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A model for grain-crushing-induced landslides—Application to Nikawa, Kobe 1995

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Abstract

A novel method is developed, motivated by one of the mysteries of the Hyogoken-Nambu (Kobe): the Nikawa landslide. To this end, prompted by the hypothesis of "sliding-surface liquefaction" advocated by Sassa [Development of a new cyclic loading ring shear apparatus to study earthquake-induced-landslides. Report for grant-in-aid for development of scientific research by the Ministry on Education, Science and Culture, Japan (project no. 03556021), 1994, p. 1–106; Keynote lecture: access to the dynamics of landslides during earthquakes by a new cyclic loading high-speed ring shear apparatus. In: Proceedings of the sixth international symposium on landslides, 1992. In: Landslides, vol. 3. Balkema: Rotterdam; 1995. p. 1919–39], a dynamic analysis of the early stages of an earth slide is presented considering two mechanically coupled sub-structures: (a) the rapidly deforming shear band at the base of the slide and (b) the accelerating sliding mass modeled as a rigid body. The proposed model for sliding is based on: (i) the concept of high pore-water pressure generation by grain crushing along the sliding surface (proposed by Sassa et al. in 1995), (ii) an experimental model developed by Hardin [Crushing of soil particles. J Geotech Eng 1985;111(10):1177–92] for crushing of soil particles under compression and shear, expressed with a set of developed equations governing the mechanism of breakage, and (iii) the hysteretic stress–strain Bouc–Wen-type constitutive model coupled with the Coulomb friction law. An attempt is made to adjust the model parameters to Sassa's experimental data in ring-shear tests. The method leads to a reasonable prediction of the large displacement of the Nikawa landslide. A sensitivity analysis is also carried out for the influence of key model parameters (e.g. shape, crushing hardness, void ratio, grain size distribution, effective normal stress) on the pore-pressure rise due to particle breakage.

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Keywords: Nikawa landslide; Particle breakage; Grain crushing; Excess pore-water pressure; Seismic triggering; Landslide evolution

1. Introduction

The 1995 M_w7 Hyogoken-Nambu earthquake was one of the few major earthquakes to directly hit a sophisticated modern city possessing an extremely high concentration of civil engineering facilities. It resulted in the worst earthquake-related disaster in Japan since the 1823 M_s8 Kanto earthquake. The port of the Kobe City, of critical importance to the Japanese economy, was left almost completely out of service, while very significant was the damage to the elevated highways which carried the traffic through the city. (See the numerous detailed reports by the Japanese Geotechnical Society [1], Committee of Earthquake Engineering [2] and Earthquake Engineering Research Institute [3].

Through all this tremendous devastation on all types of engineered structures, the nearly 400 landslides that also took place did not catch the attention of the casual observer [4]. Most of them were of relatively small size, often associated with tensile cracking, and of limited motion—not unexpectedly in view of the fact that the earthquake occurred during the "dry" season. A conspicuous exception was the Nikawa rapid landslide—one of the most devastating landslides directly related to an earthquake. With a landslide volume in the order of 110,000 m³ [4], moving in just a few seconds over a distance of more than 100 m, it destroyed 11 residential buildings causing 34 fatalities. Fig. 1a shows the plan of the slope before the Nikawa landslide and the outline of the landslide area [4].

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Nomenclature		r_u	pore-water pressure ratio
		S	parameter defined by Eq. (6)
b	parameter that controls the stress-displacement	$S_{ m t}$	parameter defined by Eq. (9)
	unloading-reloading behavior	S_w	resultant seepage force
b_{p0}	potential for particle breakage, of given size	t	time
	fraction, before loading	и	displacement
$b_{\rm pl}$	potential for particle breakage, of given size	ü	velocity
	fraction, after loading	ü	acceleration
$B_{\rm p}$	breakage potential at current time	\ddot{u}_g	seismic acceleration imposed at the base of the
$B_{\rm p0}$	initial (before loading) breakage potential	-	rigid block
$\dot{B_{\rm pl}}$	final (after loading) breakage potential	u_{v}	parameter accounting for the elasto-plastic
$\dot{B_{t0}}$	total breakage index	-	displacement tolerance
B_D	the pore-pressure coefficient after consolidation	x	co-ordinate parallel to the shear band axis
c_v	consolidation coefficient	Ζ	co-ordinate normal to the shear band axis
d_{b}	shear band thickness	ζ	hysteretic parameter that controls the nonlinear
df	differential of "percent passing" divided by 100		stress-displacement relationship
D	particle diameter	θ	inclination angle
e_0	initial void ratio	λ	pore-pressure-breakage coefficient
g	acceleration of gravity	μ	Coulomb friction coefficient
h	crushing hardness	μ_{a}	apparent friction coefficient
т	mass of the sliding block	ξ	breakage coefficient
п	parameter that controls the sharpness of the	$\sigma'_{ m oct}$	effective octahedral normal stress
	transition from the linear to the nonlinear	$\sigma'_{ m n}$	effective normal stress
	stress-displacement range	$\sigma'_{ m n0}$	initial effective normal stress
n _b	breakage number	τ	shear stress
n _s	shape number	$\tau_{\rm oct}$	octahedral shear stress
OCR	overconsolidation stress ratio	τ_y	ultimate shear strength
р	excess pore-water pressure	$\varphi_{\rm a}$	apparent friction angle
$p_{\rm a}$	atmospheric pressure	arphi'	effective friction angle

A cross-section of the landslide is depicted in Fig. 1b. In addition, of course, to strong seismic shaking, perhaps accentuated by topographic amplification [5], several deeper causes, such as "sliding-surface liquefaction" [4] and water-"film" generation [6], have been proposed to explain the rapid runoff of the slide. The goal of this paper is to develop and apply a new model for grain-crushing-induced liquefaction—extending the concept of Sassa [7,8] and utilizing available experimental data [9] to explain the rapid runoff of the Nikawa landslide.

2. The surprise of the Nikawa landslide: possible causes or contributing factors

The earthquake took place during a dry season, which followed the historically dry 1994 summer. The limited amount of rainfall possibly played a major role in reducing the number of landslides triggered by the earthquake. The most important landslides (such as Nikawa) were associated with the so-called Osaka formation layer, that consisted of limnic and marine deposits of sands and clays from Pliocene to Middle Pleistocene, of low permeability [4,9]. Within these low permeability layers, pore water could have been preserved despite the dry season. Therefore, while most landslides originated within unsaturated soil, and hence were of moderate magnitude, this was not the case with Nikawa. Engineers were surprised with the significant distance and speed of the runoff because, as reported in Sassa et al. [4]: (a) the slope inclination barely exceeded 20° , (b) the water table was not high (although there was evidence that it was above the sliding surface for a significant length), (c) the soil along the sliding surface consisted of rather dense coarse-grained sand to silty sand, a material not readily susceptible to liquefaction, and (d) laboratory tests on soil samples taken from the bottom of the moved landslide mass showed that the soil was only partially saturated, eliminating the low potential for mass liquefaction.

As depicted in the cross-section of Fig. 1b, the slope of the landslide mass did not exceed 20°. The number of blows of the standard penetration test, $N_{\rm SPT}$, ranged from 10 (near the surface) to 60 (refusal, in the Japanese scale). Secondary sedimentary layers and terrace layers were found to overlie the Osaka formation (granitic sand and clay). The bedrock granite was detected at 25–35 m depth [4]. Cyclic loading tests, conducted by Sassa et al. [9], showed that the soil layers along the sliding surface possessed a liquefaction potential. As reported in Sassa et al. [4,9], the apparent friction angle was measured to be in the order of $\varphi_a \approx 8.5^\circ$ (compared to the effective friction



Fig. 1. The Nikawa landslide: (a) plan view and (b) cross-section [4].

angle $\varphi' \approx 30.0^{\circ}$). Several witnesses asserted that water was flowing from the base of the landslide the very next day.

The recorded peak ground acceleration (PGA) reached 0.60 g at level ground very close to Nikawa. The very small distance from the North Eastern (NE) part of the causative fault can explain such a large PGA. The combination of soil and topographic amplification could have played a major role, at least in triggering the landslide. In such a case, the basic free-field motion could have possibly been amplified within the sliding mass. With such an excitation, the developing shear stresses could possibly lead to liquefaction of even marginally sensitive soil layers. However, even with such a strong triggering, the 140 m displacement could be explained.

The extent of the runoff together with its rapid nature, which left no time for response, led Sassa [7,8] in developing a hypothesis which he called "*sliding-surface liquefaction*". In the conventional (mass) liquefaction, the

strength loss is associated with pore-pressure buildup due to the tendency of the soil to contract when subjected to shearing. It is caused by destruction of the meta-stable fabric of loose saturated soil, and grain crushing is not necessary. Sliding-surface liquefaction is quite different: when the soil is subjected to shearing, and after a sliding surface has been developed, the crushing of sand grains and the consequent increase in volume of solids is the mechanism of pore-pressure buildup, leading to a different type of "liquefaction". Sassa et al. [9] utilized a high-speed ring-shear apparatus to test soil specimens from Nikawa, with shearing speeds in the order of 0.3 m/s. While in mass liquefaction the pore-pressure buildup is rapid, these tests showed a gradual increase of pore pressure and a subsequent drop of the apparent friction angle to about 8.5°, without any sign of liquefaction in the sample. The grain crushing became evident from the grain size distribution along the shear zone.

Some other alternative mechanisms can be invoked to explain the phenomenon. For instance, the mechanism of gradual "smoothing" of the sliding surface was proposed by Kokusho [6] (not for the Nikawa landslide). He supposed that when a soil layer of significant thickness underneath the sliding surface liquefies, and the soil directly on top is of low permeability (both conditions might also apply to the Nikawa landslide), then the natural tendency of the liquefied layer to settle could produce a very slim "film" of water, only a few centimeters or even millimeters in thickness. The development of this water film along the sliding surface could explain the extent of the runoff (about 100 m). However, the nature of the soils in Nikawa does not support such a theory.

Another mechanism that could possibly explain the rapid evolution of Nikawa landslide, and is well documented in the literature, is the strain-rate friction-softening behavior of a saturated clay-rich sliding surface. The motion of the slide could be further accelerated when the soil is susceptible to thermo-poro-mechanical softening. That is, shearing gives rise to heat-generated excess porewater pressure which in turn causes further frictional softening [10–14]. This theory has been successfully applied to the analysis of catastrophic landslides, among others the Vaiont in 1963 [15] and the Jiufengershan triggered by the Chi-Chi Taiwan (1999) earthquake [16]. However, it is not applicable to the Nikawa landslide as the clay fraction of the soil in the sliding surface was very small to justify such a theory.

3. Models for grain crushing: discussion

The relationship between the behavior of sand and the breakage of particles have been investigated by many researchers. A number of models of varying degrees of accuracy, efficiency, and sophistication have been developed. The models could be classified into three categories: (a) semi-empirical expressions including indices for particle breakage (e.g. [17–23]). Most of these were based on the difference in grain size distribution curves before and after loading. (b) Critical state models in which the size of the yield surface is related to the amount of particle breakage [23,24], (c) The 1-D compression models, based on the theory of fractals [25,26].

Hardin [21] exploiting the results of a series of experimental tests, showed that the strength, compressibility, and stress-strain behavior of a soil element are strongly affected by the amount of grain crushing during loading and deformation. Moreover, he related particle crushability to fundamental soil parameters such as the grain size distribution, void ratio, water content, shape, and hardness of the individual particle.

McDowell et al. [25] developed a model for crushable aggregates using a statistical function and work equation. Studying the relationship between the statistical parameter and the curvature of the normal compressional line, $e - \log \sigma'_n$, they indicated that the compressional behavior

of sand due to grain crushing could be expressed using a probabilistic approach.

McDowell and Bolton [26] studied the micromechanical behavior of crushable soils. They developed a fractal theory of particle crushing, based on the assumption that the smallest particles are in geometrically self-similar configurations under increasing macroscopic stress. The theory was then used to relate the evolution of particle sizes to the normal compression curve, in terms of fundamental particle parameters.

Nakata et al. [23], utilizing experimental results, related the individual particle crushing to the particle strength variability, applying a Weibull function. Moreover, they established a relationship between single-particle crushing properties and a particle breakage factor. They subsequently developed a Cam Clay model in which the critical state stress is expressed as a function of this factor. According to this theory, the evolution of the grain size distribution curve is related to the size of the yield surface, with larger yield surfaces causing more particle breakage. As a continuation of this work, Nakata et al. [24] investigated the relationship between the 1-D compression curve and typical failure patterns of individual particles. Furthermore, they studied the influence of various soil parameters such as the uniformity coefficient, the initial grain size distribution, and the void ratio on the compressional characteristics of the soil. They showed that the amount of grain crushing under isotropic loading conditions is much lower than under shearing.

Luzzani and Coop [27] studied the relationship between volume change and particle breakage during shearing of sand in ring-shear and direct shear apparatus. Among their findings are that (a) the breakage caused by shearing is much larger than the one caused by compression. They showed cases in which the onset of grain crushing occurred at effective pressures as low as 0.05–0.1 Mpa. (b) In shearing, an extremely high shear strain is required for the critical state of the soil to be reached (when the particle breakage ceases), which is very difficult to achieve even with the ring-shear apparatus.

Despite their efficiency, the aforementioned models for grain crushing cannot be directly applied to analyze earthquake-induced rapid landslides caused by slidingsurface liquefaction. They do not take into account the influence of grain crushing on the excess pore-water pressure generation, which controls the evolution of the landslide.

Sassa [7,8] developed the theory of sliding-surface liquefaction to explain the rapid evolution of earthquakeinduced landslides. The theory was experimentally supported by undrained loading ring-shear tests [4,28]. It was shown that grain-crushing-induced liquefaction may occur not only in fully water-saturated soils [29], as is usually the case for mass liquefaction, but also in partially saturated soils [4]. The theory was successfully used to describe the evolution of rapidly moved landslides, in the Hyogoken-Nambu 1995 earthquake: the Nikawa and Takarazuka landslides [4], and in the Mid-Niigata Prefecture 2004 earthquake: the Higashi Takezawa and Terano landslides [29].

The goals of this paper are: (a) to develop a model for grain-crushing-induced liquefaction under shearing that overcomes the aforementioned limitation, and (b) to apply the model in analysing the triggering and evolution of the Nikawa landslide.

4. Grain-crushing-induced liquefaction: equations and parameters

4.1. Problem formulation

We consider a deforming infinitely long shear band of thickness d_b consisting of fully water-saturated grain particles (Fig. 2). The field variables are the excess porewater pressure p, the breakage potential B_p , and the relative displacement u between top and bottom. The porewater pressure is assumed to be a function of time, t, and of position, z, whereas the rate of particle breakage is only a function of time. The displacement is considered to vary linearly with position from 0, at the bottom of the band, to the maximum value *u* at the top of the band—a reasonable (but not compulsory) approximation; u is also considered a function solely of time. The breakage potential B_p is a measure of the evolution of the particle size distribution curve with loading, as defined in the sequel. It is pointed out that the parameter B_p is the current value of the breakage potential, and should not be confused with that originally defined by Hardin [21], denoted in the next section as B_{p0} . The latter, B_{p0} , is the initial (i.e. before loading) breakage potential and is a constant.

The sliding process of the shear band can be divided into the following three consecutive stages:

- (a) Shearing causes particle crushing (see the experiments of Sassa).
- (b) Particle crushing causes contraction of the soil resulting in pore-pressure generation, which in turn decreases the effective normal stress and causes frictional softening.



Fig. 2. Schematic illustration of the developed shear band model.

- (c) Frictional softening is continued to occur until
 - the effective normal stress is decreased to a certain value under which grain crushing does not further take place, or
 - the excess pore-water pressure reaches the limiting value of the initial effective normal stress, or
 - the sizes of all the particles have been reduced to sizes equal to, or smaller than that of the maximum silt size ("powder").

Notice that pore-water pressure generation due to mass liquefaction (caused by destruction of the meta-stable soil structure in a mass of saturated loose soil) is not considered.

4.2. A brief introduction to Hardin's model for particle crushing

The concept of the pore-water pressure mechanism proposed herein has been inspired by the work of Hardin [21] on the crushing of soil particles. Hardin, proposed the index B_{p0} , termed initial breakage potential, to measure the potential amount of particle breakage. This is defined as

$$B_{\rm p0} = \int_0^1 b_{\rm p0} \,\mathrm{d}f,\tag{1}$$

where b_{p0} represents the initial (before loading) potential for breakage that is significant to soil behavior for a given size fraction, df, in an element of soil; df is a differential of "percent passing" divided by 100. b_{p0} is defined as

$$b_{\rm p0} = \begin{cases} \log_{10} \left(\frac{D \text{ in mm}}{0.074 \text{ mm}} \right) & D \ge 0.074 \text{ mm} \\ 0 & D < 0.074 \text{ mm}, \end{cases}$$
(2)

in which *D* is the particle size (diameter). The value D = 0.074 mm is the upper limit of the silt size. Since breakage of silt and clay size particles is less important to soil behavior than for larger sizes, it is completely ignored. As shown in Fig. 3, B_{p0} is equal to the area between the line defining the upper limit of the silt size and the part of the particle size distribution curve for D > 0.074 m. Typical values of B_{p0} are 0–0.5 for fine sand (SC), 1.5–2 for fine gravel (GF), 2–2.5 for medium gravel (GM), and 2.5–3 for coarse gravel (GC). The amount of crushing that is significant to soil behavior may be expressed as

$$B_{\rm t0} = \int_0^1 (b_{\rm p0} - b_{\rm pl}) \,\mathrm{d}f \tag{3}$$

in which b_{pl} is the potential for breakage after loading. The final breakage potential is then defined as

$$B_{\rm pl} = \int_0^1 b_{\rm pl} \,\mathrm{d}f \tag{4}$$

Hardin [21], utilizing results from laboratory tests, proposed a semi-empirical relationship for the total



Fig. 3. Definitions by Hardin [21] of initial breakage potential B_{p0} and total breakage B_{t0} .

breakage index B_{t0} (Fig. 3) of a given soil in terms of six variables describing (a) the effect of particle size distribution, (b) the state of effective stress, (c) the effective stress path, (d) the initial void ratio, (e) the particle shape, and (f) the particle hardness. B_{t0} was thus expressed as

$$B_{\rm t0} = \frac{B_{\rm p0} S^{n_{\rm b}}}{1 + S^{n_{\rm b}}} \tag{5}$$

in which S is given by

$$S = \frac{1 + e_0}{800h^2} \frac{\sigma'_{\text{oct}}}{p_a} \left[1 + 9 \left(\frac{\tau_{\text{oct}}}{\sigma'_{\text{oct}}} \right)^3 \right],\tag{6}$$

where p_a is the atmospheric pressure ($p_a = 100 \text{ kPa}$); e_0 is the initial void ratio; h is the crushing hardness, approximately equal to Moh's scratch hardness; σ'_{oct} and τ_{oct} are the effective octahedral normal and shear stress, respectively; and n_b is the breakage number expressed as a function of the crushing hardness h, the void ratio e_0 , and the shape number n_s according to

$$n_{\rm b} = \frac{h^2}{(1+e_0)n_{\rm s}} + 0.3. \tag{7}$$

Eqs. (1), (3)-(5) yield

$$B_{\rm pl} = \frac{B_{\rm p0}}{1 + S^{n_{\rm b}}}.$$
(8)

In the undrained cyclic simple shear test, taking into account grain crushing only due to shearing, Eq. (6) alters to

$$S_{t} = 9 \frac{1 + e_{0}}{800h^{2}} \frac{\sigma'_{n}}{p_{a}} \left(\frac{|\tau|}{\sigma'_{n}}\right)^{3}$$
(9)

in which τ and σ'_n are the shear and effective normal stresses, respectively.

The shape number n_s depends on the particle morphology. Hardin [21], proposed the following values for n_s according to the shape of a particle: angular, 25; subangular, 20; subround, 17; round, 15. The crushing hardness *h* depends mainly on the particle fabric and mineralogy as well as the presence of water. It ranges between 1 and 10, with the smaller values corresponding to weaker particles. As an example, a value of h = 8.5 has been reported from Hardin [21] for aluminum oxide sand, and h = 2.8 for Tokyo SF and GF. Water greatly increases the crushability or decreases the particle hardness. Experimental results from tests on Antioch sand [21] showed that the crushing hardness of the saturated samples can be as low as 50% of that of the unsaturated samples.

However, the experimental results were not enough to establish a clear relationship between the crushing hardness and water saturation. A possible explanation why the crushing hardness decreases with increasing water

а

1

0.8

0.6

saturation is as follows: all particles contain flaws in the form of microfissures and cracks. When a particle is crushed, the initial fracture is likely to originate in a preexisting flaw [26]. It is thus straightforward that the strength of a particle is strongly related to the shear resistance of its fissures. When water infiltrates into these fissures, their microtopography is being eroded resulting in smoothening of their roughness, which in turn reduces the shear resistance of the fissure and thus the crushing resistance of the particle.

4.3. Stress-displacement relationship

A versatile 1-D constitutive model is utilized to describe the shear stress-strain relationship inside the shear band. The model is capable of reproducing an almost endless variety of stress-strain forms, monotonic as well as cyclic. Based on the original proposal by Bouc [39] and Wen [30], the model was recently extended by Gerolymos and Gazetas [31] and applied to cyclic response of soils. It is used herein in conjunction with a Mohr-Coulomb friction law and Terzaghi's effective stress principle.

The stress-displacement relationship of the soil inside the shear band is given by

$$\tau = \tau_{\nu}\zeta,\tag{10}$$

where τ_y is the ultimate shear strength which is a function of the effective normal stress. The parameter $\zeta = \zeta(u)$ is a hysteretic dimensionless quantity, controlling the nonlinear response of the soil. It is governed by the following differential equation:

$$\frac{d\zeta}{du} = \frac{1}{u_y} \{ 1 - |\zeta|^n [b + (1 - b) \operatorname{sgn}(\dot{u}\zeta) \}$$
(11)

in which \dot{u} is the lateral velocity, and u_v is a parameter accounting for the elasto-plastic slip tolerance.

In the above equations, u_v is a parameter signaling yielding in the soil. It is defined as the ratio of the peak shear strength τ_{v} to the initial (at very small strains) shear modulus G_0 of the soil, multiplied by the shear band thickness $d_{\rm b}$. In the extreme case of a soil obeying an elastic-rigidly plastic stress-displacement relation, u_{y} would be exactly the displacement at which the peak shear strength is reached. As the shear band thickness is considered to be zero in our problem, u_v can be calculated alternatively from a direct or from a ring-shear test. Representative values of u_v for typical sand specimens are of the order of 1 mm, with smaller ones corresponding to either large relative densities or small effective normal stresses. The parameter n governs the sharpness of the transition from the linear to the nonlinear range, during initial virgin loading. It ranges from 0 to ∞ , with elastic-perfectly plastic behavior practically achieved when n takes values greater than 10. Values of nbetween 0.6 and 1 have been found to better fit most experimental results for cohesionless soils [31]. Monotonic loading curves for different values of n are presented in



0.25

n=0.

(10)-(11): (a) normalized shear stress versus normalized displacement curves to monotonic loading for selected values of parameter n and (b) hysteretic normalized shear stress versus normalized displacement loops for different values of b and n = 1. The Masing criterion for unloading-reloading is obtained for b = 0.5.

Fig. 4a. Parameter b controls the shape of the unloading-reloading curve. Its range of values is between 0 and 1. When b = 0.5 the reversal stiffness equals the initial stiffness and the Masing criterion for loading-unloadingreloading is recovered. The influence of parameter b on the hysteretic shear stress-displacement loop is shown in Fig. 4b. For details on the derivation of Eq. (11) and the calibration of the associated parameters, the reader is referred to the recent publication of Gerolymos and Gazetas [31].

Note that Eq. (11) has also been applied to the related problem of seismic triggering and evolution of catastrophic landslides in clayey soil [14].

The parameter τ_v is expressed as

$$\tau_y = \mu \sigma'_{\rm n}.\tag{12}$$

The friction coefficient, μ , is expressed in terms of the Coulomb friction angle φ' of the soil in direct shear:

$$\mu = \tan \varphi' \tag{13}$$

and

$$\sigma'_{\rm n} = \sigma'_{\rm n0} - p, \tag{14}$$

where σ'_{n0} is the initial effective normal stress and p is the excess pore-water pressure, generated due to particle breakage. Substituting Eqs. (10) and (12) into Eq. (9) yields

$$S_{t} = 9 \frac{1 + e_{0}}{800h^{2}} \frac{\sigma'_{n}}{p_{a}} (\mu|\zeta|)^{3}.$$
 (15)

4.4. Equations for pore-water pressure generation due to particle breakage

The mechanism of pore-water pressure generation due to particle breakage is assumed to be governed by the following set of equations:

$$\frac{\partial p}{\partial t} = \frac{\partial}{\partial z} \left(c_v(B_{\rm p}) \frac{\partial p}{\partial z} \right) - \lambda \frac{\partial B_{\rm p}}{\partial t} \sigma'_{\rm n0},\tag{16}$$

$$\frac{\mathrm{d}B_{\mathrm{p}}}{\mathrm{d}t} = \xi(B_{\mathrm{pl}} - B_{\mathrm{p}}),\tag{17}$$

in which B_p is the breakage potential and B_{pl} is the final (after loading) breakage potential as computed at the current time of loading, c_v , λ , and ξ are the coefficients of consolidation, pore-pressure-breakage, and breakage, respectively. Note that c_v is a function of B_p . In fact, c_v decreases with decreasing particle size and thus with particle crushing evolution [32]. Also note that B_{pl} in Eq. (17) is a function of the excess pore-water pressure. The physical meaning of these coefficients will be elucidated later.

The first and second terms in the right-hand side of Eq. (16) govern the excess pore-water pressure generation and diffusion mechanisms, respectively, derived from mass balance consideration and Darcy's law. According to the second term, the excess pore-water pressure buildup is proportional to the rate of breakage potential (that is, proportional to the rate of the amount of grain crushing). When $\lambda = 0$, Terzaghi's [33] standard consolidation equation is derived. Eq. (17) is based on Hardin's model (presented previously), after applying a heuristic process which satisfies the following initial and asymptotic conditions: (a) the initial value of B_p is equal to B_{p0} ; (b) the asymptotic value (when time tends to infinity) of $B_{\rm p}$ is equal to the initial value of B_{pl} , when excess pore-water pressure is not developed, and smaller than that when the excess pore-water pressure is greater than zero; (c) the rate of $B_{\rm p}$ approaches zero as time tends to infinity, meaning that the particle crushing process has been terminated. It is evident from Eq. (16) that the breakage potential $B_{\rm p}$ has two competing effects on pore-water pressure development. The larger the size of the particles (and thus the larger the value of $B_{\rm p}$), the higher the crushability potential of the soil and thus the pore-water pressure generation. On the other hand, the larger the value of $B_{\rm p}$, the higher the soil permeability and thereby the pore-water pressure diffusion. In the limit of undrained loading conditions, which is a reasonable assumption when the shear band is deformed at a large velocity (rapid landslide), and when the breakage potential B_p is not extremely high, Eq. (16) reduces to

$$\frac{\mathrm{d}p}{\mathrm{d}t} = -\lambda \frac{\mathrm{d}B_{\mathrm{p}}}{\mathrm{d}t} \sigma_{\mathrm{n}0}^{\prime}.$$
(18)

Eq. (18) instead of (16) will be used in all subsequent analyses.

The system of Eqs. (17) and (18) cannot be solved analytically because B_{pl} is a highly nonlinear function of the effective normal stress σ'_n , which in turn depends on the excess pore-water pressure p. Yet an analytical solution, even if crudely approximate, might shed light on the mechanism of grain-crushing-induced pore-water pressure. Thus, only as a first illustrative example, we assume that B_{pl} is insensitive to variations of σ'_n . Under this assumption (in the subsequent development a numerical solution is developed which does not make use of this over-simplifying assumption), the system of Eqs. (17) and (18) is solved analytically, yielding the following expressions:

$$B_{\rm p} = B_{\rm pl} + B_{\rm t0} \exp(-\xi t) \tag{19}$$

and

$$p = \lambda \sigma'_{n0} B_{t0} [1 - \exp(-\xi t)].$$
⁽²⁰⁾

It is evident from Eq. (20) that the parameter ξ controls the rate of transition from zero to the maximum excess porewater pressure. Decreasing values of ξ leads to smoother rates of transition. On the other hand, the parameter λ controls the ultimate value of the pore-water pressure. The larger the value of λ , the higher the maximum excess porewater pressure. The role of λ and ξ on the evolution of the pore-water pressure ratio is illustrated in Figs. 5 and 6. The pore-water pressure ratio is defined as the ratio of the excess pore-water pressure to the initial effective normal stress, $r_u = p/\sigma'_{n0}$. It is noted, that the curves plotted in these figures have been produced from the system of differential equations (10), (11), (17), and (18), and not from that of the simplified equations (18) and (19).



Fig. 5. Pore-water pressure ratio r_u versus displacement curve, for selected values of pore-water pressure-breakage coefficient λ , and for constant breakage coefficient $\xi = 0.2$. The values of the other model parameters are $\sigma'_{n0} = 0.3$ MPa, $e_0 = 0.55$, $n_s = 25$, h = 2.4, and $\mu = 0.52$. The rate of shearing is constant and equal to 0.3 m/s.



Fig. 6. Pore-water pressure ratio r_u versus displacement curve, for selected values of breakage coefficient ξ , and for constant pore-water pressure-breakage coefficient $\lambda = 8$. The values of the other model parameters are $\sigma'_{n0} = 0.3$ MPa, $e_0 = 0.55$, $n_s = 25$, h = 2.4, and $\mu = 0.52$. The rate of shearing is constant and equal to 0.3 m/s.

The effect of other parameters on the pore-water pressure buildup, which are not explicitly taken into consideration in the proposed model, can be partially reflected through the calibration of λ and ξ . For example, two sands with the same initial breakage potential B_{p0} but with different values of the coefficient of uniformity, do not have the same susceptibility to crushing. In fact, the most uniformly graded sand will exhibit the most severe grain splitting. A second example is the degree of water saturation which also affects the mechanism of grain-crushing-induced pore-water pressure. The proposed model is capable of representing the aforementioned differences in soil behavior provided that λ and ξ are appropriately calibrated, although the coefficient of uniformity and the degree of saturation are not model parameters.

4.5. Calibration of the pore-pressure-breakage λ , and breakage ξ coefficients, from ring-shear test

The capability of the system of equations (10), (11), (17), and (18) to lead to a satisfactory prediction of the graincrushing-induced pore-water pressure hinges on the proper calibration of the parameters λ and ξ . Such a calibration can be accomplished by fitting the results of a high-speed ring-shear test. A demonstration is given here by utilizing the results of tests conducted by Sassa [8] on soil specimens taken from the zone of slippage of the Nikawa landslide. The calibrated values of λ and ξ are subsequently used in the numerical simulation of this rapid landslide.

Sassa et al. [4] reported that the soil at the interface between the displaced landslide mass and the original ground was blue granitic sand with clay. A typical grain size distribution of this soil is plotted in Fig. 7. Sassa et al. [9] conducted undrained cyclic ring-shear tests on two soil samples, simulating the real conditions, to investigate the residual shear resistance mobilized along the sliding surface, at large shear displacement and velocity.



Fig. 7. Grain size distribution for blue granitic sand found at the bottom of the Nikawa landslide mass [4]. The initial potential for breakage B_{p0} is equal to 0.72.

The test procedure was as follows: the first sample was preconsolidated to an OCR = 1.9 and to an initial effective normal stress $\sigma'_{n0} = 0.232$ MPa, and then subjected to an initial shear stress of $\tau_0 = 0.089$ MPa, under drained conditions, followed by a cyclic shear stress loading of maximum velocity v = 0.3 m/s, under undrained conditions. The effective friction angle was reported to be $\varphi' =$ 31.0° and the maximum pore-water pressure ratio $\max(r_u) = \max(p/\sigma'_{n0}) = 0.72$ developed at the end of loading, after reaching a displacement of 28.4 m. The corresponding reported values for the second sample (OCR = 1, $\sigma'_{n0} = 0.298$, $\tau_0 = 0.082$ MPa) were $\varphi' = 27.2^\circ$, $\max(r_u) = 0.70$, and $\max(u) = 24.3$ m. The pore-pressure coefficient after consolidation was measured to be $B_D =$ 0.35 for both soil samples, indicating that the soil was only partially saturated.

The following steps shall be carried out in order to calibrate the model parameters:

- (i) Calculate the initial breakage potential from the grain size distribution curve. A value of $B_{p0} = 0.72$ is estimated from Fig. 7.
- (ii) Estimate the shape number n_s and the crushing hardness h [21]. In this example, $n_s = 25$ and h = 2.4.
- (iii) Calibrate the parameters λ and ξ to match the experimental pore-water pressure ratio versus time curve. As partial saturation is not directly treated by the model, the effect of the degree of saturation on the excess pore-water pressure evolution is partially reflected through the calibration of λ and ξ .

Applying this calibration methodology, the experimental data of Sassa et al. [9] are reproduced in Fig. 8 with $\lambda = 8.8$ and $\xi = 0.2$.

5. Parametric analysis

To investigate the influence of key model parameters on the evolution of the pore-water pressure induced by grain



Fig. 8. Pore-water pressure ratio r_u versus displacement. Comparison between measured in ring-shear apparatus (circles and triangles, [4]) and computed behavior (solid lines: gray line—OCR = 1.0, black line—OCR = 1.9), for $\lambda = 8.8$, $\xi = 0.2$, $\sigma'_{n0} = 0.3$ MPa $e_0 = 0.55$, $n_s = 25$, h = 1.2, $\mu = 0.52$, $B_{p0} = 0.72$. The test was performed at a constant velocity of 0.3 m/s. The triangles correspond to the test with OCR = 1.9, while the circles to the test with OCR = 1.0.



Fig. 9. Evolution of the pore-water pressure ratio r_u and breakage potential B_p with displacement, for selected values of initial breakage potential B_{p0} , and for coefficients $\lambda = 8.8$ and $\xi = 0.2$. The values of the other model parameters are $\sigma'_{n0} = 0.3$ MPa, $e_0 = 0.55$, $n_s = 25$, h = 2.4, and $\mu = 0.52$. The rate of shearing is constant and equal to 0.3 m/s.

crushing, Eqs. (17) and (18) together with Eqs. (10) and (11) were solved numerically. The initial value of the shear stress is assumed to be zero. The rate of shearing is constant and equal to 0.3 m/s. The values of parameters λ



Fig. 10. Evolution of the pore-water pressure ratio r_u and breakage potential B_p with displacement, for selected values of crushing hardness h, and for coefficients $\lambda = 8.8$ and $\xi = 0.2$. The values of the other model parameters are $\sigma'_{n0} = 0.3$ MPa, $e_0 = 0.55$, $n_s = 25$, $B_{p0} = 0.72$, $\mu = 0.52$. The rate of shearing is constant and equal to 0.3 m/s.

and ξ are the same in each analysis and equal to those calculated from fitting the experimental data of Sassa et al. [9]. The results of the analysis are plotted in Figs. 9–13 in the form of pore-water pressure ratio r_u and breakage potential B_p evolution. The following remarks are worthy of note:

5.1. Initial breakage potential B_{p0}

The pore-water pressure increases with increasing values of $B_{\rm p0}$ (Fig. 9), meaning that the larger the particle size the higher the amount of crushing and thus the pore-pressure rise. Recall, however, that the phenomenon is more complicated when the antagonistic mechanism of pore-pressure diffusion is considered. Note also that a steady-state condition (when pore-water pressure remains constant) is reached more rapidly as $B_{\rm p0}$ increases.

5.2. Crushing hardness h

The pore-water pressure decreases with increasing values of h (Fig. 10). Larger values of h correspond to strong particle fabric with minor preexisting flaws. The greater the



Fig. 11. Evolution of the pore-water pressure ratio r_u and breakage potential B_p with displacement, for selected values of initial void ratio e_0 , and for coefficients $\lambda = 8.8$ and $\xi = 0.2$. The values of the other model parameters are $\sigma'_{n0} = 0.3$ MPa, $B_{p0} = 0.72$, $n_s = 25$, h = 2.4, and $\mu = 0.52$. The rate of shearing is constant and equal to 0.3 m/s.

crushing hardness, the more difficult for the particle to be fractured and thus to produce pore pressures. The steady-state condition is reached more rapidly as h decreases.

5.3. Initial void ratio e_0

The pore-water pressure increases only moderately with increasing initial void ratio e_0 (Fig. 11). A small value of void ratio suggests a high co-ordination number and thus a smooth stress field inside the particle, and a smaller likelihood of fracture [26] and pore-water pressure buildup. In other words, the crushability of the soil increases with increasing void ratio. It is reminded that the co-ordination number of a particle is the number of neighboring particles that actually contact it.

5.4. Shape number $n_{\rm s}$

The pore-water pressure increases with increasing values of n_s (Fig. 12). Large values of the shape number correspond to angular particles. A particle with high angularity suggests greater stress concentration at the edges of the particle, and a higher probability of fragmentation and thus pore-water pressure buildup.



Fig. 12. Evolution of the pore-water pressure ratio r_u and breakage potential B_p with displacement, for selected values of shape number n_s and for coefficients $\lambda = 8.8$ and $\xi = 0.2$. The values of the other model parameters are $\sigma'_{n0} = 0.3$ MPa, $e_0 = 0.55$, $B_{p0} = 0.72$, h = 2.4, and $\mu = 0.52$. The rate of shearing is constant and equal to 0.3 m/s.

5.5. Initial effective normal stress σ'_{n0}

The initial effective normal stress σ'_{n0} has a significant effect on pore-water pressure development (Fig. 13). As expected, large values of σ'_{n0} lead to high and rapidly developed pore-water pressures.

To investigate the influence of the pore-water pressure buildup on the breakage evolution, analysis is carried out considering drained ($\lambda = 0$) and undrained ($\lambda \neq 0$) shearing conditions, for selected values of the model parameters. The results are summarized in Fig. 14 in the form of evolution of the pore-water pressure ratio r_u , and breakage potential B_p , with displacement. As shown in this figure, the difference between the asymptotic values of B_p under drained and undrained conditions increases with increasing pore-water pressure. It is reminded that the asymptotic value of B_p in drained conditions is equal to the final breakage potential B_{pl} , as defined by Hardin [21].

6. A sliding block model for triggering and evolution of grain-crushing-induced landslide

To apply the proposed theory for grain-crushinginduced pore-water pressures in the analysis of the Nikawa



Fig. 13. Evolution of the pore-water pressure ratio r_u and breakage potential B_p with displacement, for selected values of initial effective normal stress σ'_{n0} , and for coefficients $\lambda = 8.8$ and $\xi = 0.2$. The values of the other parameters are $B_{p0} = 0.72$, $e_0 = 0.55$, $n_s = 25$, h = 2.4, and $\mu = 0.52$. The rate of shearing is constant and equal to 0.3 m/s.

landslide, a sliding block model is developed. This is clearly an approximation of the actual dynamics of the sliding (deformable) mass. We assume here that, at least in its early stage, the earth mass behaves like a rigid body sliding along an inclined plane. This simplified approximation is not an inherent limitation of the method, but rather a simple first choice of convenience. After all, the rigid-block analogy is still being used in soil dynamics studies for earth dam and embankments [34].

It is noted, however, that the topography of the sliding surface considerably influences the runout process of the landslide. Thus, a sliding block approach could yield reasonable results only when applied to the early stages of the landslide evolution. There are several methods available in the literature capable of reproducing the actual geometry of both the basal topography and of the moving soil mass. For example, Stamatopoulos et al. [35], and Sitar et al. [36] utilized multi-block approaches to back-analyze landslide case histories. Chen and Lee [37] developed a Lagrangian finite element method formulated along with a Bingham model for simulating landslides and slurry flows, and Gerolymos and Gazetas [38] developed a depthintegrated model to describe the seismic triggering, evolution, and deposition of massive landslides.



Fig. 14. Evolution of the pore-water pressure ratio r_u and breakage potential B_p with displacement, for selected values of the pore-water pressure-breakage coefficient λ , initial effective normal stress, and for coefficients $\lambda = 8.8$ MPa and $\xi = 0.2$. The values of the other parameters are $\xi = 0.2$, $B_{p0} = 0.72$, $e_0 = 0.8$, $n_s = 25$, h = 2.4, $\sigma'_{n0} = 1$ MPa, and $\mu = 0.52$. The rate of shearing is constant and equal to 0.3 m/s. The asymptotic value of B_p at drained conditions is equal to the value of B_{p1} as is defined by Hardin [21].

The motion of the rigid block is described by the differential equation

$$m(\ddot{u} + \ddot{u}_g) + mg(\mu_a\zeta\cos\theta - \sin\theta) - S_w = 0$$
(21)

in which *m* is the mass of the rigid block, \ddot{u} the acceleration of the rigid block, \ddot{u}_g the seismic acceleration imposed at the base of the rigid block, *g* the gravity acceleration, μ_a (= $\mu(1-r_u)$) the apparent friction coefficient of the sliding surface, θ the inclination angle, and ζ the dimensionless hysteretic parameter which controls the cyclic frictional response, given in the differential form of Eq. (11).

The first term in Eq. (21) is the inertial force, the second and third terms are the resisting and gravity driven frictional forces, respectively, and the last term, S_{w} , the resultant seepage force. The resisting force is the total frictional force that develops along the sliding surface. Its magnitude depends on two factors: (a) the value of the mobilized friction angle φ' , which is a function of the porewater pressure, and (b) the distribution of the initial normal effective stress along the slip interface. Regarding the first point, Eqs. (17) and (18) are applied. For the second point, the assumption made is that the initial normal effective stress is constant along the whole failure surface. Eq. (10) yields that the initial value of the hysteretic parameter $\zeta(0)$ is the ratio of the initial shear stress ratio $(\tau(0)/\sigma'_{n0})$ to the friction coefficient of the sliding surface. For a sliding block model, $\zeta(0)$ is equal to

$$\zeta(0) = \frac{\tan \theta}{\mu}.$$
(22)

An explicit finite difference technique is used for the solution of Eq. (21), which is coupled with the constitutive equations (11), (17), and (18), and with Eqs. (8), (10), (12),

 Table 1

 Input parameters for the analysis of the Nikawa landslide

n	1	_
b	0.5	_
u_v	10^{-4}	m
λ	8.8	_
ξ	0.2	_
$B_{\rm p0}$	0.72	_
h	2.4	_
e_0	0.7	—
ns	25	_
$\sigma'_{ m n0}$	0.3	MPa
μ	0.52	_
θ	20	degrees



(14), and (15) that provide the necessary coupling between the two substructures (the rigid body and the shear band).

7. Analysis of the Nikawa landslide: results and discussion

The developed model for seismic triggering and evolution of grain-crushing-induced landslide is used to analyze the Nikawa landslide. The parameters of our analysis are summarized in Table 1. The seepage force is ignored, to be consistent with the simplicity of the rigid-block model used



Fig. 15. (a) Acceleration of rigid block (Nikawa landslide), with black line, subjected to Shin-Kobe-NS 1995 acceleration (PGA = 0.53g), with gray line, and (b) evolution of displacement of the rigid block, when grain crushing is not considered.

Fig. 16. (a) Acceleration of rigid block (Nikawa landslide), with black line, subjected to Shin-Kobe-NS 1995 acceleration (PGA = 0.53g), with gray line. The landslide is triggered at t = 7 s approximately. Evolution of velocity (b), and displacement (c) of the rigid block, when grain crushing is considered.



Fig. 17. Evolution of (a) particle breakage potential $B_{\rm p}$, (b) pore-water pressure ratio r_u , and (c) pore-water pressure ratio r_u versus displacement u.

in the analysis. To ensure that our prediction is not heavily (and spuriously) affected by the ground shaking intensity, the NS component of the Shin-Kobe record (PGA = 0.53 g) without any scaling is used as excitation. The response of the potentially sliding wedge is summarized in Figs. 15–17. Specifically:

• Fig. 15a plots the excitation and compares it with the acceleration time history that the wedge (block) experiences when grain crushing is not considered. The corresponding displacement time history of the block is plotted in Fig. 15b. Obviously, the computed 0.23 m of permanent displacement is not consistent with the observed 100 m runout distance of the landslide.

- Fig. 16a plots the acceleration time history of the wedge (block) and compares it with the excitation. Evidently, just after about 6 s, sliding originates at the interface. Initially, the acceleration transmitted into the overlying block is cutoff. Soon, however, at t > 8 s, the acceleration rises monotonically reaching 2 m/s^2 at t = 20 s. Obviously, rapid sliding is unavoidable.
- Figs. 16b and c plot the corresponding evolution of velocity and displacement, respectively. The phenomena are now clear: the speed of sliding picks up dramatically after the triggering of the landslide at approximately t = 7 s, approaching values of about 15 m/s at the end of shaking. All that is needed for a huge displacement to develop is time. During the first 20 s of the analysis, the block has already moved about 80 m. This is in satisfactory (at least qualitatively) agreement with reality.

To get an insight into the mechanics behind this disastrous response, Fig. 17a plots the evolution of particle breakage potential B_p and excess pore-water pressure ratio r_u . Notice that B_p approaches a steady-state value of 0.64 at t > 10 s; this is larger than the initial value of B_{pl} (computed to be 0.59). Recall that the initial value of B_{pl} would be the breakage potential at the end of loading, if no pore-water pressures were developed. The slightly increasing breakage potential at t > 10 s reveals that the grain-crushing process has been practically terminated. The effective normal stress is not adequate for further breakage. However, the landslide is still accelerating due to the action of gravity. Fig. 17b shows that the pore-water pressure buildup is triggered at about t = 5.5 s, while Fig. 17c shows that the pore-water pressure ratio tends to $r_u = 0.7$ at large values of displacement-in accordance with the experimental results from Sassa et al. [9].

8. Conclusions

A constitutive model for grain-crushing-induced porewater pressures has been developed for the analysis of earthquake-induced rapid landslides. The model is based on Hardin's theory [21] for crushing of soil particles under compression and shear loading, and the theory of slidingsurface liquefaction developed by Sassa [7,8]. The model parameters are calibrated against results of experimental tests conducted by Sassa et al. [9]. A sensitivity analysis was conducted to investigate the influence of key model parameters on the pore-pressure generation due to grain crushing. The model is then used to predict the triggering and rapid deformation of the Kobe (1995) Nikawa landslide through sliding block analysis. The results of the analysis are shown to be consistent with those from undrained ring-shear tests [9], as well as qualitatively with field observations [4].

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