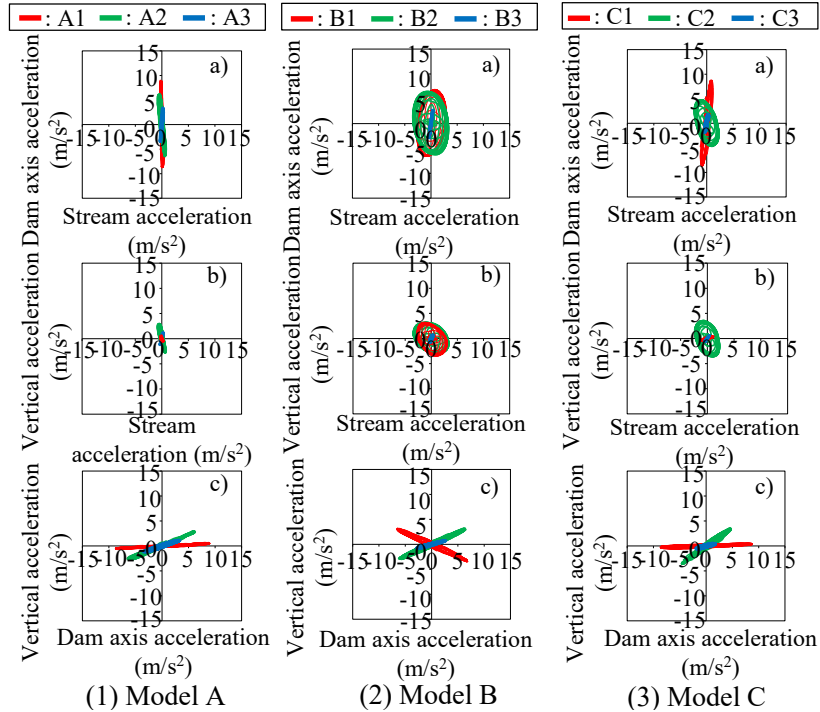


Figure 6.27 Loci of measured accelerations in Case 4

a) Stream and dam-axis plane, b) Stream and vertical plane, c) Dam-axis and vertical plane

6.4 Seismic behavior in a real dam

From the results of shaking tests, it is clarified that the seismic behavior of a dam body will change according to its shape and the direction of the input wave. In particular, the seismic behavior on the crest shows complicated three-dimensional behavior at specific frequencies when the direction of the input wave is the dam-axis. Therefore, the seismic behavior in a real dam is verified and compared with the results of shaking tests.

Dam A is a rock-fill type of dam. Its height is 84.5 m, its crest length is 262.0 m, the ratio of its crest length to its height is 3.10, the width of its crest is 10.0 m, and the volume of the dam body is 1,729,000 m³. The schematics of Dam A are shown in Figure 6.28. Seismic data for Dam A from past major earthquakes were reported as digital data (Masukawa et al., 2002, Masukawa and Nakanishi, 2005).

Dam A is located in a steep V-shaped valley through which a river runs and meanders such that its width becomes narrow. The dam axis is orthogonal to the axis of the narrow ridge at the left abutment. The slope of the left abutment is steeper than that of the right abutment, and the shape of the longitudinal profile is asymmetric with the maximum cross section, as shown in Figure 6.28 (b). Dam A makes contact with the narrow ridge at the left abutment. Its shape is asymmetric with the maximum cross section and longitudinal profile, because the river channel meanders and the left abutment is very steep. Thus, the shape of Dam A is very complicated. In the longitudinal profile, its shape is similar to Model B, while in the plane, it is similar to Model C.

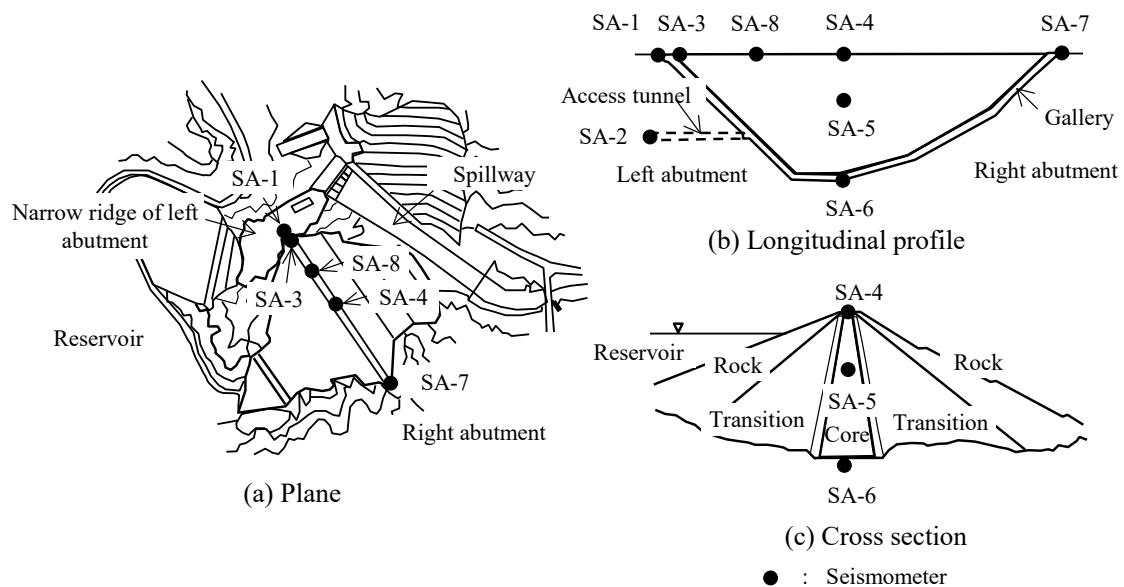


Figure 6.28 Schematics of Dam A

In Dam A, eight seismometers are set densely because of the complicated dam site condition and regional characteristics, where earthquakes frequently occur; five of the seismometers are arranged on the crest. Dams, for which several seismometers are arranged on the crest, are very rare. Thus, it is supposed that this dam is suitable for a comparison with the shaking tests.

Table 6.4 shows the maximum accelerations in four earthquakes. SA-6 is at the base of the dam body. SA-1, SA-3, SA-8, SA-4, and SA-7 are set at the crest elevation. SA-1 and SA-4 are arranged in the left and right abutments, respectively. SA-3, SA-8, and SA-4 are arranged on the crest of the dam body, and SA-4 is located in the center of the crest. The shaded parts of Table 6.4 show the maximum accelerations of the three components in each seismometer. Figures 6.29 to 6.32 show the time history records for these earthquakes occurring on 1987/2/6, 1987/4/7, 1987/4/23, and 2003/5/26, respectively. All four earthquakes exceed the criterion acceleration (0.25 m/s^2) at the base of the dam body (SA-6), above which an emergency inspection is required. From Table 6.4, it is recognized that the maximum acceleration value at SA-6 is the dam-axis component in all the earthquakes. Therefore, it is assumed that the dominant direction of all the earthquakes is dam-axis, and the seismic behavior of Dam A can be compared with the results of Case 2 in the shaking tests.

Table 6.4 Measured maximum acceleration at each point in Dam A

		SA-6	SA-1	SA-3	SA-8	SA-4	SA-7
1987/2/6	Stream	0.16	0.49	0.90	1.25	1.08	0.29
	Dam-axis	0.26	0.95	1.27	1.18	1.69	0.73
	Vertical	0.11	0.32	0.44	0.84	1.07	-
1987/4/7	Stream	0.76	1.35	1.91	2.72	1.18	0.45
	Dam-axis	0.96	2.89	3.33	2.06	3.01	1.66
	Vertical	0.40	0.78	2.17	2.03	1.42	-
1987/4/23	Stream	0.19	0.76	1.64	1.23	1.03	0.28
	Dam-axis	0.30	0.95	1.27	1.17	1.50	0.56
	Vertical	0.17	0.66	0.89	0.70	1.22	0.41
2003/5/26	Stream	0.16	0.74	1.44	1.29	0.99	0.29
	Dam-axis	0.26	0.73	1.00	3.04	3.79	0.48
	Vertical	0.11	0.32	0.48	0.70	0.80	0.31

Unit: m/s^2 , : maximum value of three components

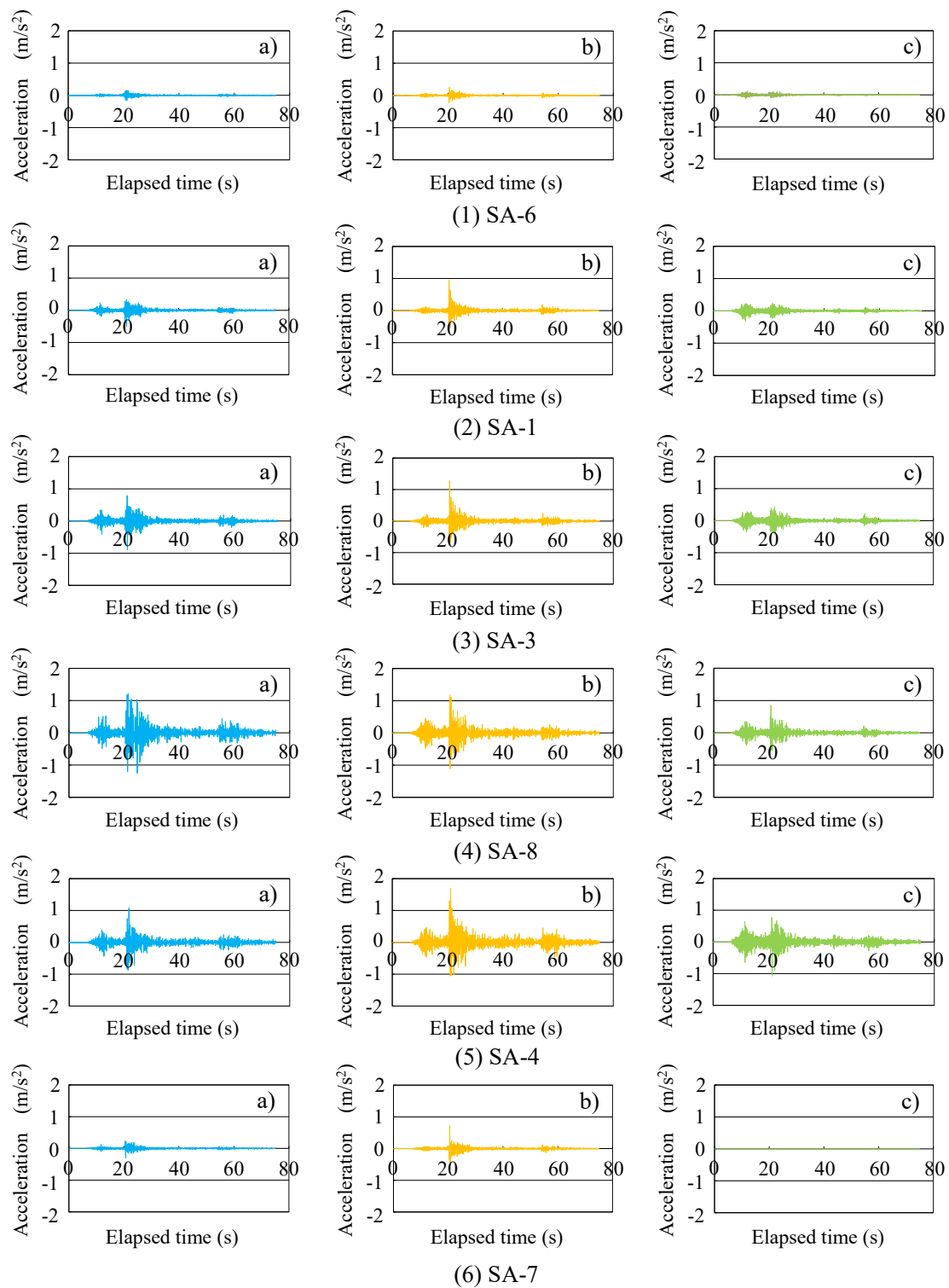


Figure 6.29 Time histories of acceleration response at each point of Dam A on 1987/2/6
a) Stream, b) Dam-axis, c) Vertical

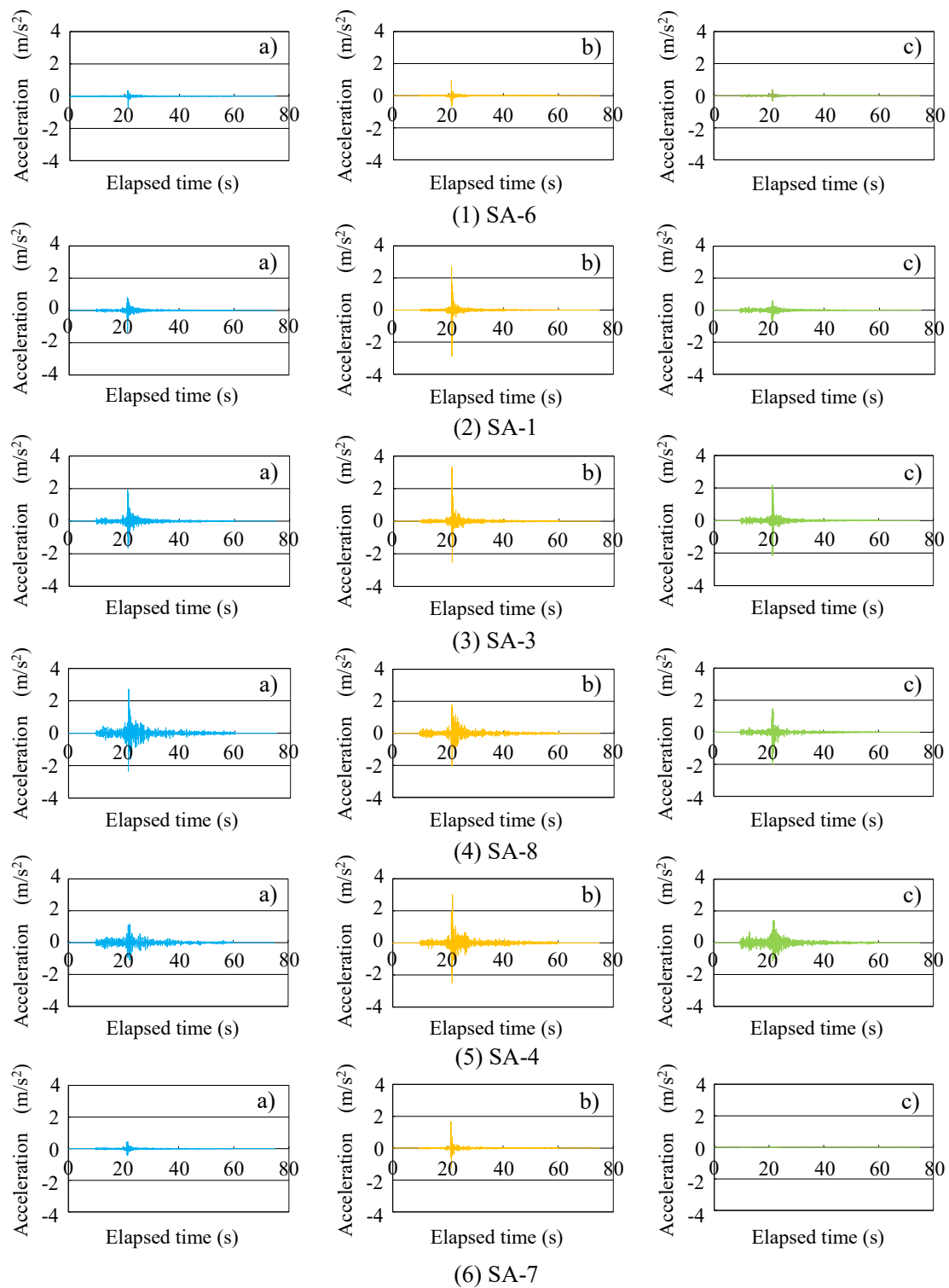


Figure 6.30 Time histories of acceleration response at each point of Dam A on 1987/4/7
a) Stream, b) Dam-axis, c) Vertical

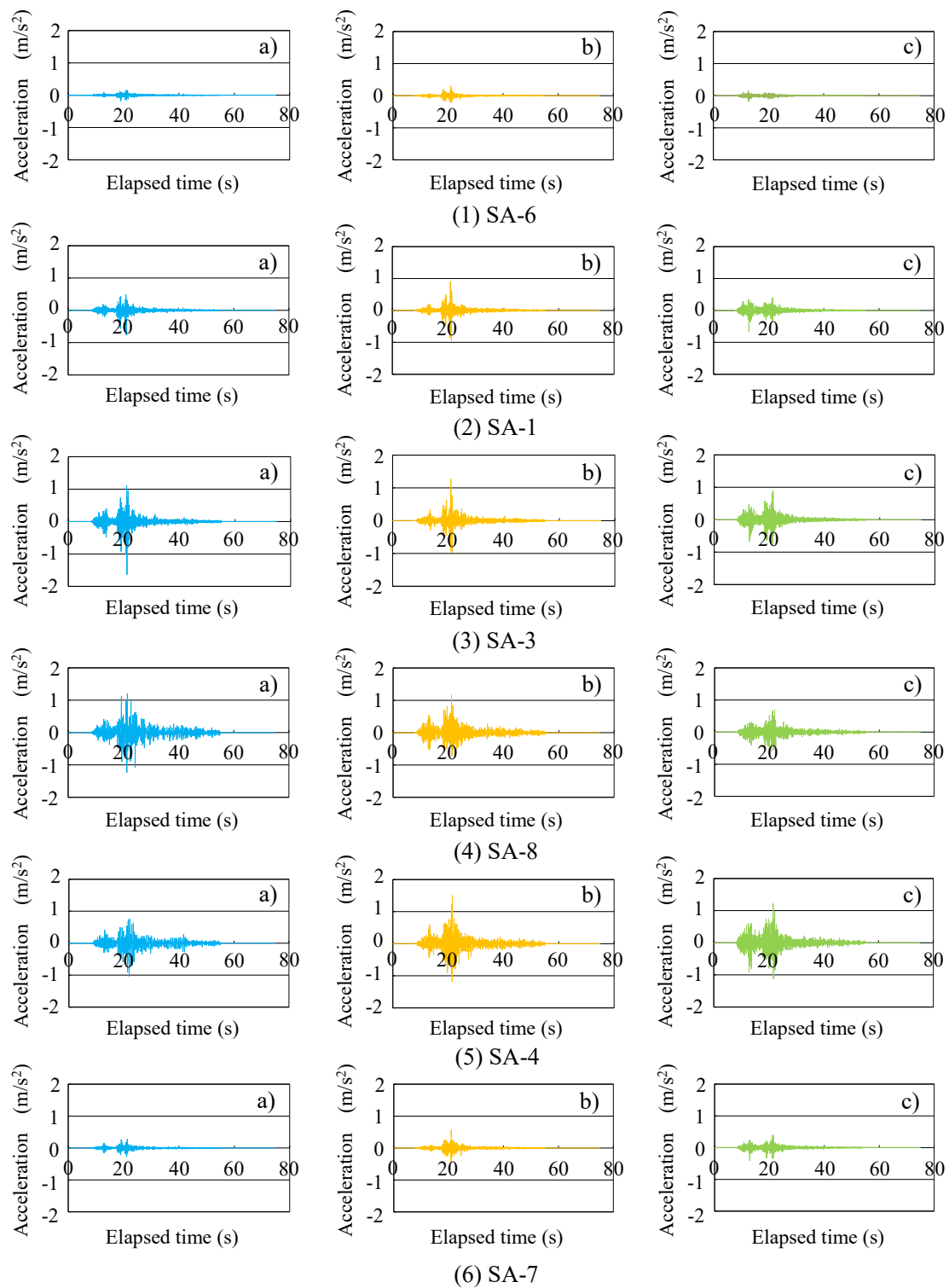


Figure 6.31 Time histories of acceleration response at each point of Dam A on 1987/4/23
a) Stream, b) Dam-axis, c) Vertical

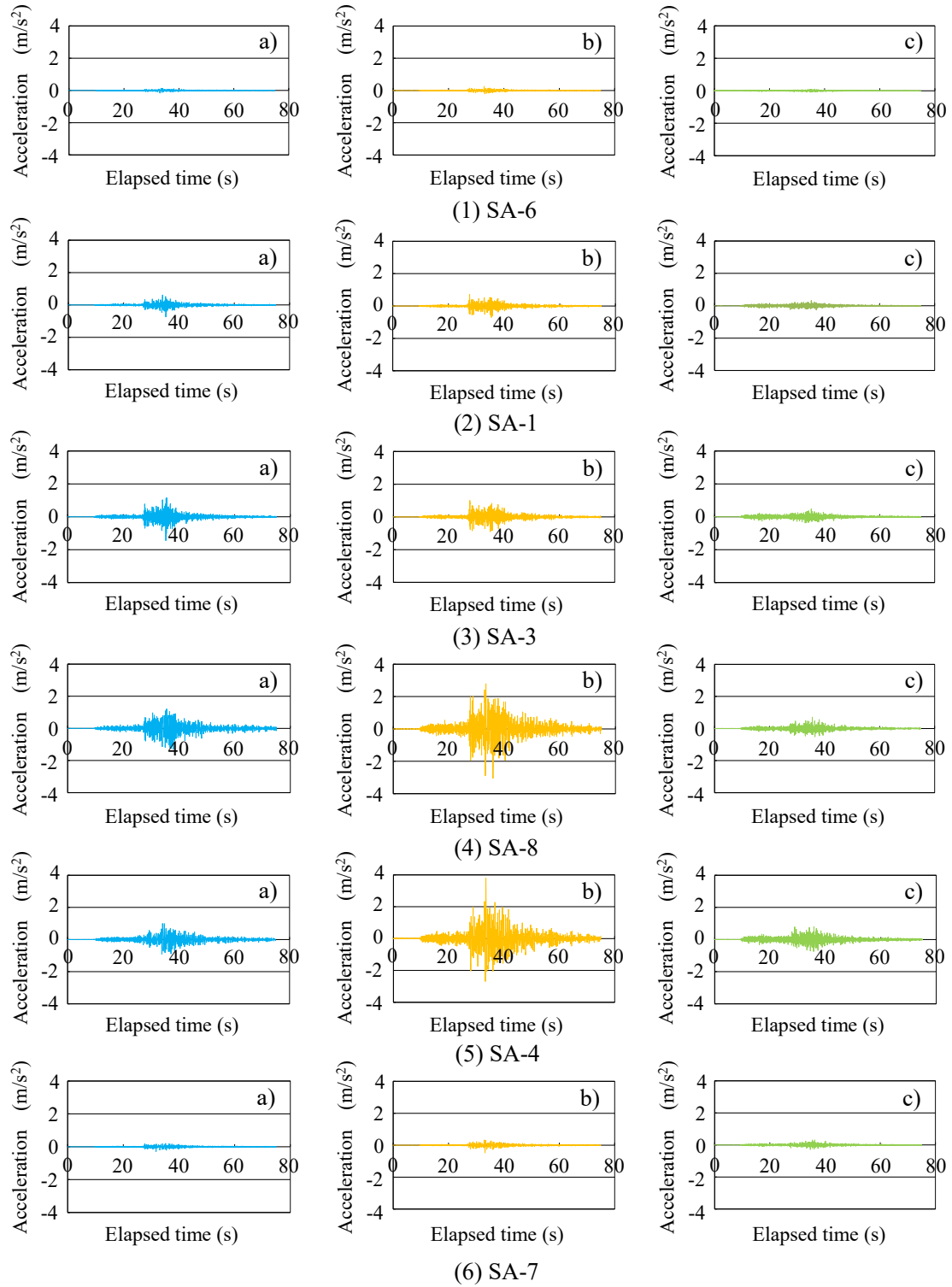


Figure 6.32 Time histories of acceleration response at each point of Dam A on 2003/5/26
a) Stream, b) Dam-axis, c) Vertical

From Table 6.4, it is revealed that the dam axis component takes the maximum acceleration value at SA-4 in all four earthquakes, as at SA-6. However, the stream component at SA-8 takes the maximum acceleration in three earthquakes, but not the fourth earthquake of 2003/5/26. During the earthquake on 2003/5/26, the dominant response direction at SA-4 and SA-8 is the dam-axis direction, as for the input wave (SA-6). The response ratios (normalized acceleration value at each point by the acceleration value at SA-6) for this earthquake are quite large (SA-4/SA-6: 14.6, SA-8/SA-6: 11.7) compared with those of the other earthquakes. Generally, the acceleration response ratio becomes large when the predominant frequency of the earthquake (input wave) coincides with the eigen frequency of the dam body. Therefore, it is assumed that the predominant frequency of the earthquake occurring on 2003/5/26 coincides with the eigen frequency of Dam A, and the response ratios on the crest become large.

Figure 6.33 shows the Fourier spectrum at SA-6 for all the earthquakes. From Table 6.4, the maximum acceleration in the dam-axis is 0.96 m/s^2 at SA-6 during the earthquake occurring on 1987/4/7. Figure 6.33 (2) b) shows that the predominant frequency of the dam axis direction in this earthquake is higher than 5 Hz. On the other hand, the predominant frequency in the dam-axis lies in the ranges of 1.5 to 2.0 Hz for the earthquake occurring on 2003/5/26, as shown in Figure 6.33 (4) b). This frequency range is near the eigen frequency of 2.6 Hz in the dam-axis of Dam A, which is estimated by the experimental formula for rock-fill dams (Okamoto, 1971). Thus, it is revealed that the earthquake occurring on 2003/5/26 has its predominant frequency near the eigen frequency of Dam A, but the predominant frequencies of the other earthquakes are lower or higher than that.

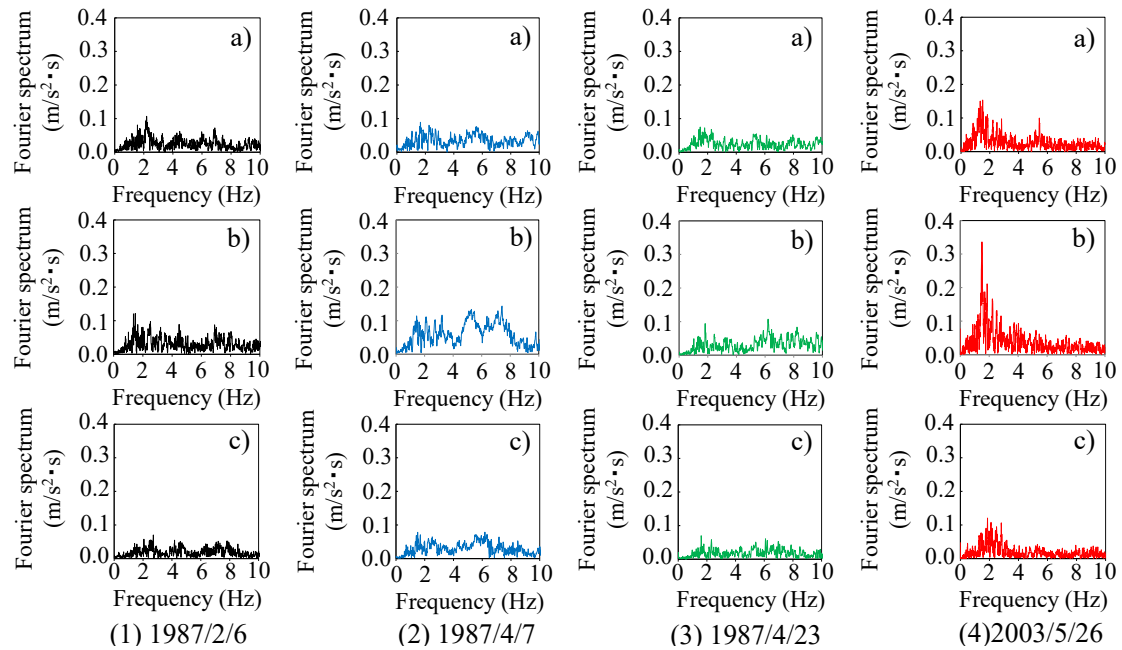


Figure 6.33 Fourier spectrum at dam-axis direction of SA-6; a) Stream, b) Dam-axis, c) Vertical

Figures 6.34 and 6.35 show the acceleration loci in the stream and dam-axis planes at SA-4 and SA-8, respectively. SA-4 is located at the right above the deepest part of the valley and at the center of the crest. SA-8 is located at the middle point between the left abutment and the center of the crest. From Figure 6.34, it is revealed that the acceleration loci at SA-4 tend to show excellence in the dam-axis, although the responses in the stream are comparably large for the earthquakes occurring on 1987/2/6 and 1987/4/23. On the other hand, from Figure 6.35, it is seen that the acceleration loci at SA-8, except for those of the earthquake occurring on 2003/5/26, tend to show excellence in the stream, although the responses in the dam-axis direction are comparably large.

These tendencies are compared with the results of Model C, shown in Figure 6.19 (3), when the shaking direction is the dam-axis and the frequency of the input wave is swept (Case 2). Here, SA-4 and SA-8 correspond to C1 and C2, respectively. From Figure 6.19 (3), the dam-axis component at C1 exceeds the stream component in the range in frequency of 6 to 9 Hz, including its predominant frequency in the dam-axis, namely, 7.8 Hz (Table. 6.3). The dam-axis component is superior to the stream component in the range in frequency of 6 to 9 Hz at C2, including its predominant frequency in the dam-axis, namely, 7.0 Hz (Table. 6.3). However, above 7.4 Hz, which is the predominant frequency in the stream (Table. 6.3), the dam-axis component becomes equivalent to the stream component.

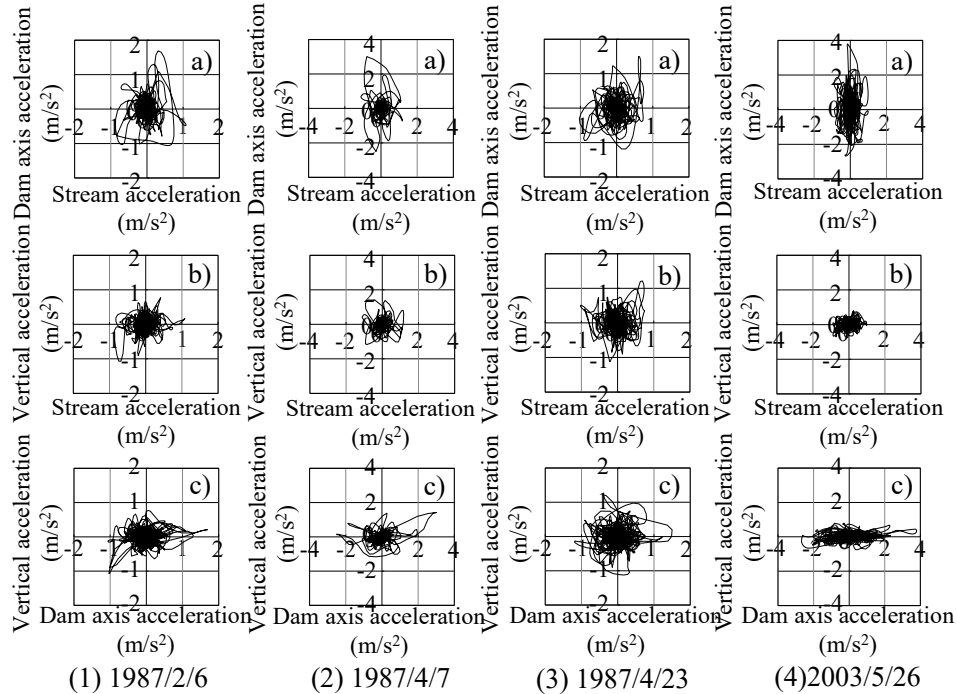


Figure 6.34 Loci of measured acceleration at SA-4 of Dam A
a) Stream, b) Dam-axis, c) Vertical

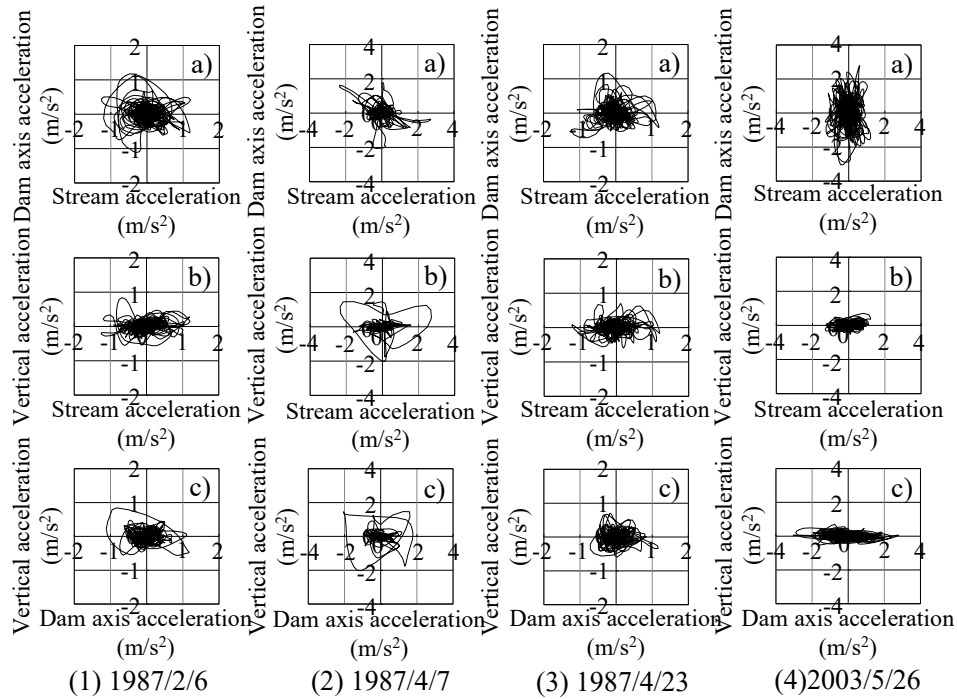


Figure 6.35 Loci of measured acceleration at SA-8 of Dam A

a) Stream, b) Dam-axis, c) Vertical

In Dam A, it is supposed that the dam-axis component is superior to the stream component at SA-4 and SA-8 around the assumed eigen frequency in the dam-axis, namely 2.6 Hz, but that the stream component becomes equal or superior to the dam-axis component at SA-8 in the lower or higher frequency range. Thus, it is concluded that the seismic behavior of real dams shows a similar tendency in shaking tests. The test models are made of an elastic material. However, real fill dams are made of materials which show nonlinearity with strain-dependence. It is considered that both sets of results show similar tendencies, because the strength of the earthquakes are comparably weak, below 1.0 m/s^2 , and the materials of real dams maintain their linearity without experiencing a decrease in stiffness during earthquakes.

Thus, seismic motion on the crest of real dams shows different tendencies at each point and complicated three-dimensional behavior. This is particularly true when the direction of an earthquake is dominant to the dam-axis and its predominant frequency is above or below the eigen frequency of the dam.

Chapter 7 Conclusions

In this thesis, the mechanisms of the possible failures of fill dams induced by earthquakes have been revealed, and a countermeasure to moderate these harmful effects on the dams has been proposed. A new measuring system to evaluate the health of fill dams has been developed and improved, and its performance has been verified for some real dams over the long term. Reviewing the foregoing chapters, the results are summarized once again as follows.

In Chapter 2, the seismic damage to dams during past massive earthquakes was reviewed and the features of the damage were summarized. Moreover, reviews of both the seismic design of fill dams and studies on the possible failures of fill dams due to earthquakes were given. In summary, it is obvious that the seismic performance of rock-fill dams is excellent, and that concrete dams are superior to embankment dams in seismic performance, while earth-fill dams have suffered more serious damage during earthquakes than other types of dams. Most of the damage was due to settlement at the crest, longitudinal and transverse cracking on the crest, transverse cracking, and a difference at the connecting part between the abutment and the embankment, and sliding, cracking, and a difference parallel to the dam-axis at the downstream or upstream slope. Their effects were mainly concentrated in the upper part and the crest of the embankment, and such portions were found to be the most vulnerable to earthquakes. Earth-fill dams composed of low-density sandy or silty materials are vulnerable to massive earthquakes; and thus, it is essential to assess the harmful effects of such earthquakes on dams and to take measures to reduce them.

In Chapter 3, the performance of the new measuring system installed within embankment dams was verified. This system can improve the efficiency of dam construction and the stability of pore water pressure measurements for the safety evaluation of fill dams. This system is called the “wireless pore water pressure transducer (WPT)”. From the results of tests conducted at real dams, it was confirmed that WPTs can measure data as well as conventional sensors with cables. The validity of WPTs is obvious from the ease of their installation and their measurement performance. However, the test models for the WPTs installed in real dams became inoperative and unable to perform data communication after only five and a half years. This was a much shorter lifetime than the expected and specified one of ten years. It was clarified by capacity tests on their batteries and a look at the voltage data of the batteries measured on site that the reason for this phenomenon was not due to a lack of battery capacity, but to a drop in voltage during the communication mode induced by the increase in the internal resistance of the batteries. Thus, the batteries were set in parallel to moderate the effect of the drop in voltage. Improved ones were installed in a real dam. The results of the improved WPTs showed that the effect of the drop in voltage induced by internal resistance can be modified compared with the test models, and that the long-term performance of WPTs can be advanced.

In Chapter 4, the influence of the regions of the modeled area and the material properties of the bedrock on the seismic behavior of the dam were investigated by numerical experiments. From the results, it was clarified that the acceleration amplitude ratio, the Fourier spectrum ratio, and the 1st predominant frequency of the dam mainly depend on the material properties of the dam's bedrock. Moreover, the modeled area of the bedrock was found to have a lesser effect on them. These phenomena can be caused by the deformation modes of the dam based on the interaction between the dam and its bedrock. The deformation modes of the dam mainly depend on the material properties of the bedrock, and the regions of the modeled area have a lesser effect on the deformation modes of the dam. The seismic characteristics of the bedrock are effected by the predominant frequency of the dam body. Thus, it is important to determine the modeled area of the dam's bedrock in seismic response analyses such that the predominant frequencies of the dam do not approach those of its bedrock.

In Chapter 5, the fracturing mechanism of the dam body developed by the interaction among the liquefiable foundation, the dam body, and the reserved water were examined by centrifugal liquefaction experiments in view of the liquefaction of the foundation which is one of the principal factors inducing large deformations of the dam body during an earthquake. From the results of the experiments, it was clarified that fractures in the dam body develop progressively according to the interaction between the liquefiable foundation and the dam body. It is also supposed that the deformation of the upstream slope of the dam body is the dominant factor in inducing the catastrophic destruction of the dam body. It was clarified that the retrofit at the toe of the downstream slope can moderate such deformation and, even when the dam body has been damaged severally, catastrophic destruction inducing overtopping can be prevented. Deformation behaviors of earth-fill dams due to the liquefaction of foundation grounds were examined by an effective stress analysis method, in which the zoning type, the dynamic properties of the liquefiable foundation ground, the reserved water condition, and the input wave were changed. It was clarified that the zoning type and the reserved water condition will remarkably affect the tendency of the dam to deform. The settlement of a dam with inclined core zoning will be larger than that with other types of zoning under the reserved water condition.

In Chapter 6, the characteristics of the seismic behavior of fill-type dams were examined by shaking table tests, in which three dam models with different shapes were shaken separately in the stream direction and in the dam-axis direction, and then the effects of the dam shape and the direction of the input wave on the seismic response behavior were examined. From the test results, it was concluded that the points at which the maximum acceleration values were recorded during the shaking in the stream and the dam-axis directions are right above the deepest part of the dam's valley. It was clarified that the response orthogonal to the shaking direction could arise depending on the shape of the dam body, and the direction and the dominant frequency of the input wave. In particular,

when the shaking direction is dam-axis, the dam body shows complicated three-dimensional behavior, and the point at which no response can be incited appears on the crest of the dam. Thus, it is important to pay attention to the relation between the seismic modes of a dam body and the dominant frequency of the input wave when the data recorded by seismometers installed in real dams are analyzed and the cause of the damage to the dam is examined.

Therefore, fill dams show very complicated behavior during earthquakes based on the interaction among the foundation, the reserved water, and the dam body. Moreover, such behaviors largely depends on the nature of the earthquake motion. However, it is technically very difficult to accurately predict the earthquake motion at a dam site. To check the seismic performance of a fill dam, the slope stability is mainly taken into account. However, it is also important to determine the possible failures under each condition of the dam and to assess its seismic performance by a comparison with the already well-known “defensive design”. Creating countermeasures for the purpose of protecting existing dams, that do not satisfy the “defensive design”, is important from the viewpoint of dam management, and it is expected that such countermeasures will be developed along with useful evaluation procedures for the safety of dams, if such dams must be utilized in the future.

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