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# GROUND DAMAGE RESULTING FROM TORRENTIAL RAINS IN FUKUI, JULY 2004

THE JAPANESE GEOTECHNICAL SOCIETY: EMERGENCY SURVEY TEAM FOR GROUND DAMAGE RESULTING FROM TORRENTIAL RAINS IN FUKUI, JULY 2004

## ABSTRACT

Torrential rains in Fukui, July 2004 resulted in heavy damage to ground in the Asuwa river basin. This paper reports an outline of the matters and causes of the following damage: 1) The dike in the lower reaches of Asuwa river was breached in city area. A detailed survey was undertaken to determine why the dike was breached at that location. 2) Many cases of damage to revetments were observed in the upper reaches of Asuwa river and its tributary streams, and the damage was worsened by inundation resulting from timber debris and sedimentation. There were many cases of slope failure above revetments, which were thought to be due to scouring. 3) Many slope failures occurred at locations distant from rivers. In many cases they were relatively shallow collapses. 4) Many mud and debris flows occurred along small rivers or in mountain valleys, and large quantities of mud and timber were displaced. 5) At five steel bridges on the JR Line, bridge girders were washed away and bridge supports collapsed or were damaged. Considerable damage was caused to road bridges and to road and rail embankments. Our survey team carried out investigations into these 5 types of damage, to grasp the full extent of the damage and establish issues for future consideration.

**Key words:** bridge, debris flow, dike breach, flood, railway, revetment, road, slope failure, torrential rain (**IGC:** C0/H0)

## INTRODUCTION

Almost all morning from dawn on 18 July 2004, the northern region of Fukui prefecture was subject to intense, sustained rainfall that brought widespread damage to the basin of Asuwa river, a branch of Kuzuryu river. 4 people were killed, one remains missing, and 19 people were injured. 57 homes were completely destroyed, 138 were partly destroyed and 211 were slightly damaged. Flooding up to or over ground floor level affected 13,635 homes, and 183 non-residential buildings were either totally or partly destroyed (Fukui Prefecture, 17 January 2005). This survey team was set up to investigate the damage, and its findings were compiled in a report (Ref. 1). This paper has rearranged and concluded that report. Figure 1 shows the basin of Asuwa river, which covers an area of 416 km$^2$, and the river is a 1st grade river which is 61.7 km in length. Figure 2 shows the hourly rainfall and accumulated rainfall according to the weather observatory. Figure 3 shows the contour of total rainfall. This information testifies to the intensity of the rainfall in the Asuwa river basin shown in Fig. 1. A summary of damage from the standpoint of geotechnical engineering is as follows: 1) Damage to Asuwa river dikes: The dike on the left bank was breached in the Kasuga section (upstream of Kida bridge) in Fukui city. The breach occurred about 90 minutes after the onset of overflow.

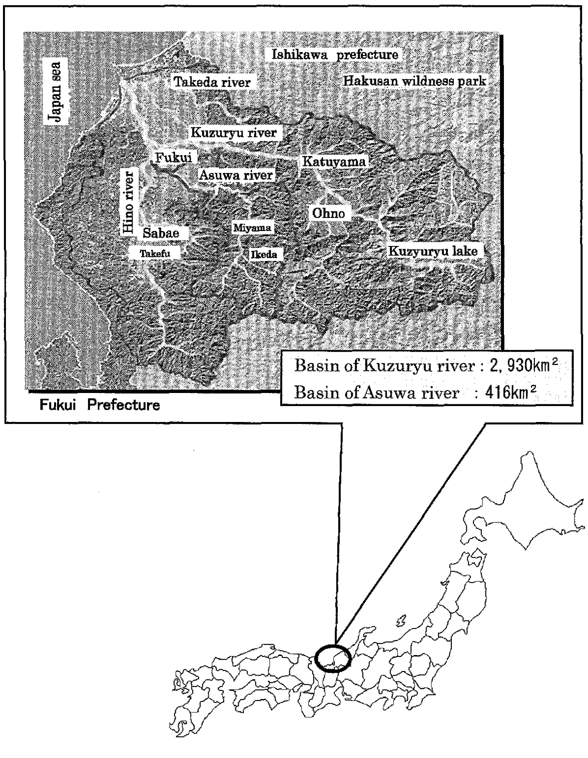


**Fig. 1. Basin of Asuwa river**

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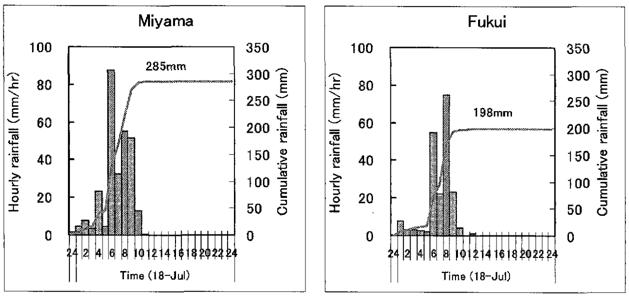



**Fig. 2. Hourly rainfall and cumulative rainfall (meteorological observatory)**

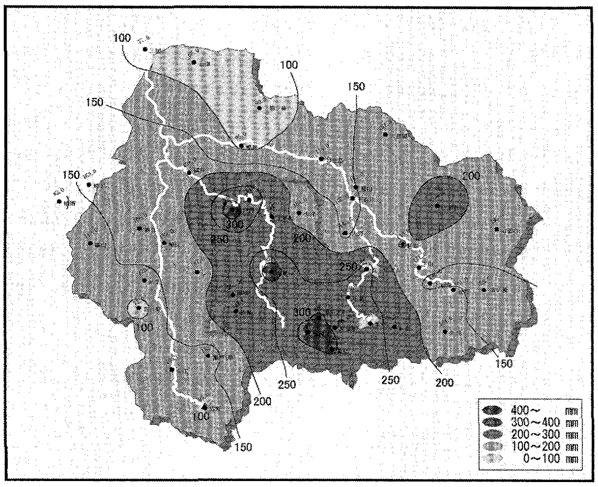



**Fig. 3. Contour of total rainfall (Committee of Kuzuryu river basin, Ref. 3, 2004)**

Dike sections that were not breached, showed many traces of overflow. A detailed survey was undertaken to determine why the dike was breached at that location. 2) Damage in upper reaches: Many cases of damage to revetments were observed in the upper reaches of Asuwa river and its tributary streams, and the damage was worsened by inundation resulting from timber debris and sedimentation. In many places deposited sediments completely covered the original river channel. There were many cases of slope failure above revetments, thought to be due to scouring. 3) Slope failure along roads: Many slope failures occurred at locations distant from rivers. In many cases they were relatively shallow collapses. 4) Damage from debris flows: Many mud and debris flows occurred along small rivers or in mountain valleys, and large quantities of mud and timber were displaced. 5) Damage to road and rail beds: At five steel bridges on the Japan Railways (JR) Etsumi North Line, bridge girders were washed away and bridge supports collapsed or were damaged. Considerable damage was caused to road bridges and to road and rail embankments. The survey team carried out investigations largely into these five types of damage, to grasp the full extent of the damage and establish issues for future consideration. The survey team was made up of the following members (alphabetical order): Katsuhiko Arai and Keisuke Kojima (University of Fukui), Junichi Azuma (Tanaka-chishitsu Consultant Co., Ltd.), Tadashi Hosoda and Fusao Oka (Kyoto University), Yoshitaka Ikeda, Isao Nakajima and Yasuto Takeshima (Oyo Corporation), Takeshi Kodaka (Meijo University), Tatsuo Konegawa (SC Co., Ltd.), KyuTae Lee (CTI Engineering Co., Ltd.), Tamotsu Matsui and Masayuki Sawazaki (Fukui University of Technology), Kazunori Morikawa and Takashi Okajima (Sanwacon Co., Ltd.), Makoto Nishigaki (Okayama University), Kenichi Sugimoto (Teikoku Consultant Co., Ltd.), Susumu Sunami (Nikken-sekkei Co., Ltd.), Tsutomu Terasaki and Takashi Matsushita (Natural Consultant Co., Ltd.), Seiji Uozumi (Dia Consultant Co., Ltd.), Toshiaki Yoden (Newjec Co., Ltd.), Yoshihiro Yokota and Naoki Tatta (Maeda-kosen Co., Ltd.), and Masaho Yoshida (Fukui National College of Technology).

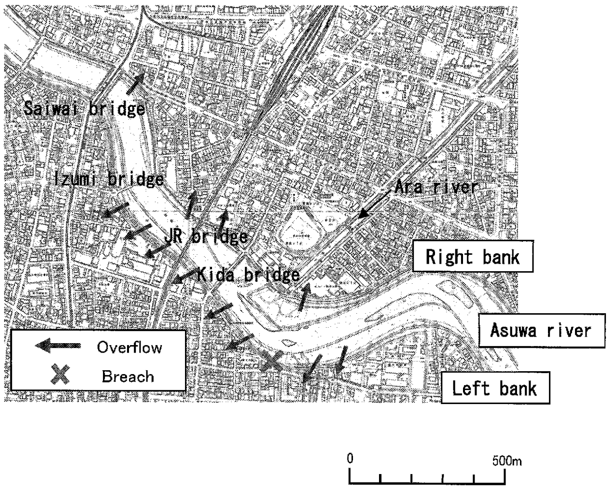



**Fig. 4. Places of overflow and breach on dike (lower reach of Asuwa river)**

## DAMAGE TO ASUWA RIVER DIKES

### *Estimation of Water Levels*

Figure 4 shows the locations of overflow and dike breach in the lower reaches of Asuwa river (inner Fukui city area). On the left bank, overflow occurred over approximately 900 m, from Izumi bridge to upstream of the dike breach. On the right bank, overflow occurred in 4 locations. In the following descriptions, the locations show the distances upstream of the confluence of Asuwa and Hino rivers. The Asuwa river dike was breached on the left bank about 350 m upstream from Kida bridge, at around 4.6 km and the breach was about 54 m wide (*see* Figs. 4 and 5). Approximately 260 ha was flooded by overflow and dike breach. Fukui Prefecture Study Group for Survey and Countermeasures of Asuwa River Flood Damage from Torrential Rains in Fukui in July 2004 (chaired by Professor Nakagawa of Kyoto University) reported the river water level as follows (Ref. 2). Through one-dimensional non-stationary analysis this study group simulated the overflow of the dike and the change of flow quantity in the river channel. Based on the results, they

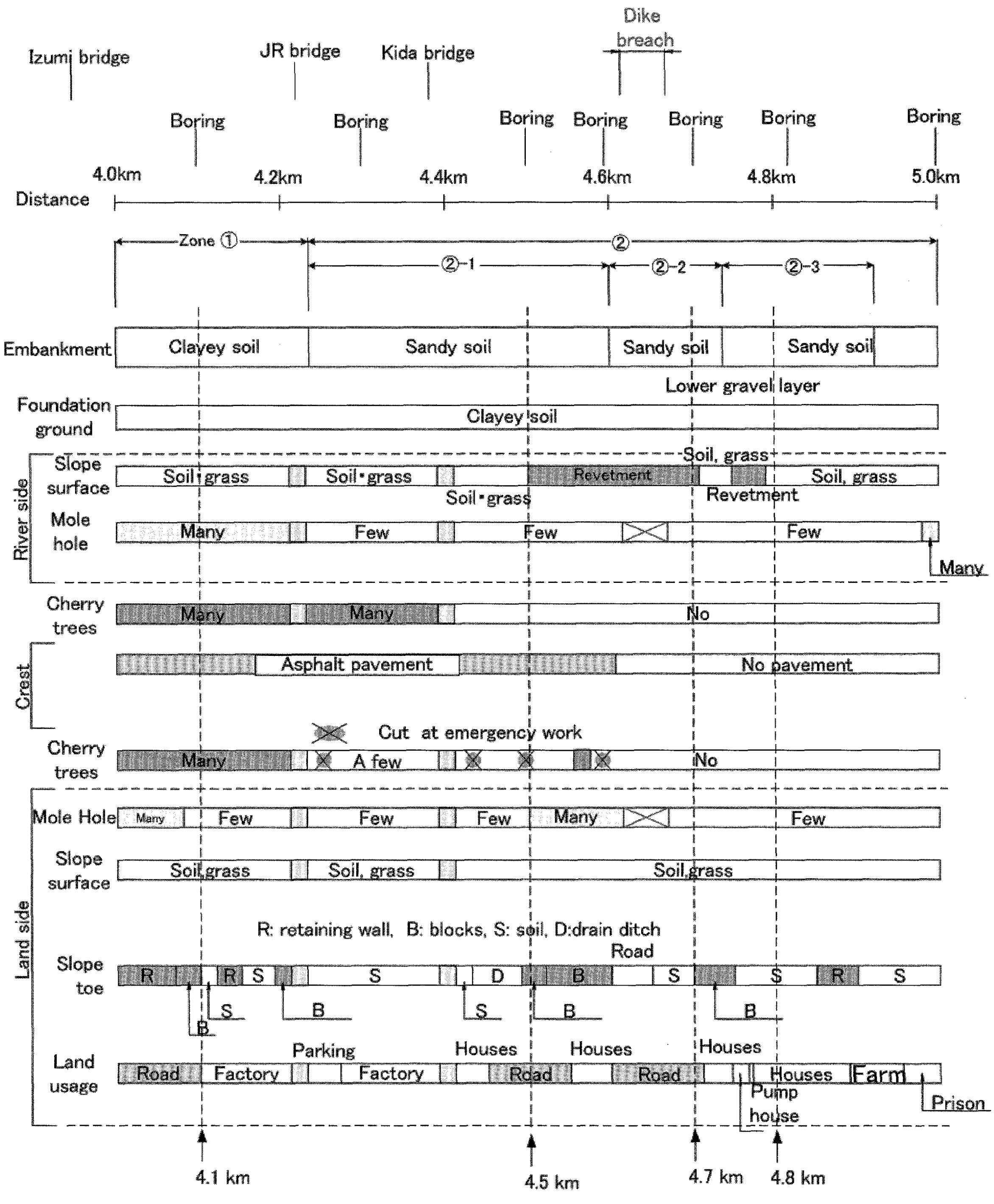



**Fig. 5. Properties of dike (left bank of Asuwa river, 4–5 km)**

reconstructed water level indications in the river channel considering the influence of bridge girders, pump outflow volumes etc. Furthermore, they estimated the time changes of plane flow condition of the river channel at the time of the highest water level and after the dike breach (water level, flow direction, velocity), using plane two-dimensional non-stationary analysis which considers those as boundary conditions, overflow quantities at various locations and after the dike breach taken from the results of the one-dimensional analysis. The two-dimensional analysis infers the following results: 1) Water levels rose rapidly in the vicinity of Kida bridge, and the difference of water level between upstream and downstream of the bridge was around 0.3 m. In the vicinity of the dike breach, the water level on the left bank was about 0.3 m higher than on the right bank. 2) At the 5 km point, the main flow veered towards the right bank, and velocity increased to reach about 3 m/s due to the effect of the curve. At a point on the left bank where the dike was breached, there was no water flow impact zone but the water flow was stationary at the time of peak flooding. This is because the main flow reached the right bank side due to the effect of the plane forms of the dike and low-water channel in the 4.4 ~ 5.4 km vicini-

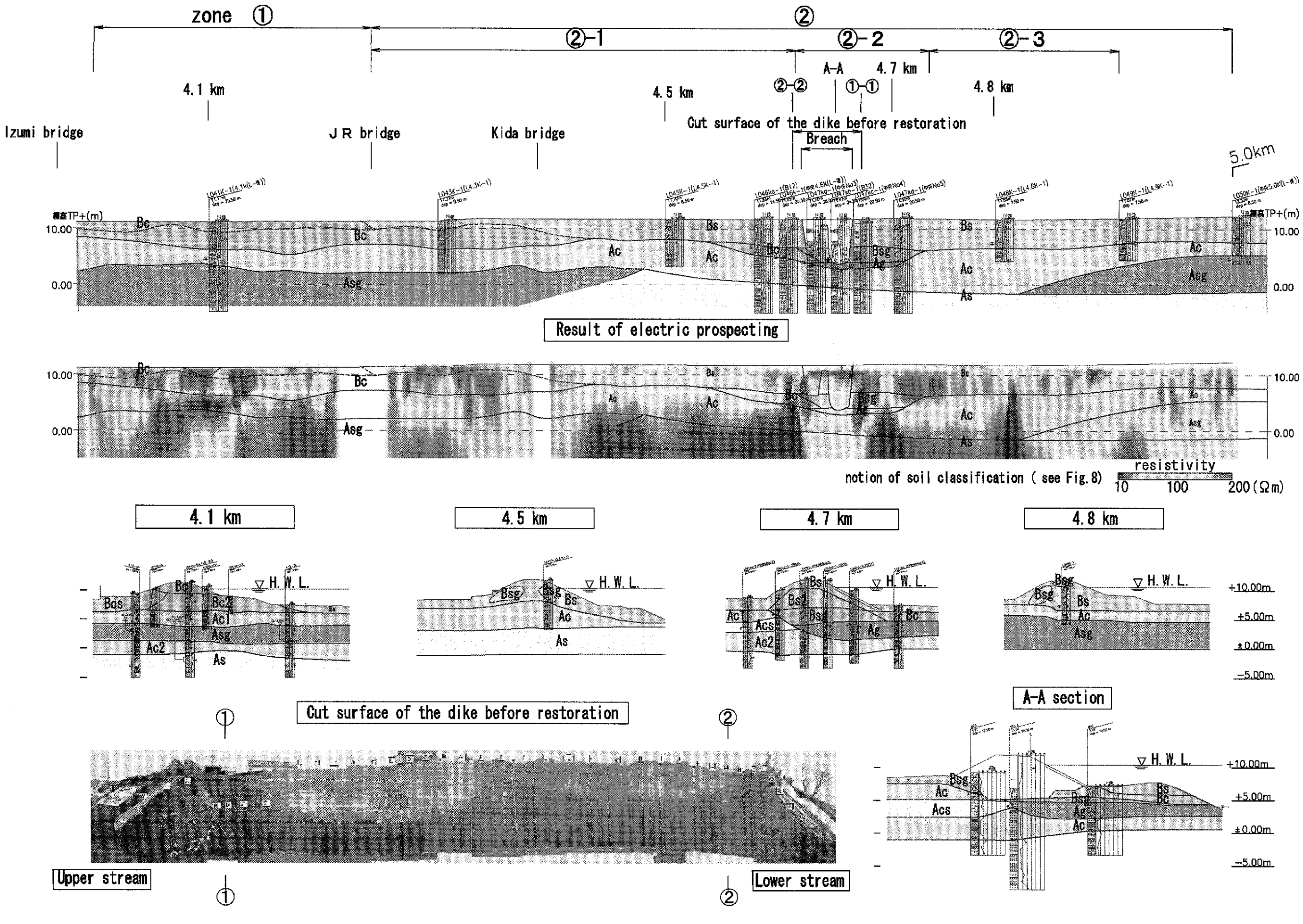



**Fig. 6. Results of geotechnical investigation (left bank of Asuwa river, 4–5 km)**

ty. A flow velocity of about 3 m/s occurred in the high water channel on the right bank at 4.8 km. 3) In the $5\sim5.4$ km section, where the curve of river becomes greatest, the flow velocity on the outside bank side reached only about $1.5\sim2.5$ m/s, resulting in no water flow impact zone also in this section.

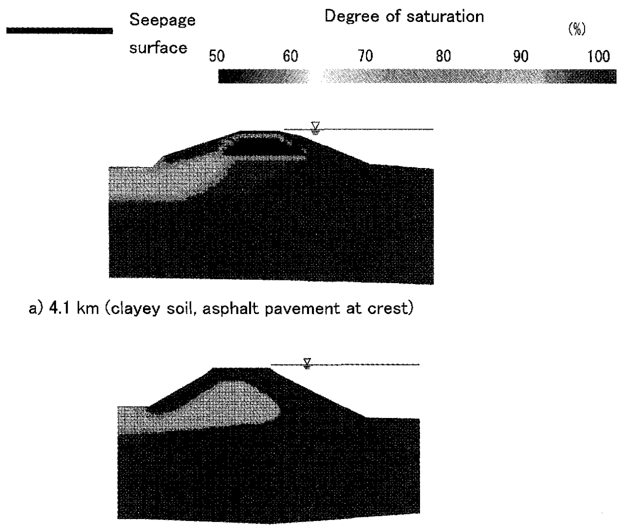
### *Dike and Ground Condition*

To clarify the factors behind the dike breach, we require a detailed and large-scale investigation survey for the strength and permeability of dikes, etc. It would be difficult for individuals to undertake such a survey because it requires considerable manpower and funding. Many members of our survey team participated in the Fukui prefecture study group mentioned above, and performed the investigation. Based on the results of that study group's survey, the factors underlying the dike breach are summarized as follows. The area surveyed was the left bank section where overflow occurred, including the dike breach (around $4\sim5$ km). The study group focused the factors affecting breaches of river dikes, such as erosion and scouring due to overflow, rainfall and flooding (water levels in the river channel), and seepage in the dike due to overflow. Erosion due to water flow in the river channel was unlikely to have caused the collapse, from the fact that we observe little damage due to erosion on the surface of river side dike. The hydraulic investigation described above gives the result that the flow velocity in the river was under 2 m/s. This result means low possibility of erosion on the river side dike. Figure 5 shows the results of site survey into the structure and condition of dike. The zones in the figure indicate the different ground characteristics of the dike itself, from the results of ground investigation. Based on the ground investigation, we estimate the longitudinal and cross sections of the ground strata as shown in Fig. 6, which includes a photograph of cut surface of the dike before restoration. The main features of the survey results are summarized as follows: 1) Dike structure: The crest of the dike is paved with asphalt up to the 4.6 km point which is downstream of the breached section, but is unpaved with a surface of soil and grass above that point. The riverside revetment above and below the 4.6 km point is concrete-pitched grating crib work. The landside toe of the slope varies from place to place, between retaining wall, masonry wall, and embankment slope. In the vicinity of the dike breach there are sloping roads on the landside of the dike. 2) Dike condition: A visual survey indicated mole holes in both the riverside and landside slopes. Downstream from the 4.4 km point, cherry trees have been planted along the dike crest. 3) Dike maintenance: Topographical features that require warning against seepage failure are a) former river channels, b) collapsed ditches and c) former collapsed ditches. A published flood control topographical chart shows that the ground topography and geology of the overflow section is largely classified into delta and that the section is not an at-risk topography in terms of a simple evaluation. From the 4.1 km point to the vicinity where the dike was breached, the Fukui prefecture study group undertook multiple borings transversely across the dike, to estimate the soil layer in this section. A number of borings were also taken around the crest of the dike breached. The study group carried out a geophysical exploration consisting of electrical prospecting and elastic wave exploration. Using these data the study group estimated ground profile in the whole section, which is shown in Fig. 6. 4) Dike soil: In the 4 to 4.2 km vicinity the dike consists mainly of clayey soil; in the 4.4 to 5 km vicinity it consists of sandy soil. The foundation ground directly under the dike is largely alluvial clay layers (Ac1), but around 5 km there are some layers of alluvial sand and gravel (Ag1). In the area of dike breach on the river side there are gravel layers which are thought to be an earlier river bed. 5) Sections studied for seepage failure: Using the data described above, a study into seepage failure was undertaken on the following 4 sections, which were classified by considering the factors behind the dike breach as given below. In Zone ① the dike ground is largely clayey soil, and in Zone ② it is largely sandy soil. Zone ② is subdivided into Zone ②-2 where the layer of sand and stones at the base of the embankment is prominent. Zone ②-1 and ②-3 upstream and downstream of Zone ②-2. The following representative sections were selected from each of the zones, considering whether or not the crest was paved and there were revetments, and whether or not the toe of landside slope was paved. The study models for 4.1 and 4.7 km sections were set mainly by using the results of borings and cut and cover study shown in Fig. 6. The models for 4.5 and 4.8 km were set by the results of geophysical prospecting, as borings were taken only for the vicinity of the crest. 1) 4.1 km vicinity: dike consisting of clayey soil, crest paved, no revetment, toe of landside slope paved. 2) 4.5 km vicinity: dike consisting of sandy soil, crest paved, no revetment, toe of landside slope paved. 3) 4.7 km vicinity: dike consisting of sandy soil, crest unpaved, revetment, toe of landside slope paved. 4) 4.8 km vicinity: dike consisting of sandy soil, crest unpaved, no revetment, toe of landside slope unpaved.

*Factors behind Dike Breach*

1) Factors behind dike breach: Erosion from water flow in the river channel is unlikely to have been a factor in the breach of the dike, because no change is observed in the riverside slope or flood channel. From the surface erosion of the landside slope that occurred in the overflow sections, it is inferred that soil particles of the dike shifted due to the tractional force of overflow, and this overflow erosion is seen as an important factor in the dike breach. A survey of water level indications indicates that the overflow depth was 32 cm and greatest in the vicinity of the dike breach. The hydraulics analysis described above estimates that the overflow continued for about 90 minutes at the point of dike breach. Although there were mole holes, which may be cited as a factor leading to dike failures, enquiries during the site survey could not confirm incidences of water leaking through these holes or of surrounding erosion, thus it is unlikely that they were a factor in causing the dike breach. In the dike upstream from the JR Hokuriku Line, there were locations where the landside slope shoulder had collapsed through vertically gouging. The form of this damage suggests that overflow erosion occurred together with its associated sliding failures and collapses. This may have been a factor in the dike breach. The dike upstream of the JR Hokuriku Line consists mainly of sandy soil, which is considered much more prone to this sort of damage than clayey soil, because water tends to percolate downwards through dike more quickly in sandy soil. Downstream of the dike breach, the crest of the dike is paved with asphalt, and the unpaved sections are likely to suffer greater damage than paved sections due to seepage of overflow water. We have the detailed discussion concerning the effect of these factors in the following paragraphs. 2) Damage due to overflow scouring: According to 'River Dike Design Guidelines (3rd edition) June 2000' and 'Handbook for Design of Dike Reinforcement against Overflow (draft) October 1998', a study was conducted on the possibility of failure due to overflow scouring. This study was subjected to 4, 4.2, 4.6, and 4.8 km points where the overflow depth was identified by water level indications. The study took the overflow depth as an external force condition, and compared the generated shear force with the shear strength defined by the dike surface material (asphalt, grass, bare ground) and dike slope gradient. This check was made at the crest, landside slope, and the toe of landside slope. For the toe of landside slope, the impact force due to overflow was not taken into account. The results are summarized as follows. At the crest of dike, the allowable shear force was satisfied and overflow scouring did not occur irrespective of whether or not it was paved. However, in the upstream section the shear force approaches the allowable value when the degree of coverage is low. On the landside slope, because the shear force greatly exceeded the allowable level irrespective of overflow depth, it is likely that overflow scouring occurred, except some locations where the degree of coverage is high. At the toe of landside slope, the shear force greatly exceeded the allowable value in the unpaved sections where the overflow depth exceeded 30 cm, and the force very slightly exceeded the allowable value in the asphalt-paved sections. Thus overflow scouring is seen as likely to have occurred on the landside slope and its toe. 3) Seepage failure: a) Seepage surface in dike was estimated by seepage analysis, in order to discuss the slope stability and piping damage due to seepage. Unsteady two-dimensional saturated and unsaturated

seepage analysis was performed over the time in rainfall and flooding. Figure 7 shows a part of the results of analysis when the overflow occurs. In the analysis, overflow scouring is not taken into consideration. The dike at the 4.1 km point consists mainly of clayey soil, only very little seepage surface appears on the landside slope as shown in Fig. 7(a). In other sections including Fig. 7(b), a seepage surface was formed on the landside slope despite dike structure. Dike strength on the landside slope may be reduced through the increase in saturated region due to the formation of this seepage surface. The dike at the 4.7 km point consists largely of sandy soil, and the dike crest is not asphalt-paved. This means that the saturated area in the vicinity of the dike crest tends to expand due to rainfall and overflow. This tendency furthers the seepage from the river channel. In this case, the saturated region reaches the toe of landside slope as shown in Fig. 7(b). This saturated region on the landside slope does not rise from the slope toe, but is formed from the surface layer, since the unsaturated regions are left from the slope toe to the dike center. The saturated region may make the soil strength decrease on the landside, which adjusts a lot of localized collapses observed at various locations. b) Stability analysis by circular slip surface method was performed in order to evaluate the stability of landside slope. The site survey confirmed dike failures seen as resulting from erosion and localized slip at several locations upstream of Kida bridge shown in Fig. 4. The seepage surface obtained from seepage analysis was used as the water level within dike for circular slip surface calculations by Fellenius method. At the riverside slope of dike, the minimum value of safety factor is greater than 1.0 and the likelihood of slip failure is small. At the landside slope of dike, the minimum value of safety factor was lower than 1.0 when the overflow depth was greatest, which is shown in Fig. 8. This is because the seepage surface had extended to the landside slope surface, and because at the 4.7 and 4.8 km points the dike consists largely of sandy soil and the crest is unpaved as stated above. There is a high possibility of slip failure on




a) 4.1 km (clayey soil, asphalt pavement at crest)

b) 4.7 km (sandy soil, no pavement at crest)

**Fig. 7. Results of saturated and unsaturated seepage analysis at the maximum depth of overflow**

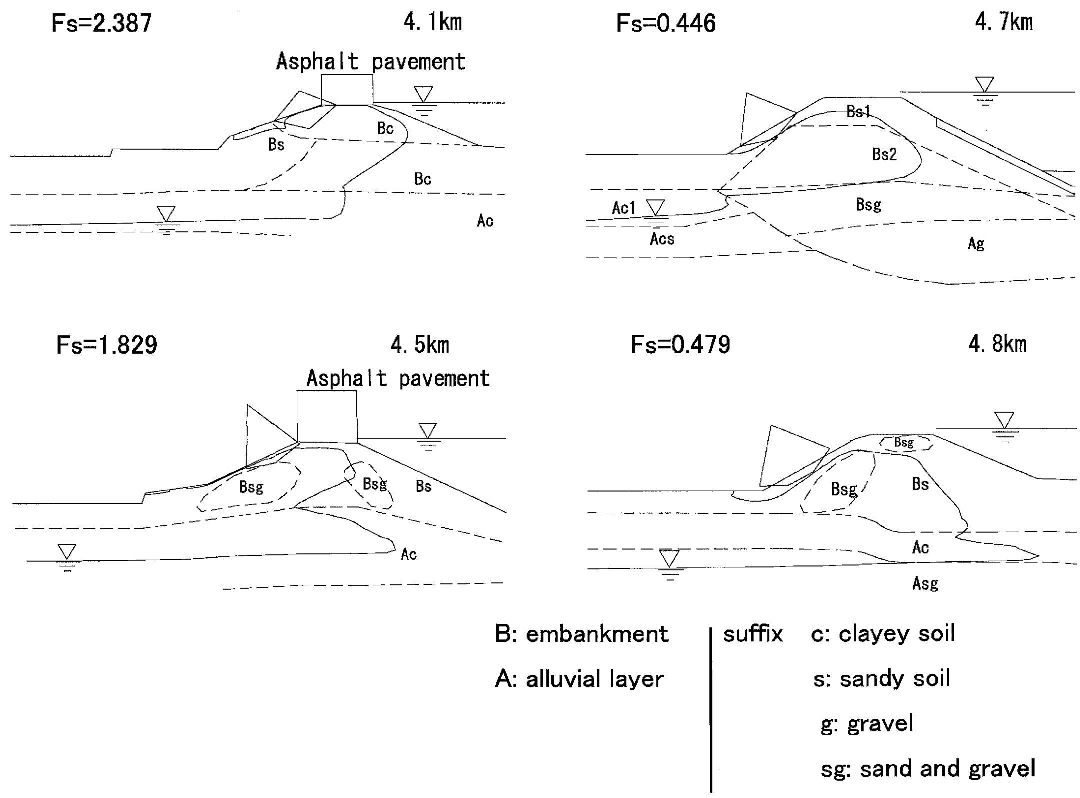



**Fig. 8. Safety factor by slope stability analysis at the maximum depth of overflow**

the landside slope of the dike, which adjusts the results of site survey. c) Piping damage was studied by critical hydraulic gradient calculated. In piping failure, soil particles are washed away by seepage water pressure, and pipe-shaped holes and channels are created within the dike. Since hydraulic gradient values calculated from the seepage analysis do not exceed the critical values in all sections, this suggests low possibility of piping. d) Dike collapses were studied through soil-water coupling analysis, which links the water pressure by seepage analysis and the pore water pressure by stress analysis of ground. This analysis gives a detailed understanding of the effective stress behaviour from the distribution of pore water pressure within the dike. Using the shear stress obtained from the analysis and Mohr-Coulomb failure criterion, a safety factor for each finite element was calculated as shear strength/shear stress. Locations where the safety factor was lower than 1.0 were taken as weakened regions. Also in the analysis, only seepage due to overflow was taken as an external force; overflow scouring was not taken into account. From the results, at the 4.1 km point where the dike consists largely of clayey soil, there was no weakened area that would lead to the collapse, even though there were narrow weakened regions in the surface layer around the dike crest. At the 4.5, 4.7 and 4.8 km points where the dike consists of sandy soil, the seepage area on the landside slope was large, and there was a weakened area with a strong likelihood of shear failure occurring. At the 4.7 km point there were many weakened regions on the landside slope and around the dike crest due to overflow. The actual damage and the analytical results conform well. 4) Summary: Whether or not the dike crest was paved, was a factor greatly affecting the amount of seepage flow into dike. When dikes consist of sandy soil, the seepage surface expands and may possibly lead to dike breach, depending on the length of continuous rainfall and flooding. Though weakened areas resulting from seepage were estimated from the results of survey and analysis, the extent of weakened areas was not so great that they would cause crest failure. It is hard to think that seepage failure alone could lead to dike breach. In the breached section, since the dike crest was unpaved, and since the dike consisted mainly of sandy soil, it can be inferred that seepage from the crest due to rainfall and overflow added to seepage from the riverside slope, extending the seepage area and expanding the weakened region. Considering the seepage area and weakened region, there is a possibility of slip failures and localized collapses occurring on the landside slope although the sliding layer is thin. From these results, it is thought that scouring and seepage resulting from overflow caused a weakening of resistant properties against scouring on the landside slope, and that stronger scouring occurred together with slip failures and localized collapses. The progressive effect of these multiple factors is seen as likely to have led to dike breach.

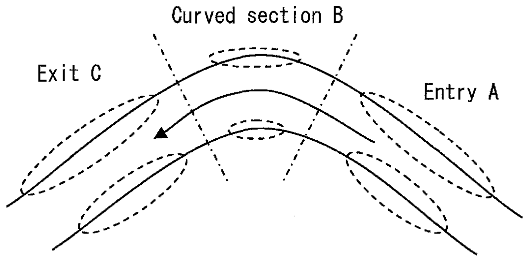



**Fig. 9. Classification of the failure points in the bending channel**

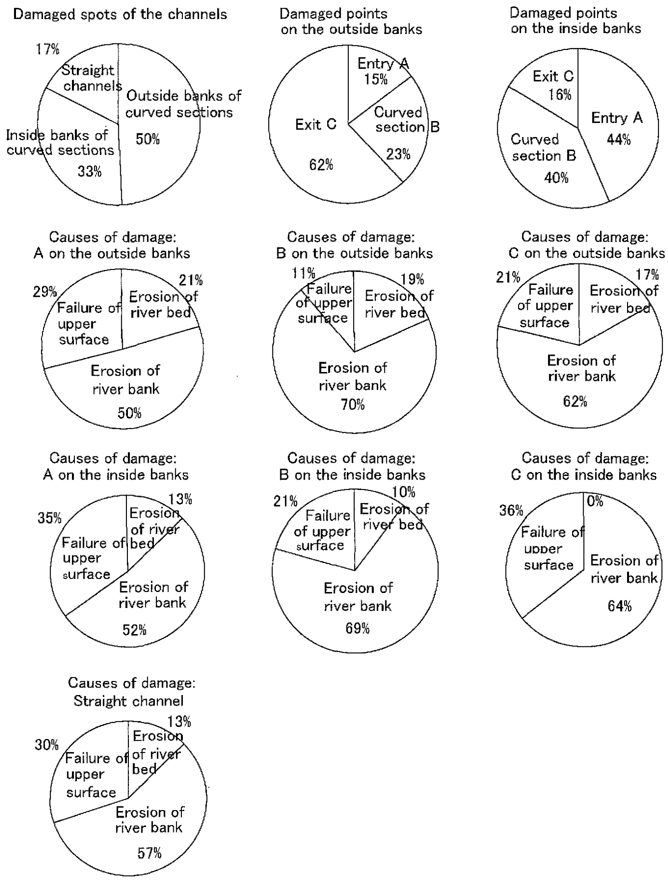



**Fig. 10. State of damaged revetments and dikes**

## DAMAGE TO UPPER REACHES

Herein only damage of river revetments is reported, although our survey team made a detailed study of the relationship between damage of river structures and their locations, strength of rainfall and accumulated rainfall, geology of surface layers, etc. We summarize here the position and shape of revetment damage, as well as its causes of damage. In the upper reaches, almost all river channels are meandering. Generally revetment damage was frequently seen in curved sections. Figure 9 groups the locations of damage on a plane figure. Figure 10 shows the location of revetment damage, the factors causing damage and the proportion of each. We got the base data from Fukui prefecture preliminary report prepared for emergency restoration work (Ref. 3, 2004).

The classification of damage causes made here, is not based on actual observation but the river section charts in the report. When water levels at the flooding time exceeded the revetment crest height in the report, crest failure was considered as the primary cause. However, it is estimated in many cases that river bank erosion occurred once the crest failed. There are also many cases where river bed scouring may have prompted bank erosion. When multiple causes are considered possible, the number of causes is higher than the number of cases of damage. Particularly the proportion of bank erosion becomes high. Also in Fig. 9, we count in duplicate when the location of damage straddles different sections, for instance, from entry A to curved section B. Figure 10 indicates the following features. 50% of damaged revetments and dikes are on outside banks of curved sections, 33% are inside banks and 17% on straight sections, thus the damage in curved sections, on both inside and outside banks, accounts for over 80% of the total. 62% of the damage on outside banks was damage to exit C, while only 15% was damage to entry A. On inside banks, 44% was to entry A and 40% to curved section B, while only 16% of the damage was to exit C. The 6 charts that compile the causes according to location, show that overall, bank erosion accounts for between 50% and 70%. Besides bank erosion, on outside banks the proportion of river bed scouring is high, while on inside banks the proportion of crest failure is high. On entry A, the proportion of crest failure is high for both inside and outside banks. On curved sections B, bank erosion accounted for about 70% for both outside and inside banks, and this was higher than for both entry and exit sections. In many cases on outside banks, river bed scouring led to bank erosion, while on inside banks crest failure led to bank erosion. Crest failure accounts for a relatively high proportion of damage in straight sections.

The rivers damaged in the upstream areas, are located in flat valley floors, and almost all river channels are dug in. River levels therefore rose suddenly due to the torrential rains, and extensive areas were inundated by flood water that could not be carried downstream. In classifying the causes of revetment collapse, the mechanism of crest failure is as follows. The crests were damaged by overflow, and the revetment damage took the form of backfill soil escaping. In almost all cases we think that the crest failures occurred from the multiple effects including bank erosion. When inundations occurred at almost all locations in the river basin, the proportion of crest failures tends to become higher. In areas where rivers make sweeping curves, muddy flows ran outside of river channels. This may make the proportion of crest failures become higher, because particularly for the inside banks the form of the damage was as if the revetment had been peeled off the crest. In many cases, on the outside banks secondary flow has the effect of deep scouring and river bed scouring tends to occur. For every position on outside banks, the proportion of river bed scouring is about 20%, which is high compared with inside banks. A plot of flow velocities in curved sections

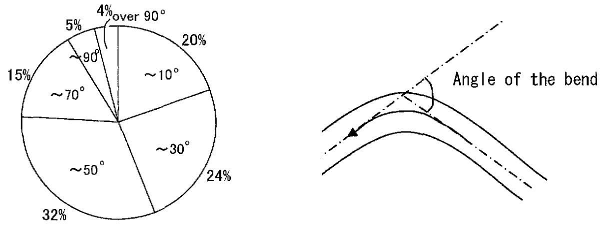



**Fig. 11. Ratio of damaged channels categorized by the angles of the bend**

shows cases where, depending on the degree of curvature, velocity on the outside bank side increases due to the secondary flow effect, and also cases where velocity increases intermittently on the outside bank side at the exit. By contrast on the inside bank side, in many cases the velocity is higher than on the outside bank from the start of the curve to the midpoint, and the velocity gradient also increases with the curve. On the outside bank side, the number of failures in the vicinity of the curve exit accounted for 60%, while on the inside bank side, the number of failures from the curve entry to the midpoint accounted for 80%. The high velocity of the river's inundating flow or the velocity gradient can be seen as having a large effect on the revetment damage. Figure 11 shows the angles of individual curves and the proportions of cases of damage. Curvature angles were taken as the angle at the intersection of two straight lines extended from the straight lines of flow on either side of the curve as illustrated in Fig. 11. Damage locations in straight sections are included with those of curvature angles of 10° or below. Up to 50°, the larger the curvature angle, the greater the proportion of cases of damage. Above 50°, the number of such steep curves declines, so the proportion of cases of damage also declines. A survey should be made of the proportion of damaged sections to non-damaged sections for various curvature angles, but in the present case an extremely large number of small and medium-sized rivers were studied, and it was difficult to go to the site and to measure the curvature angles of undamaged sections. In the case of the torrential rain damage in the Higashi Seto region in July 1974 caused by Typhoon No. 8 and the seasonal rain front, a study team from Kyoto University surveyed in detail the curvature angles and the proportions of damage for all the river channels of Shin river and Houju river in Awaji island (Ref. 7, 1974). This study confirmed that, for Shin river for which the average curvature angle was not so great, the greater the curvature angle, the higher the rate of damage, and the ratio of damage on the outside bank was particularly high. For Houju river which had a higher average curvature angle, no clear correlation could be identified between the curvature angle and the ratio of damage. Nevertheless, the overall ratio of damage for Houju river was evidently much higher than for Shin river, showing that the steeper the curves there are in a river, the greater the ratio of damage. The rivers in the upper reaches affected by this Fukui rains, had many∈ curved sections, which is seen as linked to the increased

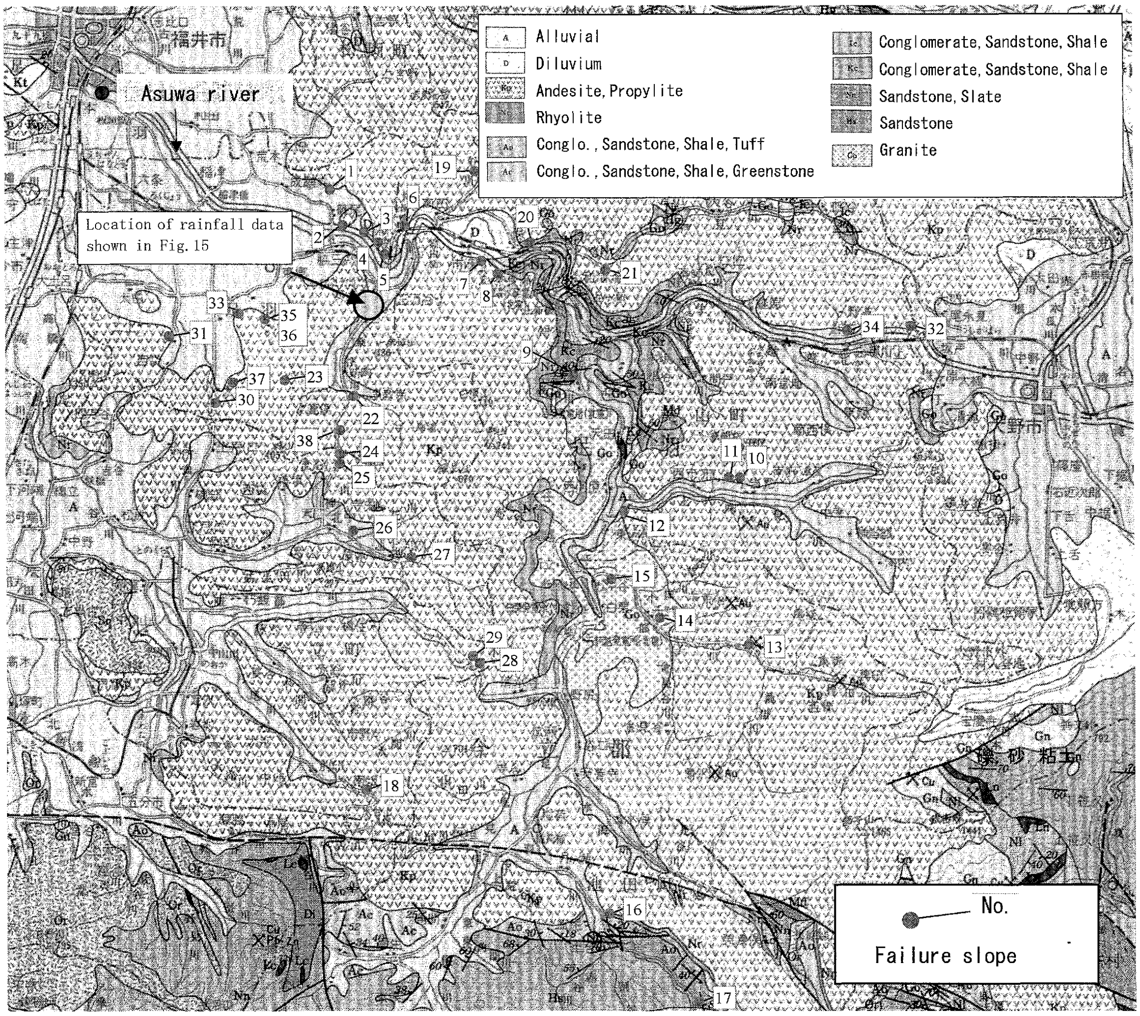



**Fig. 12. Sites of slope failure**

number of cases of damage. As Fujita et al. (1989) have reported for the upper reaches of rivers, when damaged revetments are restored, they are in many cases damaged again in the following flooding event. We should remember case studies on this occasion and in upper reaches with many curved sections it is necessary to be prepared for flood damage whatever type of revetments are in place.

## SLOPE FAILURE ALONG ROADS

### *An Outline of Topographical and Geological Characteristics*

Our survey team examined slope failures that occurred in the basin of Asuwa river shown in Fig. 1, which is located in the Echizen central mountain area. Figure 12 shows locations of slope failure investigated and a geological map of the region. In this mountainous region there is a distribution of Funatsu granite (older granite) and conglomerate, sandstone and mudstone of the Tetori and Asuwa groups of Jurassic-Palaeogene. They are widely covered by types of andesite of Itoo formation of the Neogene. At the boundary between the mountains and plains there is a distribution of terrace deposit of Diluvium, and talus deposit of Diluvium and Alluvium. Most of the failures on this occasion occurred in the Itoo formation area. The Itoo formation belongs to the so-called Green Tuff, and it can easily be weakened because of hydrothermal alteration. In many cases it forms deep weathering crust, and it is not unusual for the depth to exceed 10 m. Also, large numbers of lineaments have been identified in the Echizen central mountain area. In places where the gentle slopes around lineaments have been excavated by steep gradients, failures occur even with ordinary heavy rain.

### *Results of a Survey of Slope Failures*

The total number of slope failures investigated is 33, which is classified into 4 types according to failure patterns as illustrated in Fig. 13. Pattern 1 failure is caused in the rock mass with significantly weathered surface soil, and which mainly consists of easily weathered pyroclastic rock or andesite. Pattern 2 is the failure of scree, which is caused in the talus deposit. In some cases, the failure occurs in comparatively gentle slope or in large-scale slope; some planted big ceder trees completely fall down; the slope is located in catchment topography with some spring points. The whole stratum

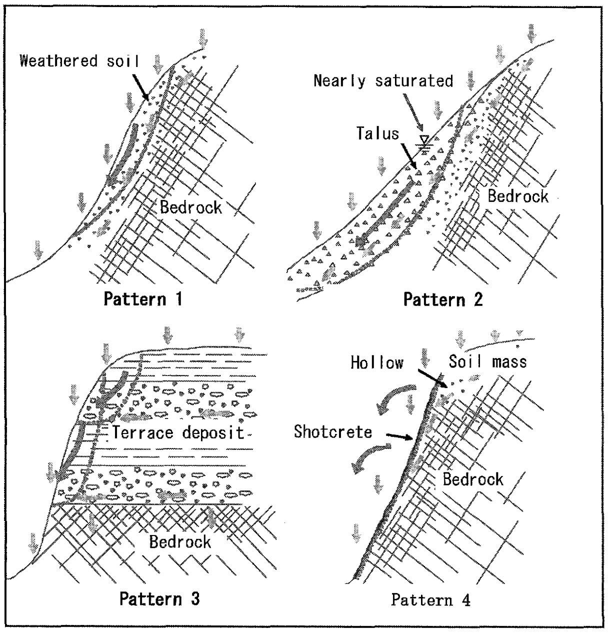



**Fig. 13. Patterns of slope failure**

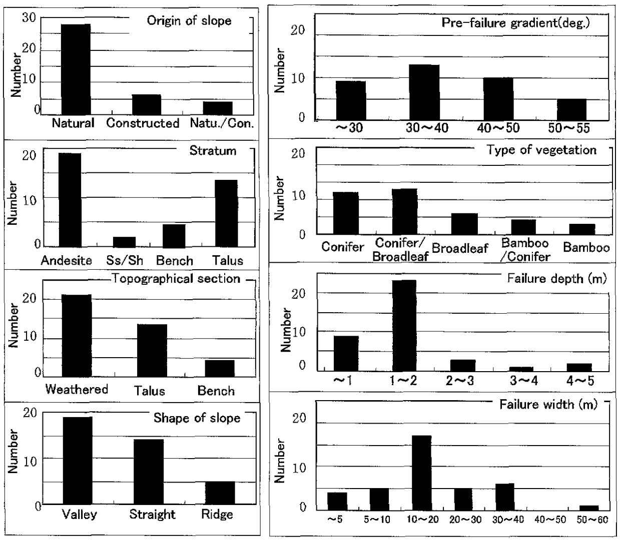



**Fig. 14. Factors affecting slope failure**

is nearly saturated by rising of groundwater and/or by forming of saturated zone from the surface due to rainfall, followed by failure like mud flow in which the moving distance of soil mass is long. Pattern 3 failure is caused in the terrace deposit on bedrock or at around layer boundary within terrace deposit, in which the pervious boundary becomes weak plane. Pattern 4 failure is the peeling off of shotcrete spraying work, which is caused by the water pressure behind separated shotcrete, although weep drains on the shotcrete are available. The bedrock consists of significantly fissured soft rock with soil mass at the top. In some cases, the hollow behind shotcrete has existed formerly.

The actual number of failures of pattern 1 is 15, pattern 2: 13, pattern 3: 3, and pattern 4: 1. Figure 14 shows the frequency distribution charts for 8 items: origin of slope, stratum, topographical section, shape of slope, pre-failure gradient, type of vegetation, failure depth and failure width. Only the whole distribution chart is shown and the chart for each pattern is omitted here. The constructed slope category in Fig. 14 means features such as excavations, embankments, structures, etc. The slope gradients and sizes of failures are outline values. Slope shapes are judged from observations at the site survey. Of the hills in the topographical areas, where there are thick distributions of talus or scree deposits, they are referred to as talus and the remaining hill areas are referred to as weathered soil. As for types of vegetation, cryptomeria, pine and cypress are all called conifer. Locations where there are bamboo and cryptomeria, are shown as bamboo/conifer, and bamboo only is shown as bamboo. A summary of the survey results is as follows: 1) Origins of slopes: Slope failures examined in this survey occurred almost entirely on natural slopes. Cases of damage to constructed slopes are thought to have been few in number because their gradients are stable and suited to structures or the land configuration, and because unstable surface soil is cleared and measures to handle surface water are already in place. 2) Strata and topography: Almost all failures were of weathered soils or taluses. The failures of weathered soils were mostly in the andesite areas. In the region concerned there are also areas of granite, but no cases were identified of surface layer failure of so-called masado. Granite is found in limited areas on both banks of Asuwa river. Simply stating, granite distributes in the banks of Asuwa river which are in an area of low elevation where slopes are steep, while andesite is found in an area of high elevation where slopes are gentler. As a result, weathered granite is constantly being eroded and carried away, and there may be nothing to offer on this occasion. Andesite, normally highly stationary, may be washed away all at once on this occasion. 3) Shapes of slopes: Failures occurred frequently in the order of valley-type slopes, straight line type and ridge-type. These results are seen as reflecting the general and empirical trend that slope failures brought about by torrential rains may occur easily in configurations collecting water. Comparing different failure patterns, weathered soil failures occurred irrespective of whether the slope was a valley- or straight line type, but talus failures occurred most often in valley-type slopes. 4) Pre-failure slope gradients: Weathered soil failures occurred irrespective of slope gradient, whereas talus failures tended to occur on relatively gentle slopes. This may be because there are more gentle deposited slopes of talus rather than weathered soil. 5) Width and depth of failures: Failures of weathered soil largely had widths in the 10 ~ 20 m range and depths of 2 m or less, whereas widths and depths of talus failures ranged more widely, with some relatively deep failures of 3 m or more. Rocky ground has low permeability, but to a depth of 2 m or so below the surface there is considerable void content due to roots, etc., and permeability is relatively good. Thus, surface water percolates easily and torrential rains

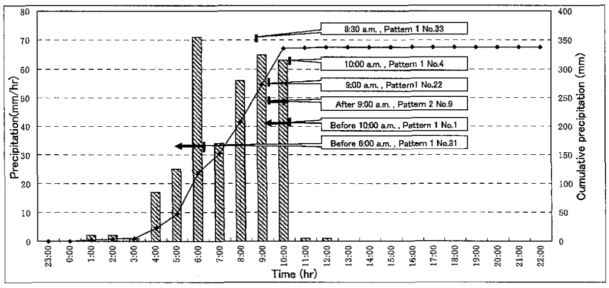



**Fig. 15. Precipitation at failure time (location for monitoring rainfall is shown in Fig. 12)**

produce a saturated zone, and as a result failure depths were concentrated around 2 m or less. By contrast, permeability is good in locations of talus, so the depth of the failed layer corresponds to the thickness of the talus layer and may be greater than with weathered soil. 6) Types of vegetation: Failures of weathered soil occurred regardless of vegetation type, whereas talus failures occurred where there was cover of coniferous trees and also mixed woodland of coniferous and broadleaved trees. However, overall, no clear cause and effect could be identified with vegetation type.

A determination of failure time was made from among all the locations of slope failure above. A majority of the locations were not close to houses, and so six locations were selected for inquiries to determine the time when failures occurred. Figure 15 shows rainfall data with the times of failures occurred. The rainfall data are for Kidonouchi, Fukui city, which is close to most of these six locations. In the case of the slope failures resulting from the torrential rains, the effect of previous rainfall is discounted, and the failures are thought to have been caused by seepage water from the ground surface resulting from intense rainfall in a short period of time. From the survey of failure time, it is estimated that five out of the six failures occurred after the cumulative rainfall exceeded 250 mm.

## DAMAGE DUE TO DEBRIS FLOWS

### *Topography, Geology and Vegetation of Locations Where Debris Flows Occurred*

Fukui prefecture has drawn up a chart for showing valleys at risk of debris flows before this torrential rain (Ref. 6). Mountain streams at risk of debris flow covered by this chart have been classified according to the type and number of facilities for protection as follows: Type I: Mountain streams flowing into locations where there are five or more houses, or an administrative building, school, hospital, railway station, power generating station etc. Type II: Mountain streams flowing into locations where there is one but fewer than five houses. Type III: Mountain streams at locations where there are currently no buildings, but where the possibility of future building exists (i.e. within city planning areas). For Fukui prefecture overall, nearly 3,000 such locations have been designated, 2002 of Type I, 629 of Type II and 404 of

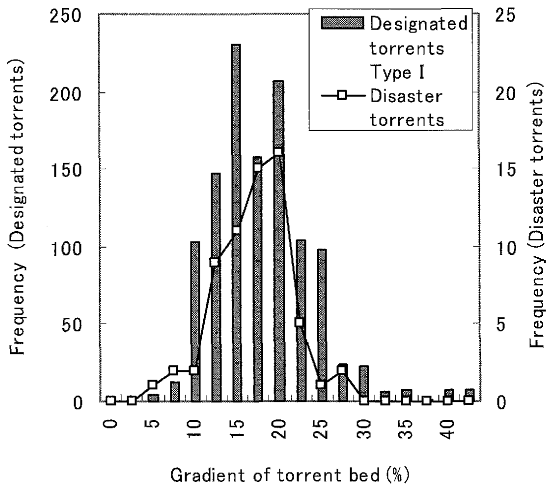



**Fig. 16. Distribution of the torrent bed gradients**

Type III. The chart of these mountain streams gives five types of data: 1) outline information on the streams such as their basin area, gradient of river bed, etc., 2) information on at-risk localities, such as area of inundation, gradient and structures for protection, 3) characteristics of valley floor and flanks of surrounding mountains, 4) flood control installations, and 5) data related to emergency evacuation, etc. Based on data from this debris flow chart and reports of damage from debris flows for Fukui prefecture, our survey team made a study to determine whether or not debris flows occurred under specific conditions of topography, geology or vegetation at locations where they occurred (for general information, *see* Ref. 8 and Ref. 9). The bar graphs in Figs. 16 to 20 correspond to data for valleys at risk of debris flows in the northern region of Fukui prefecture, and the broken line graphs are for data corresponding to locations where debris flows have occurred actually on this occasion. Figure 16 shows the average river bed gradients of torrents at risk of debris flows, which produce a normal distribution with a range from 10° to 25°, with a peak around 15°. For torrents where debris flows have occurred, the peak gradient is around 20° and therefore slightly higher than the anticipated one, but no great disparities were identified between these two kinds of data. Figure 17 shows gradients in inundated areas at the start and closure of the inundation. The gradients of the inundation start points in the at-risk valleys were relatively wide-ranging, from 4° to 16°, with two peaks at around 16° and 10°, whereas the gradients of the closure were in a small range of 0° to 4°, with a concentration around 4°. In terms of the gradients of start and closure points of the inundated areas, no great disparity was observed between the locations where debris flows occurred and those at risk of debris flows. Figure 18 shows valley shapes and types of deposits around torrent beds. Figure 19 shows the overall geology of the basins. In both, the distributions for locations where debris flows occurred and at-risk torrents were similar, and no more or fewer occurrences of debris flows due to specific

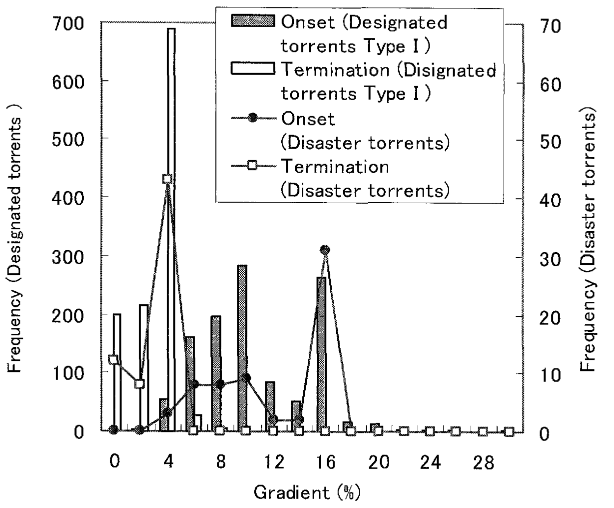



Fig. 17. Gradient of start and closure points of the debris flow

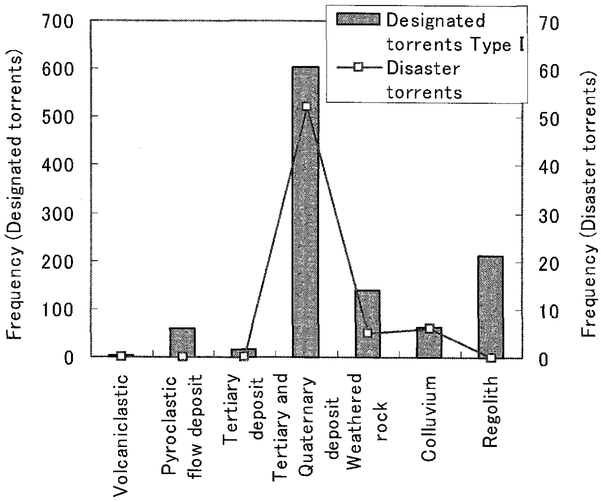



Fig. 19. Geological condition around the watersheds

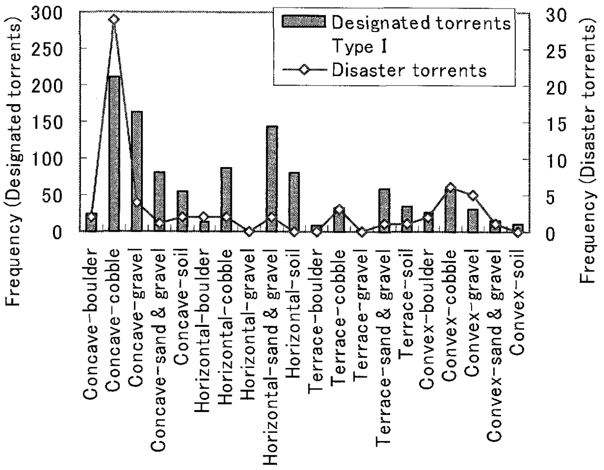



Fig. 18. Topographic feature and soil property around the torrent beds

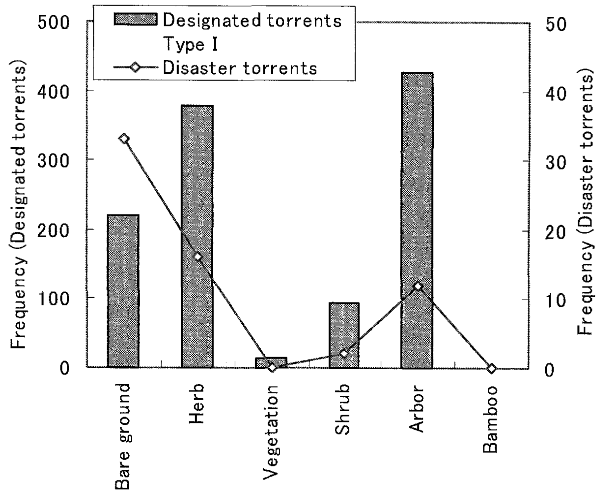



Fig. 20. Vegetation around the torrent beds

topography or soils could be identified. Next, a study was made of debris flows and the vegetation in the surrounding area. Figure 20 shows vegetation in areas surrounding valley floors. The bar graph of overall distribution around torrents at risk of debris flows is in the order of trees, grass, and no vegetation, but at locations where debris flows have occurred actually, it can be seen that in a significantly large number of cases there is no vegetation. Many debris flows in places having no vegetation, may indicate the unstable nature of the material deposited by those torrents, and it can be inferred that many debris flows occurred this time at sites where past shifts have occurred of the valley floor. In Fig. 21 the horizontal axis represents anticipated volume of debris released at points where debris flows have occurred actually this time, and the vertical axis represents actual volume of debris released, plotted for each type of surrounding vegetation. Vegetation on streams where debris flows have occurred, was taken from a Fukui prefecture vegetation distribution map. More than half is shown to be forested cryptomeria. From the map, for pine/cryp-

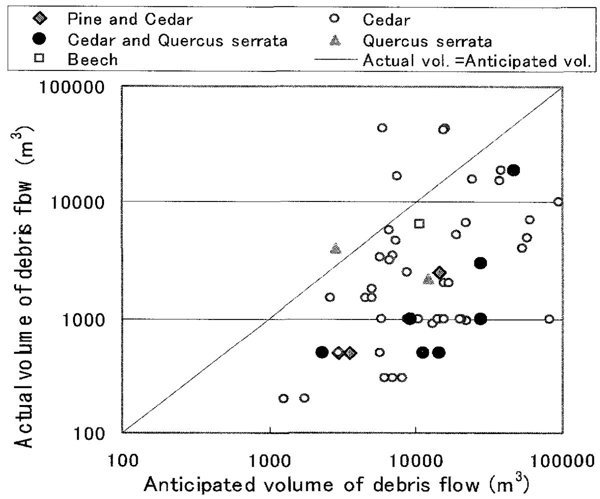



Fig. 21. Effect of surrounding vegetation to the volume of the debris flow

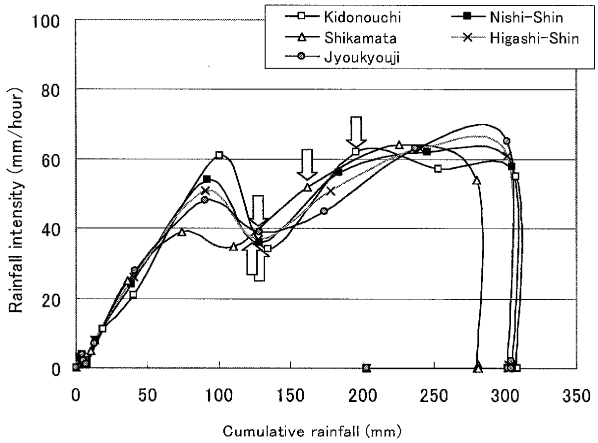



**Fig. 22. Relationship between cumulative rainfall and rainfall intensity (Fukui)**

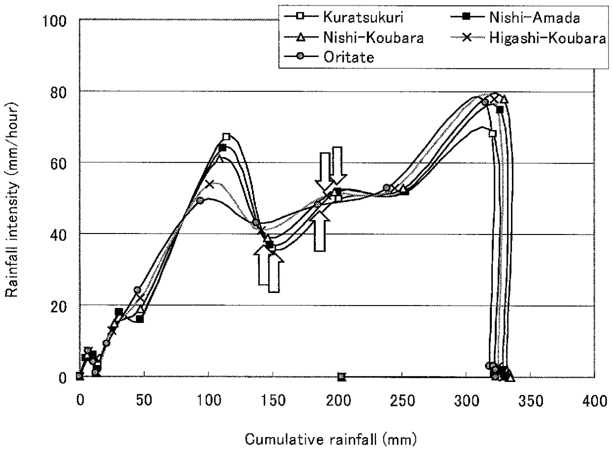



**Fig. 23. Relationship between cumulative rainfall and rainfall intensity (Miyama)**

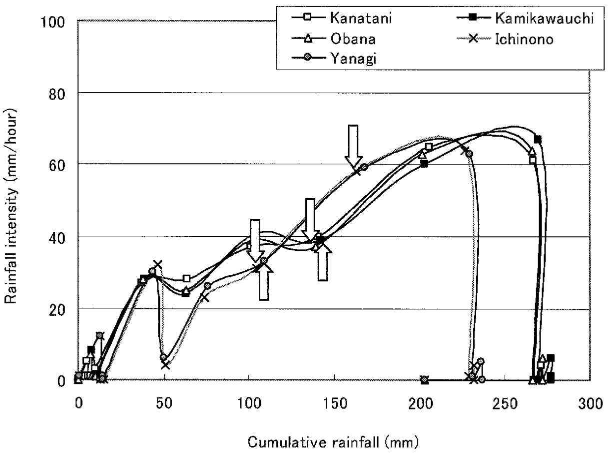



**Fig. 24. Relationship between cumulative rainfall and rainfall intensity (Sabae and Imadate)**

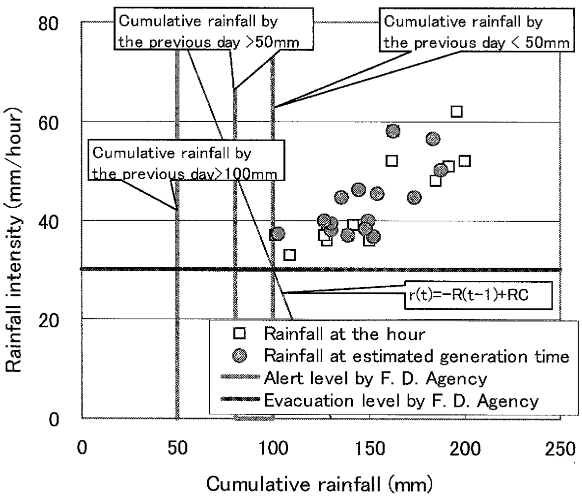



**Fig. 25. Alert and evacuation rainfall criteria**

tomeria and cryptomeria/quercus serrata, the proportion of released mud is low compared with anticipated mud. The proportion released for quercus serata and beech is higher, although there are not many data. However, from the data it is difficult to conclude that the volume of mud released is higher or lower for a specific type of vegetation. As a result, we cannot find specific trends between the distribution of topographical and geological features and vegetation for valleys at risk of debris flows, and a distribution of locations where debris flows occurred actually, except vegetation around the valley floors. A study was made of the correlation between basin area and river floor gradient with the area inundated, etc., but it is omitted here. At last, the relationship between time of occurrence of debris flows and rainfall was studied. Figures 22 through 24 are so-called snake curves showing the relationship between accumulated rainfall and strength of rain at the location of debris flows for Fukui city, Miyama town and Sabae city. Answers to a questionnaire survey on the time of occurrence of debris flows, were averaged for each locality and used as the time debris flows occurred for each stream. The rainfall at that time was marked on the charts with arrows. The figures show that in Fukui City and Miyama town, after heavy rainfall of around 60 mm/hour, rain continued to fall at about 40 mm/hour and debris flows occurred when the accumulated rainfall exceeded about 100 mm. In Sabae city there was little heavy rainfall but accumulated rainfall grew steadily and debris flows were observed once it had exceeded 100 mm. Figure 25 shows a compilation of the arrows of Figs. 22 through 24, in other words the accumulated rainfall and the strength of rain at the time debris flows occurred. In Fig. 25, symbol □ indicates the actual rainfall nearest to the time when debris flows occurred. The time of occurrence is averaged from the questionnaire answers. Symbol ● indicates the rainfall equivalent to the time of occurrence, which is estimated by a linear interpolation between actual rainfalls before and after occurrence. Figure 25 indicates that debris flows resulting from the torrential rains on this occasion occurred at times when accumulated rainfall had exceeded 100 mm, and heavy rain of at least 30 mm/hour was continuing. The horizontal and vertical lines in Fig. 25 indicate criteria for warnings of debris flows or evacuation as issued by the Fire Department. If cumula-

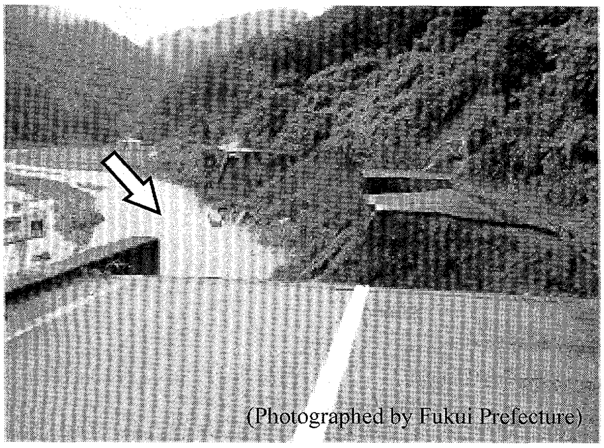



**Photo 1. National road 157 at Nakajima in Ohno city was closed because the outer side of curved river embankment collapsed**

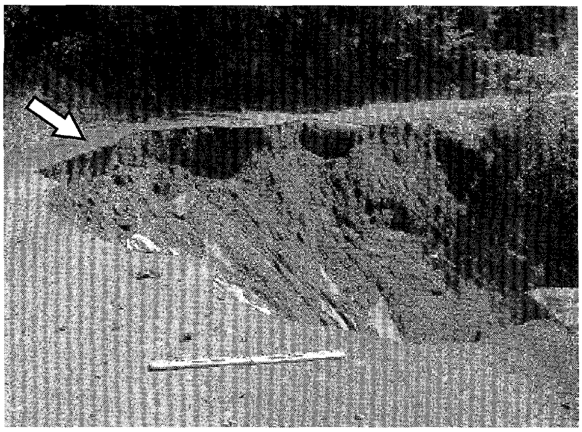


**Photo 2. Prefectural road 34 near the Houkyouji temple in Ikeda town was closed because the flow of rainwater went through from upper part of valley to drainpipe under the road**

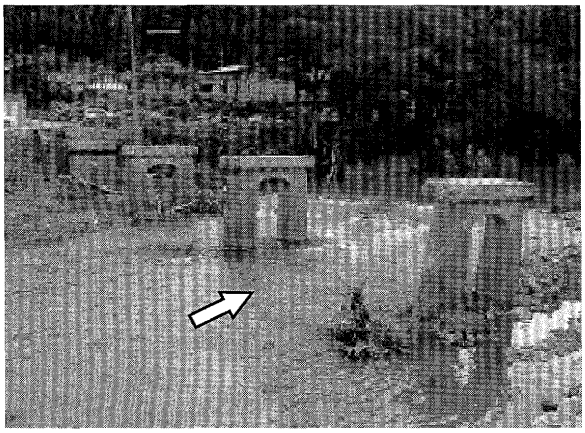


**Photo 3. Superstructure of Kawahara bridge in Miyama town was completely washed away**

tive rainfall by the previous day is less than 50 mm, the first warning stance (patrols and announcements to residents) is taken when the day's rainfall exceeds 100 mm. When the day's rainfall has exceeded 100 mm and heavy rain of about 30 mm/hour starts falling, the second

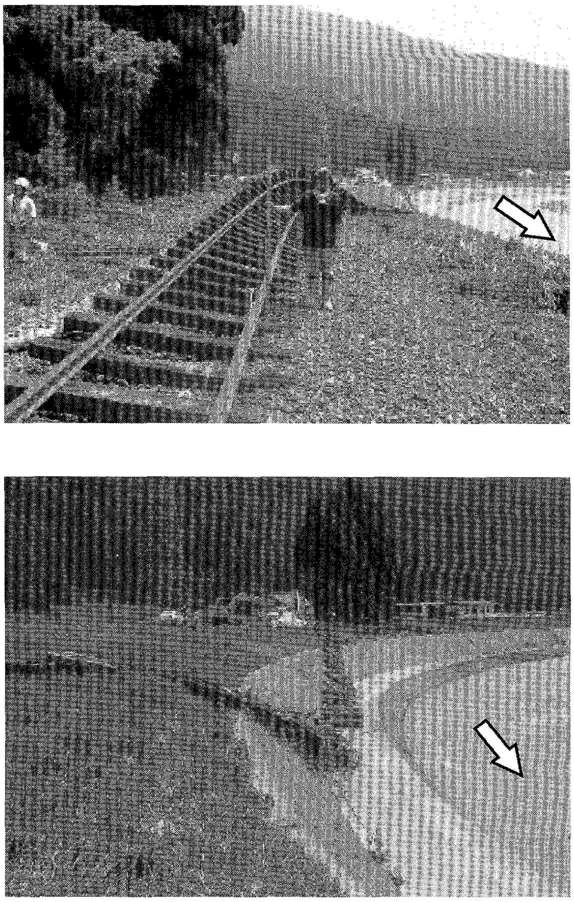
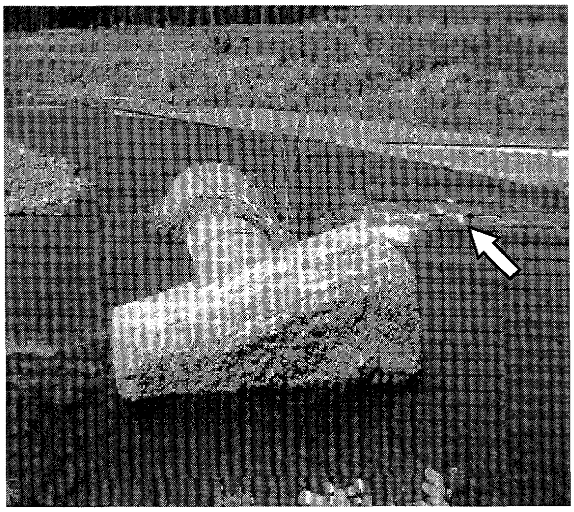


**Photo 4. Rails and sleeper ware moved toward the river due to the overflow at Shinagase in Miyama town**

warning stance (announcements to prepare for evacuation and evacuation orders) is taken. The straight line shown as $r(t)$ in Fig. 25 is set to the debris flow danger criterion line from Ministry of Construction guidelines for when accumulated rainfall passes 100 mm and heavy rain 30 mm/hour. The debris flows on this occasion all fell in the range in which warnings were necessary according to either one of the above criteria. At least it is concluded that the first debris flow to have occurred was in no sense exceptional from the rainfall conditions. As shown in Figs. 22 through 24, after the occurrence of the first debris flow, rain continued to fall more heavily even than before, with rainfall of about twice the criteria level continuing for several hours. Finally accumulated rainfall reached about three times the criteria level, or around 300 mm. Debris flows occurred continuously in waves, releasing large quantities of mud and floodwater with the likelihood of causing considerable damage.

## DAMAGE TO ROAD AND RAIL BEDS

Damage to foundations of road and railway are summarized as follows. 1) Characteristics of damage to roads: The survey of the topographical and geological features and summary of functional damage are omitted here. In river vicinities, damage was concentrated on the outside banks of river bends, and flooding caused road



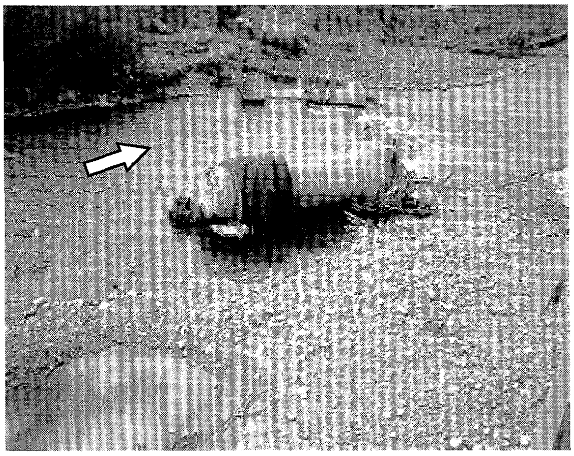


**Photo 5. The 5th Asuwa river railroad bridge collapsed due to flood: The 2nd pier fell down and some gravels of riverbed adhered to the bottom of footing and the 3rd pier was broken at the joint of footing**

revetment collapses and ground scouring behind revetments, which in turn caused road shoulders and hard surface to be washed away (for instance, *see* Photo 1). In the mountain districts, damage was concentrated on roads on embankments located in valley sections, and flooding that occurred along the valleys is seen as causing collapses of ground supporting road beds (for instance, *see* Photo 2). 2) Characteristics of damage to road bridges: There were many cases of damage to road and pedestrian bridges leading to collapse. No bridges along major highways collapsed, and so there was little effect on traffic flows. The form of damage to bridges is classified into the following 4 types, based on subsequent bridge performance: a) Impact or accretion of debris to upper structure, b) Scouring of foundation ground of lower structure, c) Collapse of upper structure due to failure of lower structure, d) Upper structure washed away (for instance, *see* Photo 3). Differences in damage are seen as due to the depth and velocity of the flood waters. The degree of damage was particularly noticeable at locations where river flow became discontinuous, such as at abutment positions and points where smaller rivers join, and at water flow impact sections created by the shape of the river. 3) Rail beds: Damage is classified into two groups. One is damage to track, such as shifting rails, ballast washed away. The other is collapse of embankment or retaining wall supporting the track. A characteristic aspect was that many cases of damage occurred at locations close to level crossings, such as mud deposited around rails, ballast washed away or collapse of retaining walls. Similar damage was seen around culverts built at the bottom of embankments for tunnels or drainage. This damage may be caused by the violent water flows generated because level crossings and culverts became water courses at the flooding time. In addition, rails shifted largely at several locations of level ground, which was seen as the effect of powerful water flows and also the buoyancy of timber rail sleepers (for instance, *see* Photo 4). 4) Damage to rail bridges: Of the 7 bridges on the JR Etsumi North Line in this area, 5 bridges collapsed. Bridge collapses took the two forms; tipping over with foundations and fracturing of supports (for instance, *see* Photo 5). No differences were identified in forms of collapse due to differences in foundation ground, and no differences were found in the degree of damage due to bridge length or number of spans. At bridges collapsed, river water levels are estimated to have reached the bridge girders or the top of the bridge girders. At bridges that did not collapse, water levels are estimated to have reached only the lower surface of the bridge girders even at their highest, because the bridges were located in places where the river or its valley was wide. To study the factors behind bridge collapses, we made stability calculations against tipping and stress calculations of bridge supports which were based on road bridge specifications, considering the effect of flow pressure as water levels reached the top of the bridge girders. The results obtained from this study showed that normally stable bridges tipped over in the case where embedding of footing was shallow, or fractured in the case where the embedding was deep, when the rains on this occasion caused the water to rise over the bridge girders.

## CONCLUDING REMARKS

1) In the breached section, since the dike crest was unpaved, and since the dike consisted mainly of sandy soil, it can be inferred that seepage from the crest due to rainfall and overflow added to seepage from the riverside slope, extending the seepage area and expanding the weakened region. Scouring and seepage resulting from overflow caused a weakening of resistant properties against scouring on the landside slope, and that stronger scouring occurred. 2) In the upper reaches, analyzed were the position and shape of revetment damage, as well as its causes of damage. 3) For many slope failures which occurred along roads, the influences of origin of slope, stratum, topographical section, shape of slope, pre-failure gradient and type of vegetation were investigated, and the frequency distributions of observed failure depth and failure width were studied. 4) For many debris flows

occurred, observed and anticipated results were compared under many items affecting debris flow. 5) A lot of types of damage to foundations of road and railway were summarized. The detailed failure mechanism of scouring, revetment, slope, debris flow and foundation under torrential rainfall will have to be investigated fully in the future. Since the damage resulting from the torrential rains in Fukui was basically damage to rivers, and since the damage of revetment and dike was closely linked to the alignment and conditions of river channels, it is natural to study the causes of the damage from the standpoint of waterways engineering. However, many cases of damage were closely related with geotechnical structures such as dike, revetment, road embankments and retaining walls. This indicates that geotechnical engineering should be actively incorporated in countermeasures to reduce damage from torrential rainfall.

## ACKNOWLEDGEMENTS

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