Chapter 12
The rock coast of Japan

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Abstract: The Japanese islands, situated in a tectonically unstable region with a highly variable geology, are exposed to high wave energy and microtidal environments in most locations. Rocky coasts are common, most having a steep cliff with coastal recession being primarily driven by wave erosion. A fundamental relationship between recession and wave force is obtained through reanalysis of previous laboratory data. On the basis of this relation a model is constructed for the development of type B platforms, that is, horizontal or subhorizontal platforms that have a steep scarp at the seaward edge. The process of wave attenuation on this type of platform and weathering-induced strength reduction of rocks are incorporated into the model. The model is applied to the southwestern coast of the Kii Peninsula and a platform at Ebisu-jima of the Izu Peninsula. Long-term development rates of platforms in the former area are examined: the model indicates that the rate of erosion when platforms were initiated at 6000 years BP is two orders of magnitude greater than present. At the Ebisu-jima platform, wave-induced erosion processes are explored on a daily basis: the model provides a description of temporal variations in platform growth, although the result is not fully satisfactory.

Rocky coast landforms, from the global perspective, differ considerably in morphology reflecting variations in wave climate, tidal range, local tectonics and glacio-hydro isostatic sea-level changes as well as lithological factors. The landforms having developed under present-day marine conditions are largely categorized into two: shore platforms and plunging cliffs. Shore platforms are subdivided into two types: sloping (type A) and horizontal (type B) (Sunamura 1983). The existence of the two types has been known for more than a hundred years since Dana (1849) first reported the presence of horizontal platforms in New Zealand. Sloping platforms are characterized by a gently descending erosion surface extending to beneath sea-level with or without a topographic break (e.g. Trenhaile 1978). Horizontal platforms on the other hand have a horizontal or subhorizontal erosion surface that terminates seawards in a marked scarp. Platforms of this type are well developed on the coast in microtidal environments (spring tidal range: <2 m). In locations without platforms, plunging cliffs occur which are precipitous slopes that plunge below sea-level as a vertical or semivertical face. In spite of the almost 100 year research history of plunging cliffs since Johnson (1925), plunging cliffs have not attracted much attention as compared with shore platforms (Sunamura 1992).

The three types of landforms – sloping platforms, horizontal platforms, and plunging cliffs – are developed along the coastline of Japan, most of which is microtidal and subject to high wave energy. The boundary conditions for the Japanese coast will be briefly described later; they include (a) a tectonic setting and relative sea-level change since mid-Holocene, (b) lithological characteristics together with distribution of rocky coast types, and (c) wave climate and tidal conditions. On the basis of these conditions, some conceptual models for rocky coast evolution will be presented.

Common to the above two types of platform morphologies is that there is a cliff at their landward side. The recession of the cliff leads to platform development. The most important driving force to cause cliff recession is wave action, even if waves have small potential merely to remove highly weathered, loose material and talus deposits at the cliff base (Sunamura 1992, figure 5.1). It may be stated that no evolution of rocky coasts takes place without the action of waves.

Two crucial factors are involved in erosion of rocky coasts: wave action and rock resistance to erosion. An important factor that always acts to diminish the rock resistivity is weathering. An extended period of time is usually required for weathering to lower the rock strength to a level at which wave erosion commences; weathering is therefore a time-dependent factor in the erosional system (Sunamura 1992, figure 5.2). However, quantitative work on the effect of weathering on the reduction of rock resistance to wave action has received little attention in the field of rocky coast studies. Considering the wave and rock factors with the latter having not been weathered, we will attempt to obtain a fundamental equation that can describe wave-induced cliff erosion which is a key process for rocky coast evolution. An equation already proposed by Sunamura (1977) is available, but some difficulty arises in its mathematical treatment. Hence, a simpler relationship will be newly developed through reanalysis of data of previous laboratory experiments.

Although some models have been presented that can be applied to sloping platforms in the macrotidal environments (e.g. Trenhaile 2000, 2005), there have been no models that can describe the development of horizontal platforms under microtidal conditions except Trenhaile’s (2008) model. Based on the new relationship developed in the present study, we will construct a model for this type of platform considering the wave attenuation process and introducing the role of weathering in reducing the strength.

Japan is a series of volcanic island arcs on the northwestern margin of the Pacific Ocean. The Japanese islands, adjacent to convergent-type plate boundaries, have been subject to active crustal movements, volcanism and metamorphism, which yielded complex geological structures and a variety of rock types. These features characterize the substratum of the rocky coasts, which occupy about 60% of the total coastline. The coasts are composed of a wide variety of (a) geological structures ranging from steeply inclined, highly fractured faulted or folded rocks to horizontal sedimentary layers without any visible cracks, and (b) material hardness ranging from strong rocks such as granite and basalt to weak cohesive strata such as Quaternary pyroclastic flow deposits (e.g. Kimura et al. 1991).

The Japanese islands, situated in a tectonically unstable region with a highly varied geology, are exposed to high wave energy and microtidal environments in most locations. Rocky coasts are common, most having a steep cliff with coastal recession being primarily driven by wave erosion. A fundamental relationship between recession and wave force is obtained through reanalysis of previous laboratory data. On the basis of this relation a model is constructed for the development of type B platforms, that is, horizontal or subhorizontal platforms that have a steep scarp at the seaward edge. The process of wave attenuation on this type of platform and weathering-induced strength reduction of rocks are incorporated into the model. The model is applied to the southwestern coast of the Kii Peninsula and a platform at Ebisu-jima of the Izu Peninsula. Long-term development rates of platforms in the former area are examined: the model indicates that the rate of erosion when platforms were initiated at 6000 years BP is two orders of magnitude greater than present. At the Ebisu-jima platform, wave-induced erosion processes are explored on a daily basis: the model provides a description of temporal variations in platform growth, although the result is not fully satisfactory.

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of rocks. The model will be applied to two sites in high-wave-energy, microtidal environments on the Pacific coast of Japan to examine (a) the long-term platform development rate at one site, and (b) the short-term development process using measurement data by a micro-erosion meter (MEM) at the other.

Boundary conditions for Japanese rocky coasts and conceptual evolution models

A tectonic setting and mid to late Holocene relative sea-level change

The geology of the Japanese islands is strongly associated with plate tectonics (Taira 2001). The present-day tectonic situation is illustrated in Figure 12.1a. The northern half of the islands is located on the Okhotsk Plate (formerly the North American Plate), their southern half is on the Eurasian Plate. A small area SW of Tokyo, the Izu Peninsula and its vicinity, is on the Philippine Sea Plate. The Pacific Plate in the northern and southern areas subducts respectively underneath the Okhotsk and the Philippine Sea Plates, and forms the Japan Trench orientated almost north–south. The Philippine Sea Plate descends underneath the Eurasian Plate at the northwestern end along the Nankai Trough and underneath the Okhotsk Plate at the northeastern end along the Sagami Trough. Such a complex tectonic setting of the islands is responsible for active crustal movements including earthquakes, volcanic eruptions and land deformation. Deeper understanding of the formative process of contemporary coastal landforms in Japan requires elucidation for these tectonic factors plus the glacio- and hydro-isostatic factors during the Holocene period, especially the mid- to late Holocene (6 ka BP to present).

Mid-Holocene relative sea-level changes for the Japanese islands have been predicted by Nakada et al. (1991) based on a glacio-hydro isostatic model. The results are found to be similar in spite of different model parameters; as illustrated in Figure 12.1b, in which the contours denote the relative height of sea-level at 6 ka BP. A contour map of relative sea-level shows that its elevation was within a range of ±1 m for the most of the open coast of Japan. Three modes of sea-level variation occur after 6 ka (Fig. 12.1b): (a) a highstand at around 6 ka BP as shown at Sendai;
(b) steady sea-level at present elevation from 6 ka at Muroto; and (c) a sea-level curve approaching present elevations after 6 ka on the western site of Sado Island. These results strongly suggest that glacio-hydro isostasy is of little importance on the relative sea-level variation from the mid-Holocene to present. It is therefore reasonable to consider that the Holocene transgression, with a rapid rising rate of \( c. 1 \text{ cm a}^{-1} \), brought the sea close to its present level about 6000 years ago, and since then the relative sea-level has been almost stationary for most sites on open coasts of Japan.

Sea-level records in Japan are complicated by tectonic and isostatic influences. For example, numbers in Figure 12.1b denote the elevation above present sea-level of palaeo shorelines of mid Holocene age (Ota et al. 2010). This clearly indicates how the Japanese islands are tectonically unstable with many locations being subjected to highly localized large magnitude of crustal uplifts, including two sites with 30 m of uplift. Most upheaval events have been associated with earthquakes (see also Chapter 13, The rock coast of New Zealand (Dickson & Stephenson 2014)). The altitude of the palaeoshoreline in Figure 12.1b is the result of the accumulation of vertical displacements over several coseismic uplifts during the past 6000 years.

Lithology and rocky coast landforms

The Japanese islands have grown along the continental margin of Asia since the Permian and their evolution is characterized by subduction tectonics (Taira 2001). Palaeogene accretionary complexes, regional metamorphic rocks and granites constitute basement rocks of the islands which are overlain by more recent volcanic products, Neogene sedimentary rocks and Quaternary deposits.

For studies of process geomorphology, it is important to treat rocks or strata as landform materials having hardness or strength rather than as geological units. In this context, a nationwide lithological map depicted on the basis of quantified hardness is useful, but such a map is unavailable owing to a dearth of strength data. Figure 12.2b shows a map of the lithology of Japan, in which basement rocks, volcanic rocks such as andesite and basalt, and sedimentary rocks older than the Neogene are represented as ‘hard’ rocks, Neogene sedimentary rocks as ‘intermediate-strength’ rocks, and less consolidated pyroclastics and Quaternary deposits (except alluvium) as ‘soft’ rocks.

Of the three kinds of rocky coast landforms commonly found on Japanese coasts (Fig. 12.2a), type A platforms always develop on the exposed or sheltered coasts wherever they are composed of soft

![Fig. 12.2.](image-url)
rocks, whereas type B platforms are commonly found on the open coast of intermediate-strength or hard rocks, and plunging cliffs occur on the hard-rock coast (Fig. 12.2b). Actually, the occurrence of these morphologies depends not only on rock resistance but on the intensity of marine agents. In Figure 12.3 the demarcation of these three landform types is given, using data taken mostly from Japan, on the assumption that (a) the magnitude of marine agents may be represented by a physical quantity, \( \rho gH \), where \( H \) is the largest height of waves occurring to a coast under consideration, \( \rho \) is the density of water and \( g \) is the acceleration owing to gravity (Sunamura 1992). Reproduced by permission of John Wiley & Sons.

\[
\frac{\rho gH}{S_c} \leq 1.3 \times 10^{-2}, \quad \text{type A platforms}
\]

\[
\frac{\rho gH}{S_c} < 1.7 \times 10^{-3}, \quad \text{type B platforms}
\]

\[
\frac{\rho gH}{S_c} > 1.7 \times 10^{-3}, \quad \text{plunging cliffs}
\]

Oceanographic conditions: wave climate and tides

Most of Japan’s coasts face the Pacific Ocean or the Sea of Japan, only the small northeastern area being exposed to the Sea of Okhotsk and the southwestern portion facing the East China Sea (Fig. 12.4a). Instrument-based measurements of waves (mainly by ultrasonic-type wave gauges (UWG)) have been conducted in many coastal waters by the Port and Airport Research Institute (PARI). We selected 31 measuring sites located in the open sea, from which wave data for the past 20–30 years are available (Nagai 2002; Nagai & Ogawa 2003; Nagai & Satomi 2005, 2006; Shimizu et al. 2007, 2008; Kawai et al. 2009, 2010). The height and period of the maximum significant waves ever recorded during the measurement term at each site are described in Figure 12.4b. The height and period of storm waves are respectively 9–11 m and 9–15 s off the southwestern coast facing the Pacific Ocean, 6–10 m and 10–17 s in the northern part of the Pacific coast, and 7–11 m and 8–13 s off the Sea of Japan coast. The summarized data of extremely large waves show no marked regional difference in wave climate. However, ordinary storm waves, which occur once or twice a year, have been generally recognized to have smaller heights and shorter periods at the Sea of Japan side, compared with those at the Pacific side, because the fetch of the former waters is limited.

Storm waves off the southwestern coast of the Pacific side are caused by typhoons in summer–autumn seasons, while those occurring in the Sea of Japan are generated by strong low-pressure systems, mainly in winter. Waves off the northern coast are caused either by typhoons or by low pressures. Large waves on the Sea of Okhotsk coast are unlikely to occur owing to (a) shallow waters with a limited fetch; and (b) freezing in winter (Isozaki & Suzuki 1999). Wave magnitude of the East China Sea is considered to be similar to that of the Sea of Japan. Normal waves under calm sea conditions off the Pacific coast are 0.5–1.5 m in height and 7–9 s in period, and those off the Sea of Japan coast are 1.0–1.2 m in height with a 5.5–6.5 s period.

Most coasts are situated in a microtidal environment as shown in Figure 12.4b; the tidal range on the Sea of Japan coasts is especially small, 0.1–0.2 m, and that on the Sea of Okhotsk coasts is 0.5 m; the northern half of the Pacific coasts is in a range of 0.6–1 m and the southern half is in a range of 1.1–
1.9 m. The spring tidal range on the coasts facing the East China Sea and the Seto Inland Sea is mesotidal. A very localized area in sheltered waters connected to the East China Sea is exposed to the largest range in Japan (4.9 m). The tidal type of Japan is semidiurnal-dominant mixed or diurnal-dominant mixed tides depending on the location.

Rocky coast evolution models under uplift conditions

We will construct some models for the evolution of rocky coasts that have experienced coseismic uplift during the mid- to late-Holocene of \(<1\)–2 m. This is the prevailing situation on Japan’s open coasts. Coasts under consideration are assumed to be composed of insoluble, uniform rocks with no marked structural influence and to be situated in a microtidal environment. Coasts made from highly resistant rocks, where little morphological change occurs in any circumstances \((\rho g H_s / S_c < 1.7 \times 10^{-2}\) in Fig. 12.3), are not considered. Tidal fluctuations are neglected in the modelling, that is, constant water level. A simple scenario (Fig. 12.5a) in which three uplift events with the same magnitude at the same interval of time during the past 6000 years will be adopted, and the most severe wave condition for causing morphological changes will be taken into account.

Let us first take a case of coasts satisfying \(\rho g H_s / S_c \geq 1.3 \times 10^{-2}\) (Fig. 12.3), which is named model A. Immediately after the postglacial marine transgression that ceased at 6000 years ago, a steeply inclined initial cliff is assumed to be exposed to breaking waves (stage 1 in Fig. 12.5b), although considerable modification might have occurred on the cliff during the rapid sea-
level rise. The action of breaking waves just in front of the cliff begins to form a type A platform with vigorous horizontal cutting of the landward cliff and concurrent vertical lowering of the shallow bedrock owing to the low erosional resistance of the rock. The cliff recedes with the cliff–platform junction being at sea-level. An elevated platform geometry immediately after the first uplift occurred at $t_1$ (Fig. 12.5c) is depicted by the symbol $t_1^*$ (stage 2 in Fig. 12.5b). The position of wave-breaking, the breaker position, shifts offshore owing to a decrease in water depth caused by the uplift. Soon after the uplift, a low cliff cut into the elevated platform appears (‘c’ at stage 2 in Fig. 12.5b) and it recedes landward to the high cliff in time. Eventually waves begin to cut back the high landward cliff, which results in the formation of a gentler platform. A similar erosion process recurs after the second and third uplift events, and no evidence of uplift is left on the final platform profile (stage 4 in Fig. 12.5b). Irrespective of the initial boundary condition, steep or gentle, the final platform profile does not preserve elevated features, as far as the condition $\frac{\rho g H_t}{S_c} \geq 1.3 \times 10^{-2}$ is fulfilled.

The next case is for coasts with a condition of $1.7 \times 10^{-3} \leq \frac{\rho g H_t}{S_c} < 1.3 \times 10^{-2}$ (Fig. 12.3). The morphological response of these shores is quite different for this case depending on the initial boundary condition (Sunamura 1992, fig. 7.25); therefore, two boundary conditions are taken into account: (a) a gently
sloping profile with a uniform gradient and (b) a steeply inclined profile. For the former condition, a gently sloping case (named model B-1), Figure 12.5c shows that each stage is characterized by the development of a type A platform, each of which has receded by the same distance because the energy of waves reaching the coast remains constant immediately after uplift (owing to unvaried breacher position from the coast). The final profile has three elevated platforms (stage 4 in Fig. 12.5c), providing evidence of the uplift.

For the case of a steep initial slope, two models are considered: models B-2 and C. For both models, horizontal cutting takes place into the slope at the first stage (stage 1 in Fig. 12.5d, e) to yield a type B platform, the elevation of which is assumed to reside at sea-level, ignoring the influence of rock strength on the platform elevation (e.g. Sunamura 1991; Dickson 2006; Thornton & Stephenson 2006). There are two modes of the subsequent landform evolution: one is the evolution with a series of stepped landforms in response to uplift events (model B-2; Fig. 12.5d) and the other is the landform developing without any marked features of upheaval (model C; Fig. 12.5e). In either case, erosion does not occur on the initial seaward slope. The former model (B-2) describes the recession of both seaward and landward cliffs of an elevated platform with a slight lowering of its surface. The latter model (C) indicates that the retreat of landward cliff of an elevated platform and the lowering of its surface occurs either simultaneously or after some time period. This model is constructed on the premise that the lithology is so vulnerable to abrasive processes and sub-aerial weathering that the platform surface is easily lowered (e.g. Porter et al. 2010). This facilitates wave propagation across the platform, allowing for waves to attack the landward cliff and cause erosion. The two distinctive morphologies are shown in stage 4 in Figure 12.5d and e, respectively, and they often coexist along a small stretch of the Japanese coast when the lithology is similar. Physical parameters that enable demarcation of the two different landforms have not been scrutinized.

A basic equation for cliff erosion by waves

Modelling the role of physical erosion in the evolution of rocky coasts requires the establishment of a rudimentary relation describing wave-induced cliff erosion. An application of linear automatic coast treatment of equation (2) owing to the logarithmic function; so the present study attempts to establish a simpler relation for $F$. The following linear function is assumed for $F$:

$$ F = (F_W/F_R) - 1 \text{ for } F_W > F_R \tag{3a} $$

$$ F = 0 \text{ for } F_W \leq F_R. \tag{3b} $$

Waves exert two kinds of assaulting force on the cliff face: hydraulic and mechanical. The hydraulic action consists of compression, tension and shearing. On some occasions it also includes wedge action of compressed air in a joint or fault-associated opening in a cliff. When waves are armed with sediment, mechanical action arises and consists of abrasive and impact forces. Almost simultaneous occurrence of these processes characterizes wave assault force.

It is emphasized that, without hydraulic action, waves do not exert any force on a cliff. Hydraulic action is of vital importance and is directly related to wave energy. With the purpose of examining the assaulting force of breaking waves, Sunamura (2010) derived force from the kinetic energy of water particles at the crest of breaking waves against the cliff face, and obtained:

$$ F_W = \rho g H_b \tag{4} $$

where $H_b$ is the height of waves just in front of the cliff irrespective of breaking and broken waves and $A$ is a dimensionless coefficient. Note that $F_W$ has a unit of stress, $[\text{FL}^{-2}]$.

In order for equation (3a) to have a physical significance, the rock resisting force $F_R$ must have the same units as $F_W$. There are three main rock-strength indices having units of stress: uniaxial compressive, tensile and shearing strength, which are positively correlated each other. Uniaxial compressive strength has been employed as a surrogate for the resisting force of rocks because it is a widely used index with well-established testing criteria (Sunamura 1992, pp. 52–63). Uniaxial compressive strength is hereafter referred to as ‘compressive strength’. The resisting force of rocks, $F_R$, is assumed to be proportional to the compressive strength, $S_c$:

$$ F_R = B S_c \tag{5} $$

where $B$ is a dimensionless coefficient.

Compressive strength values can be obtained through a testing machine installed in a laboratory by crushing a precisely cut rock specimen. Compression testing, however, needs expensive facilities and elaborate work. Contrary to this, non-destructive tests are available for assessing compressive strength by the use of three kinds of light, portable and economical devices: the Schmidt hammer, the Equotip hardness tester and a needle-type penetrometer, all of which can be used for rapid estimation of in situ rock hardness. The Schmidt hammer has been widely employed in geomorphological research (Goudie 2006); hardness is estimated from the distance of rebound of a plunger after its collision with a rock surface. There have been many conversion formulas to relate the hammer rebound value to compressive strength (references cited in Goudie 2006) and evaluation of compressive strength from the Schmidt Hammer hardness is possible through an appropriate formula. The Equotip hardness tester has a similar principle in the measuring system to the Schmidt hammer. Attempts have been made to examine the relationship between the Equotip reading and compressive strength (e.g. Votaw & Mulder 1993; Aoki & Matsukura 2008) and to apply this instrument to geomorphological studies (Aoki & Matsukura 2007; Viles et al. 2011). A needle-type penetrometer (e.g. Suzuki & Hachinohe 1995; Nga-Tillard et al. 2011), a specially designed penetrometer for weak rocks, has a correlation chart
between compressive strength and penetration readings given by the manufacturer.

Substitution of equations (4) and (5) into equation (3a) yields:

\[ F = \kappa (pgh_f/S_c) - 1 \quad \kappa = A/B. \]  

(6)

From equations (1) and (6), one obtains:

\[ \frac{dX}{dt} = K [(pgh_f/S_c) - \varepsilon], \]

(7)

where \( K \) is a coefficient with the same unit as erosion rate, [LT\(^{-1}\)], and

\[ K = C/\varepsilon, \]

(8)

where \( \varepsilon \) is a dimensionless coefficient that denotes

\[ \varepsilon = 1/\kappa = B/A. \]

(9)

Examination of the validity of equation (7) and determination of the coefficients \( K \) and \( \varepsilon \) require several datasets that contain (a) the rate of erosion, (b) the height of waves that caused erosion and (c) the strength of cliff-forming rocks. For this purpose data obtained under controlled laboratory conditions will be used.

Six datasets (Fig. 12.6) are selected from two experiments of cliff erosion conducted using breaking waves (Sunamura 1973, 1991), and five datasets (Fig. 12.7) are from two experiments using broken waves (Sunamura 1973). Figure 12.6 shows the temporal change of a notch cut in a steep model cliff in the breaking-wave test, in which waves were allowed to break immediately in front of a model cliff. Figure 12.7 illustrates the notch development that occurred in the broken-wave test, in which waves after breaking acted on the cliff face. The model cliff was made of a mixture of Portland cement, well-sorted fine quartz sand (0.2 mm in diameter) and water. By changing the mix ratio, model cliffs with different strengths were constructed. The advantage of Portland cement is that, after curing, the strength of the material remains constant (Sunamura 1992, pp. 82–83). The model cliffs were unweathered and lacked discontinuities (cracks and fissures), so that no variation in the resisting force of cliff material occurred during the experiment. Tides were not considered with water level being kept constant. The experimental conditions are listed in Tables 12.1 and 12.2.

Erosion experiments on model cliffs composed of the sand–cement mixture frequently showed the following erosional process. Hydraulic action of waves first erodes the cliff, which results in a supply of sand to the cliff base, which is increasingly deposited in front of the cliff with time. Waves then begin to use the sand as abrasives to exert mechanical action on the cliff. At this stage the wave assailing force dramatically intensifies to give rise to an abrupt increase in erosion rate (Sunamura 1976).

Erosion distance is defined as the horizontal distance from the cliff face to the deepest part of a notch. All data of the breaking-wave experiment (Fig. 12.8) and no. 105 of the broken-wave test (Fig. 12.9) indicate that erosion distance gradually decreases with time. Such a temporal change can be described by the following exponential function:

\[ X = M(1 - e^{-Nt}), \]

(10)

where \( M \) and \( N \) are coefficients with units of [L] and [T\(^{-1}\)], respectively. On the other hand, four datasets of the broken-wave experiment (nos 101–104 in Fig. 12.9) show that erosion can be expressed by a linear function of time.

The rate of erosion at the very early stage of the experiment should be employed to relate cliff erosion to the height of input waves immediately in front of the cliff: the assailing force of waves alters its characteristics with time as a notch develops, leading to the change in notch configuration. For cases of the linear \( X - t \) relation, as shown by nos. 101–104 in Figure 12.9, the gradient of the straight line will be employed as the initial rate of erosion. For the other cases expressed in equation (10),
the following relation, obtained by differentiating this equation and then inserting \( t = 0 \), will be used to calculate the initial rate:

\[
\left( \frac{dX}{dt} \right)_{t=0} = MN. \tag{11}
\]

The values of \( M \) and \( N \) are given beside the curves in Figures 12.8 and 12.9.

Values for the erosion rate, \( \frac{dX}{dt} \), in Tables 12.1 and 12.2 are thus obtained and the values are plotted against \( \rho g H_f/S_c \) in Figure 12.10; the straight line shows the result of a best fit of equation (7) to the data. The two points at which the best-fit lines intercept the x-axis give the respective values of \( \varepsilon \) for the breaking-wave and broken-wave tests (inset in Fig. 12.10). The two lines denote that:

\[ K = 264 \text{ mm h}^{-1} \text{ and } \varepsilon = 0.0015 \text{ for breaking waves} \tag{12a} \]
\[ K = 95.6 \text{ mm h}^{-1} \text{ and } \varepsilon = 0.0040 \text{ for broken waves}. \tag{12b} \]

*Obtained from the gradient of the straight line in Figure 12.9.

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**Table 12.1. Experiment condition and results for breaking waves**

| Dataset no. | Wave height \( H_f \) (cm) | Wave period \( T \) (s) | Compressive strength \( S_c \) (kPa) | \( \rho g H_f/S_c \) | Erosion rate \( \frac{dX}{dt} \) (cm h\(^{-1}\)) | Remarks |
|-------------|----------------------------|-------------------------|-----------------------------------|----------------|---------------------------------|---------|
| 1           | 7.0                        | 2.0                     | 15.7                              | 0.044          | 1.1                             | Sunamura (1973, figure 106b) |
| 2           | 4.0                        | 1.2                     | 10.8                              | 0.036          | 0.68                            | Sunamura (1991) |
| 3           | 6.0                        | 1.2                     | 10.8                              | 0.055          | 1.4                             | Sunamura (1991) |
| 4           | 10                         | 1.2                     | 10.8                              | 0.091          | 2.5                             | Sunamura (1991) |
| 5           | 5.0                        | 1.2                     | 52.9                              | 0.0093         | 0.070                           | Sunamura (1991) |
| 6           | 7.0                        | 1.2                     | 52.9                              | 0.013          | 0.35                            | Sunamura (1991) |

**Table 12.2. Experiment condition and results for broken waves**

| Dataset no. | Wave height \( H_f \) (cm) | Wave period \( T \) (s) | Compressive strength \( S_c \) (kPa) | \( \rho g H_f/S_c \) | Erosion rate \( \frac{dX}{dt} \) (cm h\(^{-1}\)) | Remarks |
|-------------|----------------------------|-------------------------|-----------------------------------|----------------|---------------------------------|---------|
| 101         | 2.1                        | 1.2                     | 33.3                              | 0.0062         | 0.028*                          | Sunamura (1973, figure 74) |
| 102         | 2.4                        | 1.2                     | 33.3                              | 0.0071         | 0.025*                          | Sunamura (1973, figure 74) |
| 103         | 1.8                        | 1.2                     | 33.3                              | 0.0053         | 0.011*                          | Sunamura (1973, figure 74) |
| 104         | 3                          | 2                       | 15.7                              | 0.0088         | 0.049*                          | Sunamura (1973, figure 74) |
| 105         | 7.9                        | 2                       | 15.7                              | 0.049          | 0.43                            | Sunamura (1973, figure 108, section B-B) |

*Obtained from the gradient of the straight line in Figure 12.9.

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**Fig. 12.8.** Temporal variation in erosion distance in breaking-wave experiments.
Because \( e = B/A \) (equation (9)), \( A = 667B \) for the case of breaking waves from equation (12a). If the resisting force of rocks can be represented by the value of compressive strength (for simplicity, \( B = 1 \)), equation (5) reduces to:

\[
F_R = S_c \tag{13}
\]

then the assailing force of breaking waves should be written as:

\[
F_W = 667 \rho g H_B \tag{14}
\]

where \( H_B \) is the height of breaking waves. Similarly, from equation (12b) we have the following equation for the broken-wave case:

\[
F_W = 250 \rho g H_b \tag{15}
\]

where \( H_b \) is the height of broken waves. Comparison of equations (14) and (15) indicates that the assailing force of a breaking wave is 2.7 times greater than that of a broken wave if they have the same height when acting on a cliff without discontinuities such as joints and faults.

From equations (8), (12a) and (12b) we obtain:

\[
C = 0.396 \text{ mm h}^{-1} \text{ for breaking waves} \tag{16a}
\]

\[
C = 0.382 \text{ mm h}^{-1} \text{ for broken waves}. \tag{16b}
\]

These results indicate that \( C \) takes on a similar value. This implies that equation (1) is valid, and hence equation (7) can be applied to rocky coast erosion problems if the unknown coefficients \( K \) and \( \varepsilon \) can be reasonably determined from field data. Some models have been presented to predict the future morphological change of rocky coasts as a result of sea-level rise (e.g. Sunamura 1988; Trenhaile 2011), which are not constructed on a sound physical basis. Equation (7) would be useful in reconstructing such a predictive model.

A physical model for type B platform development in Japan

Wave height attenuation on type B platforms

In considering the evolution of type B platforms, we should explore both horizontal and vertical landform changes. The former change occurs first and the latter follows, or both changes occur simultaneously; in either case the former change is of a much larger order of magnitude. Although the vertical change, that is, lowering of the platform surface (e.g. Stephenson & Kirk 1998, 2000b) is actively occurring especially in tectonically unstable regions like Japan, the horizontal cutting into a cliff is essential for the development of type B platforms. The term ‘horizontal cutting’ is used in this paper to denote that horizontal erosion is dominant over vertical erosion. The present modelling will only consider the horizontal component.

The driving force for furthering the platform growth is obviously the action of waves at the foot of the inland cliff, regardless of whether the cliff material is weathered or not. This wave action is closely associated with the hydrodynamic properties of water movement across a platform. Studies of wave behaviour on coral reefs have been intensively conducted since the early 1980s; they include field measurements (e.g. Gerritsen 1980; Hardy & Young 1996; Kench & Brander 2006), laboratory experiments (e.g. Kono & Tsukayama 1980; Gourlay 1994) and theoretical and numerical models (e.g. Massel & Gourlay 2000; Monroy & Sato 2003; Madin et al. 2006). In contrast, research into wave behaviour on shore platforms has been lacking. Stephenson & Kirk’s (2000a) work was the first, concluding that waves exert little effect on platform development at Kaikoura, New Zealand. Hydrodynamic research on type B platforms has been recently performed also in New Zealand by Beetham & Kench (2011) and Ogawa et al. (2011, 2012): they have elucidated not only the process of wave attenuation but also the presence of infragravity waves that influence wave behaviour on platforms. However, infragravity waves themselves have no direct effect of erosion on the landward cliff, but they act as an energy carrier of the waves which have the potential to cause erosion and control the elevation of the assailing force of waves. Water depth on platforms affects the characteristics of infragravity waves, which means that it is an important controlling factor for wave attenuation.
Results of numerical modelling by Madin et al. (2006) emphasize the effect of water depth on wave attenuation over a coral reef having a horizontal surface just like a type B platform; their results strongly suggest that (a) wave height tends to decay exponentially with the distance from the reef edge; and (b) the degree of decay depends on water depth—the more gradual attenuation occurs with greater depth. These two suggestions led us to write the following equation for wave height attenuation:

$$\frac{H}{H_e} = \alpha (e^{-\beta h} - 1) + 1,$$

(17)

where $H_e$ is the height of waves on the reef edge, $H$ is the height of waves at a distance, $X$, from the edge, $\alpha$ is a dimensionless coefficient representing the effect of water depth, and $\beta$ is a decay coefficient with units of [L$^{-1}$]. The range of $\alpha$ is 0 $\leq \alpha \leq 1$; $\alpha = 0$ means that no wave transformation occurs owing to large water depth, while $\alpha = 1$ indicates that a reef platform is free from water, a dry bed.

Prior to application of equation (17) to type B platform development, determination of the two coefficients $\alpha$ and $\beta$ will be made using data obtained from a subhorizontal type B platform at Tatabouri, North Island, New Zealand by Ogawa et al. (2011). It should be noted that the data are limited in quantity and have been collected under calm sea conditions. Figure 12.11 shows the result of a best-fit of equation (17) to the data; in this figure $h$ denotes the water depth, averaged over the shore platform with a slight seaward inclination, and the parameter $h/H_e$ is the relative water depth. Although some scatter of data points is seen, a general trend can be represented by equation (17): slower wave attenuation occurs with increasing relative water depth. The values of $\alpha$ and $\beta$, determined from the best-fit curve, are plotted respectively against $h/H_e$ in Figure 12.12a and b. The $\alpha$ v. $h/H_e$ relation (Fig. 12.12a) is given by:

$$\alpha = \exp[-0.016 (h/H_e)^4].$$

(18)

The $\beta$ v. $h/H_e$ relation (Fig. 12.12b) is described by:

$$\beta = 0.030 \exp[-0.70 (h/H_e)],$$

(19)

where $\beta$ has a unit of [m$^{-1}$].

Knowledge of wave height on the seaward edge of shore platforms and tide conditions (water depth on the platform) enables us to obtain $\alpha$ and $\beta$ from equations (18) and (19), respectively, and to calculate wave height in an arbitrary distance from the seaward edge from equation (17).

Almost all type B platforms in Japan are located in the intertidal zone or slightly above mean high water spring (MHWS). Spring tidal range is less than 2 m on the Pacific coast and 0.1–0.2 m on the Sea of Japan coast. Storm surge height is considered to be several tens of centimetres at maximum on the open coast. Under these conditions, we will estimate a possible maximum water depth on shore platforms that are supposed to be situated at mean low water spring (MLWS), when they are under the action of severe storm waves at spring high tide. The estimation indicated that water depth does not exceed 3 m even on the Pacific coast. Assuming that the height of severe storm waves (usually in a range between 6 and 10 m) is equivalent to that of waves on the edge of platforms, we have $h/H_e < 1$. This leads to $\alpha \approx 1$ through equation (18) (Fig. 12.12a). Inserting $\alpha = 1$ into equation (17) leads to:

$$\frac{H}{H_e} = e^{-\beta h}.$$

(20)

This relation can be used to calculate the height of waves reaching the landward cliff of type B shore platforms in a coastal region with small values of $h/H_e (< 1)$, such as in Japan.

**Modelling**

We will attempt to introduce the strength of rocks suffering from weathering into the model. Takahashi et al. (1994) investigated the effect of wet–dry weathering on erosion using Pliocene sandstone blocks used for a masonry bridge pier constructed in 1951 on the Aoshima coast in Miyazaki where the mean spring tidal range is 1.6 m. The pier is a truncated pyramid 2.5 m high and has a precipitous wall on the four sides; it is situated at about 10 cm above mean sea level (MSL) on a type B shore platform. The platform width in front of the south-facing wall is approximately 250 m, measured in the direction of dominant wave incidence. The platform developing in front of the north-facing wall is slightly

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Fig. 12.11. Wave height attenuation on a type B platform. Data from Ogawa et al. (2011, figs 2a & 7b).

Fig. 12.12. (a) Dependence of $\alpha$ in equation (17) upon relative water depth, $h/H_e$. (b) Dependence of $\beta$ in equation (17) upon relative water depth, $h/H_e$. 
lower in elevation with a width of only 20–30 m. Erosion distances of sandstone blocks located in the upper part (1.7–2.5 m above MSL) of the supratidal zone of the south- and the north-facing walls are estimated respectively at 110 and 35 mm during the period of 38 years from 1951 to 1989 (Takahashi et al. 1994, figure 11.7). The average erosion rate is 2.9 mm a$^{-1}$ on the south wall, while it is 0.92 mm a$^{-1}$ on the north wall: the former is three times greater than the latter. Waves reaching the north wall at high tides throw up masses of water along the wall face, and spray and splash reach the higher portion of the wall. Because the north wall is immune from direct solar radiation it is always damp, even at low tides. The south wall is also exposed to waves at high tides, but waves reaching the wall are significantly attenuated by the presence of the wide platform. In contrast to the north wall, the south wall receives the full amount of insolation (Takahashi et al. 1994). The upper portion of the wall is wetted only by spray and splash associated with frequent storm waves. We consider that this process has caused much reduction in rock strength, which has resulted in considerable erosion in spite of lower frequency and intensity of wave attack. This strongly suggests that the efficacy of wet–dry weathering in reducing rock strength as the duration of drying increases.

At the first stage of type B platform formation where the effect of wet–dry weathering is minimal because of the damp intertidal zone of an initial steep cliff, waves act on the cliff to give rise to horizontal cutting, although the erosion surface formed by hydraulic action alone is not smooth but rugged. One of the roles of weathering is to plane the rugged surface and to lower the overall surface of the platforms once formed. Another significant role of weathering is to decrease the resistance of the landward cliff to wave erosion. We consider that, as platforms develop through increasing width, the material of the landward cliff reduces its strength owing to wet–dry weathering. Actually the strength reduction is a function of time and space. However, modelling both variables is beyond the present state of knowledge. Considering simply that the weathering-induced reduction in cliff strength proceeds as the width of platforms increases, we assume that the strength ratio of weathered rocks to intact ones follows an exponential decreasing function of distance from the platform seaward edge:

$$\frac{S_f}{S_e} = e^{-\gamma X},$$

where $S_f$ is the compressive strength of the landward cliff surface and $S_e$ is the compressive strength of unweathered rocks, and $\gamma$ is a weathering coefficient with a unit of [m$^{-1}$]. The value of $\gamma$ decreases with increasing resistance to weathering, and $\gamma = 0$ means that no weathering occurs.

A basic relation for erosion of the landward cliff can be obtained from equation (7) by replacing $H_i$ with $H$ and $S_e$ with $S_f$:

$$dX/dt = K[(pgH/S_e^2) - e],$$

where $H$ is the height of waves in front of the landward cliff at a distance $X$ from the seaward edge of the platform. From equations (20)–(22), we obtain:

$$dX/dt = K[(pgHe e^{-\delta X}/S_e) - e],$$

where

$$\delta = \beta - \gamma.$$  

The coefficient, $\delta$, with a unit of [m$^{-1}$], includes the effects of wave attenuation and rock weathering. Integration of equation (23) with the initial condition of $X = 0$ at $t = 0$ yields:

$$X = \frac{1}{\delta} \ln \left\{ \frac{1}{\varepsilon} \left[ \Phi - (\Phi - \varepsilon) e^{-K\delta t} \right] \right\},$$

where

$$\Phi = pgHe/S_e.$$  

Equation (25) indicates that the parameter $\Phi$ should be greater than $\varepsilon$ for platforms to develop ($X > 0$); thus, $\varepsilon$ denotes the threshold value for platform growth:

$$\varepsilon = \Phi.$$  

Equation (25) describes that type B platforms develop rapidly at the initial stage and later they attain equilibrium with time, and the final width is:

$$X_{\text{final}} = (1/\delta) \ln (\Phi/\varepsilon).$$

The ultimate width is dependent on (a) wave attenuation on the platform and weathering resistivity of the landward cliff material, both incorporated in $\delta$, (b) input wave height and compressive strength of rocks, both included in $\Phi$, and (c) the threshold value for platform development, $\varepsilon$. Differentiation of equation (25) gives the temporal change in the rate of platform development:

$$dX/dt = \frac{K(\Phi - e e^{-K\delta t})}{(\Phi - (\Phi - \varepsilon) e^{-K\delta t})}.$$  

The maximum rate, which appears at the initial stage, is given by:

$$dX/dt_{\text{max}} = K(\Phi - e).$$

Application 1: long-term growth rate of type B platforms on the southwestern coast of the Kii Peninsula

In order to examine long-term rate of platform development, an attempt will be made to apply the present model (equation (25)) to an existing dataset of platform width and rock resistance, collected by Davies et al. (2004) from the SW Kii Peninsula coast, facing the Pacific Ocean and covering 65 km of shoreline (Fig. 12.4c). Miocene sedimentary rocks of Muro and Tanabe Groups are exposed in the coastal zone. The Muro Group consists of an alternating sequence of sandstone, mudstone and siltstone of varying thickness with some intercalated pebble conglomerate. The Tanabe Group, consisting of siltstone, conglomerate and massive sandstone, unconformably overlies the Muro Group.

Mid-Holocene shoreline features are found at elevations of 5–6 m above sea-level and are the result of coseismic uplift occurring at intervals of 500–1500 years (Maemoku & Tsubono 1990). Type B shore platforms are well developed in the study area (Takahashi 1973) and can be divided into two morphologies: some of them are characterized by the presence of an elevated ledge at the seaward side (stage 4 in Fig. 12.5d) and the others are shelf platforms without uplifted features (stage 4 in Fig. 12.5e). Davies et al. (2004) selected 14 sites from the latter platforms that have a horizontal or slightly seaward-inclined surface. Water depth at the seaward cliff, measured from MSL, termed the front depth, varies between a few and several metres; however, precise data on front depth are not available. The platform width ranges from 50 to 250 m. Most of the platforms reside within the upper intertidal zone between MSL and MHWS, 0.7 m above MSL, some being at or slightly above MHWS. The study area is in a microtidal setting with a mean range of about 1 m, observed at Tanabe in the middle of the study area by the Hydrographic Department, Maritime Safety Agency.

Ocean wave measurements using a discus buoy, operated by the PARI, from 1983 to 1997 were utilized, the buoy being located 25 km SW of the study area at a water depth of 170 m. The wave climate off the southwestern coast of the Kii Peninsula is characterized by an annual average wave height of 1.2 m and period of 7 s, with severe (once a year) storm waves being 7–9 m in height (period 11–15 s; Nagai 2002). The largest waves
recorded, caused by typhoons in summer–autumn seasons, were 11.4 m in height (period 13.8 s).

Platform width was measured in the shore-normal direction from the base of the landward cliff to the top of the seaward cliff by use of telemetric equipment and tapes. Rock hardness testing was carried out employing an N-type Schmidt hammer, which was applied to platform-forming rocks of visually judged ‘unweathered’ nature. Davies et al. (2004) represented the rock resistance in terms of hammer rebound values. Application of equation (25) requires the conversion of the rebound numbers to compressive strength values and Kahraman’s (2001) relation for the N-type hammer will be used for this:

\[ S_c = 6.97 e^{0.014r_R}, \] (31)

where \( S_c \) is the compressive strength in MPa, \( R_r \) is the rebound number, and \( \rho_r \) is the rock density in g cm\(^{-3}\).

Let us assume that large storm waves, occurring once or twice a year, result in marked morphological changes on the platform and are responsible for its initiation and development irrespective of the degree of weathering of the cliffs. According to wave characteristics under storm conditions, waves 8 m high would be considered as representative of the erosive agents in the study area. We assume that (a) no regional difference is present in wave properties, and (b) when the 8 m waves run up onto the platforms, waves with a similar height occur on the platform edge: \( H_l = 8 \) m. This value plus \( \rho = 10^3 \) kg m\(^{-3}\) and \( g = 9.8 \) m s\(^{-2}\) will be employed to rewrite equation (26):

\[ \Phi = 0.08 \frac{MPa}{S_c}, \] (32)

where \( S_c \) should have a unit of MPa. The parameter \( \Phi \) is expressed as a function of \( S_c \).

Figure 12.13 shows the relationship between the platform width and compressive strength, and although considerable scatter is present, there is a general trend that the width decreases with increasing rock strength. From this figure, we assume that the critical value for rock strength against erosion under the action of 8 m waves is 80 MPa. Substituting \( S_c = 80 \) MPa into equation (32), and then from equation (27), we have:

\[ \varepsilon = 0.001. \] (33)

Assuming that (a) type B platforms in the study area commenced developing 6000 years ago and (b) the tectonic movements since then have little affected the horizontal platform morphology, we will examine a general trend of the width v. strength relationship (Fig. 12.13) through calculation of equation (25). The calculation will be made replacing \( \Phi \) in equation (25) with equation (32), substituting \( t = 6000 \) a and \( \varepsilon = 0.001 \), and selecting suitable values for \( K \) and \( \delta \). The curve in Figure 12.13 is drawn with \( K = 85 \) m a\(^{-1}\) and \( \delta = 0.006 \) m\(^{-1}\). Equation (25) represents a general trend if appropriate determination of the coefficients (\( K \), \( \delta \) and \( \varepsilon \)) is made. The vertical dashed line at the left margin of the graph denotes a line of \( S_c = 6.2 \) MPa, the value for delineating type A and type B platforms, which is obtained through substitution of \( H_l = 8 \) m into the relation: \( \rho g H_l / S_c = 0.013 \) in Figure 12.3.

Applying the coefficient values, \( K = 85 \) m a\(^{-1}\), \( \delta = 0.006 \) m\(^{-1}\) and \( \varepsilon = 0.001 \), the relationship between platform growth rate and rock resistance will be examined through equation (29). To calculate this equation, we selected \( t = 0, 2000, 4000 \) and 6000 years. The result is plotted in Figure 12.14. The diagram shows that growth rates of platform width at a given time decrease with increasing rock strength as anticipated, and they dramatically drop as the strength approaches 80 MPa, the critical \( S_c \)-value. The curve for \( t = 0 \) has a point of inflection (at c. 40 MPa); the
growth rate markedly increases from this point towards the type A/type B platform boundary, 6.2 MPa. The other three curves are convex-upward and do not exhibit such a marked change in growth rates.

Reading the diagram downwards along a vertical line through a specific strength gives the temporal change in growth rates for a site with that strength. A considerable growth-rate decrease with time can be recognized irrespective of strength-values. Comparison of the curves for \( t = 0 \) and \( t = 6000 \) yields that the rate of platform initiation is two orders of magnitude greater than that of the present-day platform development, that is, the cliff recession rate at present being on the order of \( 10^{-1} \) to \( 10^{0} \) mm a\(^{-1}\). The model indicates that the platforms in the study area have begun to form at markedly high rates: \( 2.5 \times 10^{2} \) mm a\(^{-1}\) at a site composed of soft rocks (\( S_{c} = 20 \) MPa) and \( 1.2 \times 10^{1} \) mm a\(^{-1}\) at a site of hard rocks (\( S_{c} = 70 \) MPa). It should be noted that these rates provide us only with an average because they are based on \( K, \delta \) and \( \varepsilon \)-values used to depict the general trend passing through the middle of a cluster of data (Fig. 12.13).

Application 2: short-term development process of Ebisu-jima platform, Shimoda

Equation (22) will be applied to surface erosion rates as measured by a MEM on the landward cliff of a platform on the southeastern coast of the Izu Peninsula (Fig. 12.4b). The study site, Ebisu-jima, is located at the southern tip of the Susaki Promontory, east of Shimoda (Fig. 12.4d). Ebisu-jima is a small, flat-topped, pear-shaped island, about 150 m long and 100 m wide. A 20 m high marine terrace (of unknown age) occurs on the top of the island. Ishibashi et al. (1979) reported that a site near the base of the promontory was uplifted by 1.6 m at 1300–1500 a BP, but no evidence of uplift is found on or around Ebisu-jima. The southern half of the Izu Peninsula is mostly covered with Neogene volcanics (Sumi 1957), of which tuffaceous sandstone and andesite breccia of the Pliocene Shirahama Group are exposed. A body of tuffaceous sandstone dips c. 20° towards the NW, in which scoria-rich layers are intercalated, while andesitic breccia overlies the tuffaceous sandstone on the island (Fig. 12.15).

A type B shore platform fringes the island; it has a maximum width of 50 m on the most exposed southern side and a minimum width of 5 m on the most protected northeastern side. The eastern two-thirds of the southern platform are composed of tuffaceous sandstone and the western third is made of andesitic breccia. The sandstone platform is almost horizontal (Fig. 12.15) with a relief of less than several tens of centimetres related to protrusions of scoria-rich layers. Ramparts with a relative height of c. 1 m occur in some places around the seaward edge of the platform. Furrows less than 1 m deep develop along major fault lines. The height of platform ranges between 0.7 and 1.2 m above MSL with no significant difference in height between the sandstone and breccia units. There is little loose rock material on the platform surface.

The cliff landward of the sandstone platform, where the maximum width attains 50 m, was selected as an MEM measuring site. The cliff is steep (c. 80°) and about 10 m high. The upper part of the cliff is composed of volcanic breccia and the lower is sandstone. The MEM measurement was performed on the surface of the sandstone where there were no visible cracks or fissures (Fig. 12.16); its elevation is 1.5 m above the cliff–platform...
Wave data during the MEM measurement period are available from the Shimoda measuring station of the PARI using an ultrasonic-type wave gauge (UWG), 1 km SW of the study site (Fig. 12.4d), at a water depth of 50 m. Unfortunately, there is a lack of wave data during five months from August to December 2007. The data for this period were supplemented by those collected at the Habu measuring station (in a water depth of 49 m) of the PARI, 40 km east of the study site. The PARI data at both sites include time-series records of significant wave height and period, processed based on measurements at 2 h intervals. Other wave data are available from the Irozaki measurement station of the Japan Meteorological Agency (JMA), 15 km SW of the study site, where an UWG has been set up at 50 m water depth. The JMA has also provided tidal records at an observational station on the Irozaki coast near the station.

Under normal sea conditions, no waves can reach the MEM measuring site even at high tides; however, storm waves can reach the landward cliff. Most storm waves reaching the southern Izu coast are generated by typhoons, or by low-pressure systems. Storm waves with 6–8 m height and 8–12 s period were recorded at Shimoda (or Habu) during the 4.3 year MEM monitoring period.

Defining the erosion distance during a storm event as $\Delta X$ and the duration of storm waves causing erosion as $\Delta t$, we have the following equation rewritten from equation (22):

$$\Delta X = K[(\rho gH^*/S'_{\text{crit}}) - e]\Delta t,$$

(34)

where $H^*$ is the wave height at the inland cliff base 50 m far from the platform edge. The value of $S'_{\text{crit}}$ can be represented by the compressive strength value obtained from the Equotip testing: $S'_{\text{crit}} = 4.0$ MPa, which will be treated as a constant in relation to time and space. For the calculation of erosion distance by this equation, it is necessary to determine $e$, the threshold value for cliff erosion, and to estimate $H^*$.

**Determination of the threshold value $e$.** Because $S'_{\text{crit}} = \text{const.}$, the $e$-value depends on the minimum height of erosion-causing waves at the cliff-platform junction:

$$e = \frac{pg(H^*)_{\text{crit}}}{S'_{\text{crit}}},$$

(35)

where $(H^*)_{\text{crit}}$ is the critical wave height for erosion. The surface of the substrate forming the landward cliff is highly weathered as described before, so that detachment of some sand grains from the MEM measurement point is likely to occur if up-rushing waves reach the point. Knowledge is needed of the minimum height of waves at the cliff base which could reach the MEM measuring site at 1.5 m elevation.

In order to examine the relationship between the height of waves rushing to a cliff and their run-up height on the cliff, a simple laboratory test was conducted by one of the authors, H. Aoki, using a mini-sized wave flume (3 m long, 0.3 m high and 0.1 m wide in which a plunger-type wave maker was installed at one end and a horizontal model platform was set up at the other. A steep model cliff (slope angle 80°) was placed on the platform at various distances from its edge where laboratory waves were forced to break. Broken waves forming bores run towards the cliff, and their behaviour around the cliff-platform junction was recorded using a video camera. From the video images the height of waves reaching just in front of the cliff, $H^*$, and the run-up height to the cliff, $R$ (vertical distance), were measured. The result showed that $R/H^* = 3.5$. Based on this result, the minimum height of waves to affect the MEM site was determined to be 0.5 m: $(H^*)_{\text{crit}} = 0.5$ m. Substituting this value and $S'_{\text{crit}} = 4.0$ MPa into equation (35), one obtains:

$$e = 0.00125.$$  

(36)
Estimation of wave height $H^*$. Not only deep-water wave characteristics but also water depth on the platform strongly affect the height of waves approaching the foot of the landward cliff. The water depth is controlled by astronomical tide fluctuations plus episodic sea-level changes such as storm surges. Figure 12.17 shows the relationship between significant height of waves and the height of storm surges (deviation from the astronomical tide), plotted using the JMA’s wave and tidal data at Irozaki. In general storm surge is positively related to wave height. Although large scatter of data points is found, a general trend can be described by the S-shaped curve equation:

$$\eta = 0.9(1 - 2e^{-0.17 H_s} + e^{-0.34 H_s}),$$  \hspace{1cm} (37)

where $\eta$ is the storm surge height and $H_s$ is the significant wave height, both having the unit of [m].

The following procedure will be introduced to estimate the wave height $H^*$ that exceeds 0.5 m:

1. Given significant wave height $H_s$ from the PARI time-series data at Shimoda or Habu, we will evaluate the value of $\eta$ from equation (37) on the premise that the $H_s - \eta$ relation at Irozaki can be applied to the study site. The elevation of astronomical tides at the time of occurrence of waves under consideration is denoted as $h_0$, measured from MSL (Fig. 12.18) and $h_i$ can be obtained from the JMA’s tide table. Let us assume that the platform surface is perfectly horizontal and situated at MHWS, 0.7 m above MSL (Fig. 12.18), that is, the level of the cliff-platform junction. A further assumption is that waves occurring when the height of the tide level plus storm surges exceeds the platform elevation, that is, $h_i + \eta > 0.7$ m, are likely to produce waves having a height of more than 0.5 m at the inland cliff base. In this situation water depth on the platform, $h_i$ is given as:

$$h_i = h_0 + \eta - 0.7 \text{ m}. \hspace{1cm} (38)$$

In this calculation the effect of wave setup is ignored.

2. Using the period, $T$, and the height, $H_s$, of significant waves measured at the water depth of wave gauge installation, $H^*$, the wave height in deep water, $H_o$, is calculated using linear wave theory; $H^* = 50 \text{ m}$ is applied to the Shimoda data and 49 m to the Habu data. The tidal fluctuation is ignored for both calculations. Knowing the value of $H^*/H_o$ ($L_o$ is the deep water wavelength, $L_o = (g/2\pi f)^{1/2}$), we obtain the value $= H_i/H_o$, from which $H_o$ is derived.

3. The water depth in front of the platform, denoted as $h_i$, is given by (Fig. 12.18):

$$h_i = h_0 + \eta - 6 \text{ m}. \hspace{1cm} (39)$$

4. The wave height in front of the platform, $H_i$, is calculated from the value of $H_i/H_o$ via $h_i/L_o$ again using linear wave theory. In this calculation wave reflection from a steep seaward cliff is not considered.

5. If the wave height on the seaward edge of the platform, $H_e$, can be approximated by $H_i/H_o$, then the value of $\beta$ is obtained from equation (19) because $h_i$ is given by equation (38). Substitution of $X = 50 \text{ m}$ and $H = H^*$ into equation (20) leads to:

$$H^*/H_o = e^{-0.50f}. \hspace{1cm} (40)$$

From this equation, we can finally estimate the wave height $H^*$.

Calculation of equation (34) and the result. Using $H^*$ thus obtained, the erosion distance $\Delta X$ was calculated on a daily basis by use of the following equation that is obtained by substituting $\Delta x = 4.0 \text{ MPa}$ and $\varepsilon = 0.00125$ into equation (34):

$$\Delta X = K[(\rho g H^*)/4.0 \text{ MPa}] - 0.00125|\Delta t|. \hspace{1cm} (41)$$

In the calculation, the value of $\Delta t$, the duration of erosion-causing waves ($H^*>0.5 \text{ m}$), is determined using tidal curves at the Irozaki observatory and the time-series wave data acquired at 2 h intervals at Shimoda or Habu. It should be noticed that calculation of $\Delta X$ involves an unknown coefficient $K$ at this stage. The erosion value will be determined after calibration of $K$.

Actual recession distances measured by the MEM are plotted by the star symbol in Figure 12.19a and temporal variations in the wave height in front of the platform $H_i$, which is supposed to be equivalent to $H_e$, the wave height on the platform edge, are illustrated in the bar chart of Figure 12.19b. The first term of the 4.3 year MEM measurements is denoted as I at the bottom of the diagram and the following three terms are indicated by II–IV.

Equating $\sum \Delta X$ to the actual total recession distance, 13.4 mm, led to $K = 74 \text{ mm h}^{-1}$, which enabled calculation of $\Delta X$ at each storm event. The value of $\Delta X$ is exhibited in a bar chart at the bottom of Figure 12.19a. The dotted line in this figure denotes the cumulative erosion distance that is plotted by adding the individual erosion distance. This step-like distance v. time relationship indicates that the calculation result is considerably larger than the actual measurements of the terms I–III – almost double.

Figure 12.19 indicates that significant erosion ($\Delta X > 0.5 \text{ mm}$) took place under the action of waves with an approximate height

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Fig. 12.17. Height of storm surge plotted against significant height of waves. JMA’s wave and tidal data at Irozaki were used. Waves more than 2 m in height were selected from data during the MEM measurement term of 4.3 years (17 May 2004 to 31 August 2008).

Fig. 12.18. Idealized profile of the Ebisu-jima platform and definition sketch.
waves had a daily-averaged, offshore height of 7.5 m ($H_o = 7.5$ m) and a period of 12 s ($T = 12$ s). The tidal record shows that a combined effect of storm surges and high water at diurnal tides usually occurring in this season raised water level about 0.2 m higher above the platform ($h_t = 0.2$ m, and hence $h_i = 6.9$ m) and kept it there for a long time, approximately 10 h ($\Delta t = 10$ h). Calculation using the values of $H_o$, $T$ and $h_i$ yielded that $H_i = 8.4$ m, leading to $H_e = 8.4$ m. Because $h/H_e = 0.024$, we had $\beta = 0.03$ from equation (19); equation (40) led to $H'/H_e = 0.22$, which gave $H' = 1.8$ m. Substituting the $H'$-, $\Delta t$- and $K$-values into equation (41), we obtained $\Delta X = 2.4$ mm. Unfortunately, no actual erosion data were available during term IV, which makes it impossible to validate this result.

**Discussion**

**Wave assailing force and rock resisting force**

Trenhaile (2008) constructed a model to describe the development of subhorizontal platforms; however, the resistance of rocks is not explicitly expressed in his model. The present model is based on equation (7), which includes both wave assailing force and rock resisting force, represented respectively by equations (4) and (5). Equation (7) is validated using data of laboratory experiments with no inclusion of weathering, discontinuity and abrasive effects. In actual field situations, however, weathering processes, discontinuities in rocks and abrasive action constitute major elements in the rocky coast erosion system (e.g. Sunamura 1992, figure 5.2).

Weathering is directly related to the rock resisting force. If coastal rocks are weathered, then we must replace $S_e$ in equation (5) with reduced strength owing to weathering. The strength of rocks subjected to weathering has been measured by many researchers using Schmidt hammers (references cited in Aoki & Matsukura 2007). Weathering of a coastal rock mass without discontinuities begins at the rock surface and penetrates into the interior with time, which results in the formation of strength gradient, that is, weathering profile, with the weakest part being located at the surface forming a thin layer. For testing this thin weathered layer a Schmidt hammer is not a suitable device (Aoki & Matsukura 2007), rather an Equotip hardness tester or a needle-type penetrometer is used. Using the Equotip tester, Aoki & Matsukura (2007) investigated weathering-induced strength reduction of sandstone blocks used for a masonry bridge pier at Aoshima, described before, and reported that the surface strength of the blocks located in the inter- to supratidal environment reduced to 70% of that of fresh blocks. In the Ebisu-jima study, the compressive strength converted from the Equotip readings was applied. A needle-type penetrometer was used for a study on strength reduction of coastal rocks owing to weathering (Suzuki & Hachinohe 1995; Hachinohe et al. 1999a, b). They applied it to three cores of sandstone, which were obtained by drilling vertically from the surface of three uplifted coastal terraces with different ages near the tip of the Boso Peninsula, Japan, and Hachinohe et al. (1999a) revealed the temporal change in weathering profiles.

The effect of discontinuities on rocky coast morphologies and processes has been intensively studied; some recent studies include Benumof et al. (2000), Duperret et al. (2004), Hénaff et al. (2006), Trenhaile & Kanyaya (2007), Cruslock et al. (2010), Kennedy (2010) and Naylor & Stephenson (2010). Meso-scale (centimetre to metre order) erosion often occurs on the surface of shore platforms in a short period of time in a mode of ‘quarrying’ or ‘plucking’, the removal of blocks bounded by joints and faults, with the erosion scale depending on the dimension of blocks. Kennedy & Beban (2005) reported from the Wellington coast, New Zealand that the highly fractured nature of bedrocks yielding palm-sized fragments facilitates severe downwearing on the platform surface. Naylor & Stephenson (2010) demonstrated the importance of role of discontinuities in altering rock resistance from their study at Glamorgan, UK and Marengo, Australia. Benumof & Griggs (1999) found that rock properties determine the rate and mode of cliff retreat on the San Diego County coast, California; erosion rate is best correlated with joint spacing.

The presence of discontinuities in the surface of a rock mass influences not only the erosional resistance of rocks but also the assailing force of waves. When waves hit a cliff face having...
discontinuities, they exert the force to grow the interstice wider and deeper by a well-known process, wedge action. Yamamoto et al. (1990) have attempted through a laboratory experiment to measure wave pressures inside a V-shaped crevice in a vertical cliff installed in a wave flume. Considerable rise in pressures within the crevice was recorded, the result being summarized in Sunamura (1994). Recent laboratory work by Wolters & Müller (2004) has indicated a high possibility of crack growth owing to incessant wave action on a sea cliff, Adams et al. (2005) emphasized that long-lasting cyclic loading to cliff-forming rocks could generate crack initiation and propagation, leading to strength reduction. The process, called fatigue (e.g. Sunamura 1992, pp. 68–69), brings about deterioration of overall rock mass strength or facilitation of detachment of blocks, or both, depending on rock type.

Discontinuities play a twofold role: they augment the wave assailing force $F_w$, and diminish the rock resisting force $F_R$, yielding favourable conditions for waves to expedite erosion. This means that, compared with the case of a sea cliff with no significant discontinuities in the cliff face coefficient $A$ in equation (4) takes a smaller value, while $B$ in equation (5) has a smaller one, so that the value of $e$ in equation (7), a threshold value for cliff erosion, reduces because $e = B/A$, implying that erosion occurs with smaller wave height. In this study we used $F_k = S_e$ (equation (13)) taking $B = 1$; for such a case, $A$ should have much larger value for $e$ to have the same value.

Abraslon has been recognized to be a major process operating on rocky coasts; however, there have been some instrument measurements (by use of MEM), including the first study of Robinson (1977), and recent ones of Fooke et al. (2006) and Blanco-Chao et al. (2007). For a deeper insight into abrasion processes, measuring abrasive forces and rock resistivity as well as the resultant topographic change are requisite. Aside from rock strength properties, field measurements of abrasive forces have been lacking. There has been only one study: Williams & Roberts (1995) attempted to measure impact forces of waves armed with pebbles on the macrotidal coast of south Wales; they used a specially designed instrument to provide measurements of the momentum of pebbles.

Clastic sediments on rocky coasts act both as an abrasive tool to accelerate erosion as well as a protective layer to decelerate it in the other, as clearly demonstrated in a wave flume experiment by Sunamura (1976); the role of sediments is contradictory. This depends on a dynamic balance between wave energy and the amount and size of material mobilized (Sunamura 1982). Such an ambivalent role has been observed on chalk shore platforms of the Channel coasts (Costa et al. 2006; Hénaff et al. 2006). The quantification of such a role is difficult in the field; even in a well-controlled laboratory environment the quantification was not sufficient: no generalized relation has been presented (Sunamura 1982).

It is difficult at present to evaluate individually the role of weathering, discontinuities and abrasion and to incorporate them into the $A$-value in equation (4). However, such incorporation is not always necessary for the application of equation (7) or its integrated form, equation (25), if the value of $e$ can be determined from field data showing the causality relationship, such as that presented in Figure 12.13. The $e$-value obtained from such data contains collectively the effect of the three factors. Regarding the case of Kii Peninsula coast, $e = 0.001$ (equation (33)), which indicates $A = 1000$ because of $B = 1$, leading to $F_k = 1000pghH_t$. The physical quantity, $pghH_t$, itself is wave pressure intensity in a hydraulic sense, and does not imply the assailing force of waves at all. The wave assailing force is a quantity obtained by multiplying $pghH_t$ by 1000 in this case, because the resisting force of rocks is represented by compressive strength values themselves, equation (13).

### Platform width

The width of type B platforms is a fundamental component of platform morphology (Stephenson 2001). The platform width is the horizontal distance between the landward and seaward cliffs and the stability of the latter has been debated: does the seaward cliff retreat or not? If the seaward scarp does not recede, the width increases monotonously with time as the landward cliff is cut back; otherwise, the width cannot be described in such a simple way.

One of the authors, T. Sunamura, has asserted that the seaward descent does not recede based on (a) laboratory findings that the assailing force acting on the submerged part of the seaward cliff much diminishes owing to the buffer effect of water (Sunamura 1975, 1991); and (b) field findings that rich marine flora and/or fauna continue to cover the cliff face even immediately after violent storm wave assault (Sunamura 1992, p. 167). No field measurements of the recession of seaward cliffs had been conducted until Stephenson’s (2001) analysis using aerial photographs was first made, which provided us with the result that no measurable recession could be detected. Dickson (2006) has reported that the seaward cliff does not retreat, but cliffs of eolianites are undercut, so that the overhanging roof block becomes unstable and lies detached seaward of the platform edge, which results in cliff recession. He considered that such undercutting is presumably caused by some combination of solution and biological erosion. Trenhaile (2008) also reported the presence of a collapsed block at the edge of the seaward cliff of eolianites. An investigation by Kennedy et al. (2011) stated that the seaward cliff is receding from the fact that block failures had occurred, owing probably to undercutting. They inferred that joint-associated, deeply incised furrows are destroying the once-formed platform geometry.

Apart from coasts made from soluble rocks and coasts where the platform geometry is highly governed by the structural weakness, it is reasonable to consider that the seaward cliff does not retreat during platform development in microtidal environments under stationary sea-level conditions from the mid Holocene to present. Based on this consideration, the present study attempted to construct a model to describe the recession of the landward cliff, that is, the platform width, which is described by equation (25).

Trenhaile’s (1999) shore-platform study using data from three areas – (a) Vale of Glamorgan in Wales, UK, (b) Gaspé in Québec, Canada and (c) southern Kii Peninsula, Japan – presented the conclusion that the platform width is determined by the intensity of wave action, the resistance of rocks and the length of time. These three variables are all incorporated in equation (25). As far as the present-day platform width is concerned, the time factor can be assumed to be constant. As a result, this equation shows that the platform width increases with increasing wave exposure if a local difference in rock resistance is small in the region under consideration, while wider platforms develop in weaker rocks. The former relation has been reported from the southwestern coast of Japan (Takahashi 1974) and the Wellington coast in New Zealand (Kennedy & Beban 2005), and the latter from the Glamorgan Heritage coast, UK (Davies et al. 2004) and Lord Howe Island in the Tasman Sea (Dickson 2006).

Applying equation (25) to the southwestern coast of the Kii Peninsula (Fig. 12.13), we could quantify the platform width $v$. rock strength relation on the assumption of uniformity of wave incidence to the coast. Actually, however, the wave assailing force may be different at each measuring site. The examination of this point was not possible owing to a lack of data to enable evaluation of local wave magnitude. A quantitative relationship between the platform width and the two controlling factors, waves and rocks, is still difficult to establish.

Assuming that little recession of the seaward cliff has occurred during the platform development since mid-Holocene when the sea
reached the present level, de Lange & Moon (2005) estimated long-term mean recession rate of sea cliffs from the width of platforms cut in soft flysch lithologies of the Waiotapu Group (upper Oligocene to lower Miocene), Auckland, New Zealand. The calculated erosion rate was 1.4–14.3 mm a\(^{-1}\), robust results (8.0 mm a\(^{-1}\)) at the North Shore site and 1.8–13.8 mm a\(^{-1}\) (5.3 mm a\(^{-1}\)) at the Tawharanui Peninsula site.

The present model (equation (25)) shows that the platform width increases with time, the relation being represented by an upward convex curve. Therefore, the calculation using such a platform-width method as de Lange & Moon (2005) employed provides a larger value than the present rate of cliff erosion obtained from equation (29) with substitution of \( t = t_0 \) (the duration of platform growth) after determination of the other coefficients. Incidentally, such an erosion rate calculated from the model must be treated with caution, because it may mask large fluctuations reflecting sporadic nature of present-day cliff recession processes (Sunamura 1992, pp. 102–106).

**The role of weathering in the landward cliff recession**

In an attempt to gain a better understanding of the role of waves in the development of type B platforms, we examined the short-term recession of the landward cliff at the Ebisu-jima platform, employing wave and cliff recession data. Figure 12.19a shows that the model calculation of landward recession produced a considerably overestimating outcome in term I, the first period of MEM measurements. This has the effect that the subsequent cumulative relationship between distance and time was overestimated. Term I has a period of about 8 months from 17 May 2004 to 12 January 2005, during which the site experienced three large storm events (over 2 months) with a wave height, \( H_c \), exceeding 5 m. Compared with terms II–IV, the frequency of such sizable waves is much higher.

The overestimation of erosion in term I could probably be related to the strength of cliff material used for the model calculation. The strength was treated as a constant, \( S^\star = 4.0 \) MPa, based on the Eqotip measurement of weathered tuffaceous sandstone. Once the weathered portion was eroded, then the less weathered interior would have constituted the rock surface and therefore the cliff would have been of higher resistance to erosion. It is conceivable that a certain period of time is necessary for the substrate forming the new rock surface to weather and deteriorate and that this time was too short in term I with the frequent storm attacks on the cliff. Nevertheless, a constant low strength value was used for the calculation, which might have caused the overestimation of erosion in term I.

Let us take a different viewpoint in which we assume that the landward cliff had been weathered prior to the beginning of the MEM measurement such that its surface to 10 mm depth had already reduced in strength to 4.0 MPa. We will attempt to then determine a \( K \)-value through a best fit of cumulative erosion distance for the three measurements for terms I–III. The best fit result is depicted by the solid, step-like line in Figure 12.19a, which led to \( K = 411 \) mm h\(^{-1}\). The final value of the cumulative result is much smaller than the actual one (13.4 mm) by almost half. In order for the former value to agree with the latter, the erosion distance in term IV must be much increased. This implies that the cliff should have been subject to a further reduction in rock strength related to weathering about one year from the end of term III prior to the storm waves (7.2 m in height) on 15 July 2007. The question arises: could such a weathering progress really be possible in the short period of time?

It can therefore be concluded from this modelling that weathering rather than waves plays an important role in controlling platform development. This is understandable because the landward cliff of type B platforms is situated in the supratidal zone, the most severe salt-weathering environment (Sunamura & Aoki 2011). In this zone rapid weathering processes are always operating and the weathered surface layer is ready for removal. However, (a) the depth change in strength of the weathering profile, and (b) the time required for weathering profile to form after the removal of the surface layer are unknown. Elucidation of these points and their quantification are crucial for furthering morphodynamic studies of shore platforms.

The above discussion is based on the premise that the wave factor could be appropriately incorporated in the model. However, the following must be critically examined in the future: (a) the assumption of \( H_c = H_b \), (b) the determination of \( (H^\star)^{\text{crit.}} = 0.5 \) m, and (c) the effect of wave setup on the water level at the landward cliff base. Re-examination of the \( K \)-value is also needed using data of erosion occurring in a very short term, for example, before and after a single storm event.

**Conclusions**

Through reanalysis of the existing laboratory data for cliff erosion we obtained a fundamental relation (equation (7)), which is useful for describing rocky coast evolution. A developmental model for type B platforms dominant in Japan was constructed based on this equation. The process of wave attenuation on this type of platform and the effect of weathering on the strength reduction of rocks were considered. The model was applied to two Japanese Pacific coasts in a high-wave-energy, microtidal environment. On the southwestern Kii Peninsula coast, long-term development rates of type B platforms were examined using the previous data of platform width and rock strength: the result indicated that the rate of platform initiation (6000 years BP) is two orders of magnitude greater than that of the present development (Fig. 12.14). At the Ebisu-jima platform, short-term development processes were explored using wave records and MEM measurements on the landward cliff. Although the model enabled us to describe the temporal variation in platform development, the model calculation and the actual result were not in satisfactory agreement (Fig. 12.19). This is probably because the model could not appropriately include the influence of subaerial weathering on the resistivity of the superficial part of cliff-forming rocks.

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