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Shear Bifurcation and Gravelization of Low-Strength Concrete

Yuto Yamanoi¹ and Koichi Maekawa*²

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Abstract

Shear failure experiments of concrete beams containing a weak layer were conducted with a focus on the bifurcation of shear localization appearing at the boundary between structure and soil foundation. Low-strength concrete, which is analogous to artificial soft rocks and strengthened foundation, was used to create a weak layer that caused dispersal and bifurcation of the shear localization area, resulting in ductile fracturing of members. Pulverization of hardened cement paste and gravelization (the loss of aggregate particle’s cementation) were observed in shear planes appearing in the weak layer. This confirmed the difficulty of simulating bifurcating shear localization solely by the constitutive law of concrete, which assumes firm cementation by hardened cement paste. In reference to the simulation of the disintegrated concrete slabs for bridge decks under fatigue loads, the transient model from hardened concrete to gravelized assembly was proposed, and it was successfully applied to the bifurcating shear localization of weak layers of low-strength concrete.

1. Introduction

Structural damage caused by the direct impact of fault displacement in the ground has been reported in multiple studies (Ohmachi 2000; Konagai et al. 1999; Lin et al. 2009). Given the difficulty of simulating the rupture process and evaluating safety with respect to these low-frequency, high-risk phenomena, construction sites are determined in the project planning stage so as to avoid the direct impact of principal faults (IAEA 2016).

However, the probability of auxiliary faults derived from principal faults reaching underground structures increases at greater depths of the structures. At nuclear power generation facilities, safety checks are now required in response to auxiliary fault displacement [e.g., dozens of centimeters (IAEA 2019)]. In these checks, it is required to consider not only the failure of man-made structures but also the interaction between man-made structures, bedrock and soil. Ultimate limit states of structures with respect to design values for auxiliary fault displacement have been examined in the past (Yonezawa et al. 2016; Ariga 2007; Anastasopoulos et al. 2008). Furthermore, there is a need for efforts to limit damage to the extent possible (efforts toward resilience), even when limit states are exceeded.

At the Kashiwazaki-Kariwa Nuclear Power Plant, artificial bedrock (roughly 5 MPa of compressive strength) with rigidity and strength designed to be less than that of the concrete foundation was used as a soil improvement additive (Kurihara et al. 1994). When fault displacement reaches the power plant, the artificial soft rock is expected to undergo shear failure before the foundation, thereby mitigating damage to the nuclear power facilities. The same concept was the basis for constructing a soil bentonite wall confirmed in both experiments and analysis to have the effect of limiting damage (Fadaee et al. 2013).

The authors inserted a weak layer into a concrete beam to function as an interface, conducted shear failure tests of structures with different rigidity and strength in each layer, and confirmed that the presence of the weak layer prevents the progression of shear cracks (Yamanoi et al. 2019). This examines weak layers as a possible means of avoiding damage to structures, and proposes a method of simulating the effects in numerical analysis.

The scope of this study is the states in which shear localization occurs in each of the materials of different strength and rigidity that comprise a structure, and in which overwhelming shear deformation could occur in the interfaces between the two materials. There is no guarantee that existing constitutive laws and analytical methods can be applied to forecast the bifurcation and convergence of shear localization areas.

In light of the above, the purpose of this study is to enhance shear localization analysis, including the constitutive law of the low-strength concrete forming the weak layer. The authors also inserted low-strength concrete at the boundary between structural concrete and soil in order to examine the possibility of limiting shear failure modes and simulating them through analysis.

2. Shear localization of members containing parts with sudden changes in rigidity

2.1 Fault displacement and multiple shear localization

Reverse fault displacement acting on underground structures can cause shear localization in each of the
bedrock, the structure and the interface between the two (Fig. 1a). To consider possible combinations of localization, we devised a specimen comprising an upper layer of normal-strength concrete to simulate a concrete foundation, and a lower layer of high-strength concrete to simulate hard bedrock (Fig. 1b). Then, shear forces were applied to the specimen to observe the shear localization of members containing parts with sudden changes in rigidity. The purpose of this experimental observation was to verify the scope of application of analytical models in terms of the bifurcation and gravelization of shear cracks. The experiments were contrived to examine the damage modes and scale effects at actual scale in numerical analysis along extensions of the cracks.

2.2 Overview of experiments

Three types of specimen were made: one type each for Cases I and II with differing joint interface strength, and another for Case III in which a weak layer was inserted as an intermediate layer under the same conditions as Case II. Figure 2 shows the dimensions of each specimen. Table 1 shows the properties of the concrete materials comprising each layer. The weak layer is low-strength concrete with a water to cement ratio of 150%. To prevent material segregation, fine limestone powder was added to achieve a water to powder ratio of 76% of sound consistency. The effective height and reinforcement ratio of Case I differ from those of Cases II and III, but the ratio between the shear span to effective depth (a/d) was 1.0 in all cases. This is because we intended to apply shear forces, which are more intense than bending in consideration of the fault attack to massive concrete slabs. In all cases, two main rebars whose diameter is 32 mm, which is specified as SD390 in JIS G3112 are arranged. No yielding of reinforcing bars occurred in the experiments.

The shape and strength of the joint interface differ between Case I and Case II (Fig. 3). The Case II specimen was washed to expose the coarse aggregates on the interfacial casting plane. In Case III, the specimen with the weak layer inserted, the aggregates were exposed across all joint interfaces.

A mosaic pattern was painted onto one surface of the specimens (Fig. 4), and image measurement was used to measure the displacement of the entire surface of the specimens (Sutton et al. 2009). To double-check precision, displacement was measured by using grounded displacement gauges installed at several locations, including near the specimen supports.

2.3 Results of experiments

Figure 5 shows the semi-space-averaged strains at standard intervals of 1 cm² immediately before failure obtained from photographs and image measurement of the ultimate state of each specimen. In Cases II and III,
two images are placed side by side because each shear span is taken by different cameras so as to avoid the center column of loading machine. Figures 6 to 8 are load-displacement relations of each specimen. The baseline is the position directly above the supports below each specimen, with relative displacement directly below the loading plate in the positive direction (and in the direction of gravity in the case of the grounded displacement gauges). Displacement values obtained from image measurement were referred to for the purpose of double-checking in Case I. Figures 6 and 7 also contain load bearing capacity design values (JSCE 2017), which is calculated by assuming the homogeneous beam with the compressive strength of the normal strength layer in the experiment.

Diagonal shear cracks occurred in both Cases I and II, the two-layered structures. In Case I, which featured a flat joint interface, failure occurred in line with the joint interface. According to the mean distortion immediately before failure obtained from image measurement, delamination could not be observed until immediately before the maximum bearing stress. Coupled with the fact that the load bearing capacity is roughly 20% lower than that of the design bending stress, delamination

| Ultimate state | Image capture (Maximum principal strain distribution) |
|----------------|-----------------------------------------------------|
| **Case I**     | ![Delamination →Shear failure](image)               |
| **Case II**    | ![Shear compression failure](image)                 |
| **Case III**   | ![Anchorage failure](image)                         |

Fig. 5 Failure modes and principal strain distribution of 1 cm² referential area immediately before failure obtained from image measurement.
seems to have caused the collapse of the entire structure. However, delamination was not observed, and shear compression failure occurred in Case II, which has a rough joint interface, indicating the same failure mode as a normal deep beam (Zhang and Tan 2007; Lertsrisakulrat et al. 2002). The load bearing capacity was the median value of the bearing stress derived by using the strength of the concrete on the upper layer in the shear capacity equation from the standard specifications of the Japan Society of Civil Engineers (JSCE) and the bearing stress calculated using the values for the high-strength concrete on the lower layer. This suggests that, on structures containing parts with sudden changes in rigidity, the bond between parts with sudden changes in strength has a greater impact on the entire structure.

In Case I, the joint interface between the upper and lower layers was the weak part. In Case III, a weak layer with some thickness (low-strength concrete layer with finite volume) exists between the upper and lower layers. Straight, diagonal cracks passing directly through the upper and lower layers occurred under a load of roughly 350 kN, and damage in which the shear localization area spread horizontally through the weak layer appeared under a load of roughly 580 kN. Afterward, the cracks progressed toward the lower layer from the edge of the damaged area in the weak layer, forming a “crooked path” of shear localization. Shear cracks reached the area of reinforcement anchoring, causing brittle anchorage failure.

In contrast to Case II, in which the diagonal cracks passed straight through the upper and lower layers, the existence of the weak layer in Case III resulted in the dispersal of the shear localization area. This confirms that inserting layers with differing strength can change the progression of shear localization areas. Additionally, as shown by the load-displacement curves (Fig. 8), the existence of a weak layer improves member ductility.

3. Failure response analysis

3.1 Analytical models

Numerical analysis models with proven applicability on normal-strength concrete were used to perform failure response analysis in each of the cases. The mesh discretization of each case is shown in Fig. 9. Reinforced concrete (RC) elements were used for reinforced parts and unreinforced elements were used for all other areas (An et al. 1997). The gravity center of RC elements zone coincides with the location of the deformed bar. In Case I, where the bending fracture prevailed, the analysis result is more sensitive to the RC zone than that in Cases II and III, where shear failure prevailed. Thus, in Case I, the RC zone was strictly adjusted to the location of reinforcement as well. Then, the analytical bending rigidity matches the reality. A non-orthogonal multidirectional cracking model was applied to the RC elements as stated above. Table 2 is a summary of the major component models (Maekawa et al. 2003; Maekawa and Fukuura 2014).

In order to consider the weak joint of the interface between upper and bottom layer, the joint interface includes in-plane joint elements capable of accounting for both delamination and misalignment in Case I. The joint element model (Maekawa et al. 2008) was used for both shear slides and joint element’s openings.

In Case I, moderate heat Portland cement was used for high-strength concrete and ordinary Portland cement was used for normal-strength concrete (Yamanoi et al. 2019), whereas in Cases II and III, the whole concrete...
was made from early-strength Portland cement with large volumetric shrinkage (Tazawa and Miyazawa 1995). Initially induced stresses may not be ignored as Sato and Kawakane (2008) pointed out the reduced shear capacity caused by autogenous shrinkage. The multi-scale thermo-hygral simulation, which integrates micro-scale events of cement hydration and moisture migration with macro-scale structure responses, has been applied. Then, the effect of autogenous and drying shrinkage of concrete was taken into account (Maekawa et al. 2009; Yoneda et al. 2013) for Cases II and III. The input values of each case are summarized in Table 3. It is noted that the simulation traced the hydration of cement-concrete accompanying the strength development and the curing conditions. Thus, the comparatively large shrinkage of the weak layer was also considered in the analysis.

### 3.2 Results of damage response analysis

Figures 10 and 11 show the results of numerical analysis and experiments for Case I and Case II, respectively. In Case I, the analysis result overestimated the experimental diagonal crack strength. This is probably due to the effects of concrete shrinkage (although smaller than Cases II and III). However, the delamination of the joint interface caused the entire collapse the same as experiment. In view of the failure mode, the analysis can reproduce the experimental result by using joint elements. In Case II, both cases with and without shrinkage of concrete are compared in the load-displacement curve. If the thermo-dynamics of concrete is not considered (“not integrated” in Fig. 11), the rigidity and diagonal crack strength are overestimated but ultimate failure mode is not so changed. On the other hand, the initial strain induced by the integrated analysis realizes the early diagonal cracks followed by gradual increase in load. These results confirm that it is possible to simulate damage on the whole using the results of experiments in both cases in light of the simulations of delamination using joint elements; moisture migration and drying shrinkage from the thermodynamic interaction analysis; and the like.

However, Case III (integrated analysis) resulted in earlier damage to the weak layer and bearing capacity estimations roughly 60% of experimental values despite of the fact that the initial rigidity of the members was nearly identical. In addition, the shear localization area failed to progress from the weak layer to the high-strength area of the lower layer (Fig. 12). Although the behavior in which the damage spread from the span center of the weak layer to its edge was identical, analysis revealed excessive compressive deformation. For the low-strength part, the presumed result is a lower estimation of confinement effects than actual figures (Yamanoi et al. 2019).
Table 3 Input value in each case.

| Case | Property | Input value | Input value*2 | Input Value common to all layers*2 |
|------|----------|-------------|---------------|-----------------------------------|
|      | $E_0$   | $f_c$       | $f_t$         | $\varepsilon_s$ | $\varepsilon_{WG}$ | $\varepsilon_{WS}$ | $\varepsilon_{WL}$ | $\varepsilon_{WP}$ | $\varepsilon_{BLN}$ | $\varepsilon_{BLNLS}$ | $\varepsilon_{P3A}$ | $\varepsilon_{P3S}$ |
| I    | GPa     | MPa         | MPa           | micro          | kg/m$^3$          | kg/m$^3$          | kg/m$^3$          | %          | cm$^3$/g         | cm$^3$/g         | %          | %          |
| N    | 31.4    | 46.1        | 3.43          | 0.0           | -                 | -                 | -                 | -          | 4210.0          | 7000.0          | 9.00        | 63.00       |
| H    | 45.7    | 110.2       | 6.05          | 0.0           | -                 | -                 | -                 | -          | P4AF            | P2S             | %          | %          |
| II, III | N | 29.7 | 44.9 | 3.40 | -459.9 | 958.0 | 861.0 | 0.0 | 50.0 | % | % | % |
| L    | 13.1    | 11.14       | 6.33          | -139.0        | 815.0             | 598.0             | 0.0              | 50.0        | 8.00            | 12.00           | 6.45        |

W/C: Water cement ratio, N: Normal strength concrete, H: High strength concrete, L: Low strength concrete.
$E_0$: Young's modulus, $f_c$: Compressive strength, $f_t$: Tensile strength, $\varepsilon_s$: Shrinkage strain computed by the 1 element analysis under the same environmental condition of each case.
WG, WS, WL: Unit weight of each coarse aggregate, fine aggregate and non-cementitious powder, WP: Water to cementitious powder ratio.
BLN, BLNLS: Effective Blaine values of each cement and non-cementitious powder, P3A, P3S, P4AF, P2S, PPCS2H: Weight percentage of mineral compounds as C$_3$A, C$_3$S, C$_4$AF, C$_2$S and gypsum of ordinary Portland cement.

*1: The value obtained from the cylindrical test is directly inputted (Case I). The value calculated from the solidification model included in the integrated analysis is inputted (Case II and Case III).

*2: Only the case of the integrated analysis (Case II and Case III).

Fig. 10 Comparison diagram of analysis and experimental result of Case I.

Fig. 11 Comparison diagram of analysis and experimental result of Case II.

Fig. 12 Comparison diagram of analysis and experimental result of Case III.
The constitutive laws of normal-strength concrete appear incapable of sufficiently reflecting the dynamic properties of low-strength concrete with inferior cementation by hardened paste. In reality, granular materials like soil with no cementation exist along extensions of low-strength concrete. Thus, further discussion will be conducted in the next section focusing on the constitutive laws of concrete with compressive strength (denoted by $f_c$) of 10 MPa or less under large localized shear deformation.

4. Transition model for degree of cementation

Figure 13 shows the failure state of the weak layer associated with the bifurcation of shear localization. In areas with minor deformation, scattered cracks, i.e., a defining characteristic of concrete, were confirmed. In areas that suffered from major damage, the hardened cement paste is pulverized, resulting in the nearly complete loss of cementation of the sand and gravel. Compressive struts deployed on the RC beams confine the weak layer above and below. Furthermore, the weak layer undergoes progressive shear deformation. In numerical analysis, the principal compressive stress in the weak layer nearly exceeds the compressive strength, reaching the softening area.

As shown in Fig. 14, as the failure of the hardened cement paste spreads, it transitions from a binder of consolidated aggregates to a granular state that enables the aggregate particles to rearrange themselves. On the other hand, deterioration of the degree of cementation can be considered to assume the properties of sandy soil materials that depend heavily on confining pressure.

For the trial analysis, the soil constitutive model (Soltani and Maekawa 2016), which was validated by the experiments of bearing capacity on the flat and inclined foundation with different sizes, was applied to the weak layer as shown in Fig. 15. Here, we assumed isotropic clay-soil model with zero internal frictional angle.
and a cohesive strength of 3.83 $f_c$ (MPa), which is equivalent to the crack shear transfer strength of concrete (Li et al. 1989). The initial stiffness was adjusted to be the same as that of concrete as discussed in Fig. 12.

The computed shear band propagates from the lower area of the high strength concrete to the upper one though the weak layer without bifurcation. This is thought to be attributed to the sustained shear strength irrespective to the confinement, and the capacity of the specimen gets higher. Then, it is implied that the shear band bifurcation as shown in Fig. 5 is driven and/or initiated by the high anisotropy like concrete.

Here, the spatially mobilized concept is proposed for examining the shear behavior of both concrete and soil materials of gravelized state. Examples have been reported in which component models based on this concept have been expanded into models for cement-modified soil and other materials with low-strength consolidation (Matsuoka and Sun 1995). Similarly, in this paper, we attempt to evaluate quantitatively the deterioration of the degree of cementation by combining a normal-strength concrete model and a frictional material model and varying the mix ratios of each.

Figure 16 shows the concept of the proposed model of low-strength concrete. The left and right parts in the figure are the representative mechanical behavior of concrete and frictional material respectively. The center part expresses the internal stress intensity, which is considered in the constitutive model of concrete based on an elasto-plastic and fracture model (Maekawa et al. 2003). The fracture parameter denoted by $K$, which expresses damage, represents shear elastic energy retention, and its initial value is unity. It shows that, as fine micro-cracks progress, the effective volume for resisting shear stress decreases. The remaining effective volume indicated by $K$ is calculated from the shear elastic rigidity at unloading. The model is made to reflect the fact that the lost effective volume of mechanics is not recovered afterward, because the cracking is irrecoverable.

However, if the cementitious binding of the hardened cement paste is lost, the aggregate particles should be able to rearrange themselves with dilatancy, and the paste could assume the properties of a frictional material. Thus, we assume that the lost effective volume of concrete composite bears some of the stress under triaxial confinement as a frictional material from that point forward, and propose the following mixture rules.

$$\sigma_\alpha = \sigma_\alpha (K) + \alpha (1-K) (S_\alpha + \delta_y I_1)$$ (1)

where $\sigma_\alpha$ is the total stress tensor, $\sigma_\alpha$ is the stress tensor yielded by an elasto-plastic and fracture model for concrete, $S_\alpha$ is the deviatoric stress tensor yielded by a soil model, $I_1$ is the first invariant of the total stress of soil, $K$ is the concrete fracture parameter, and $\alpha$ is a coefficient to consider the difference of pure soil and the pulverized one ($=1$ tentatively in this study). The model approximates the behavior of soil materials linked to the deterioration of cementation. The fracture parameter $K$, which determines the transition rates of both models, is expressed in the following equation using an existing elasto-plastic and fracture model.

$$K = \exp \left[ \frac{F}{3.25 \left[ 1 - \exp \left( \frac{F}{0.8} \right) \right]} \right]$$ (2)

where $F$ is given as a function of elastic strain invariants $I_1$, $J_2$, and $J_3$ (Maekawa et al. 2003).

When the adherence of hardened cement paste is high, the confining stress that builds within structures generally remains sufficiently small in comparison with the uniaxial compressive strength of the concrete. In areas with high compressive strain, separation into block groups several times the size of aggregates occurs. Under dry conditions, the hardened cement paste is pulverized. However, under repetitive stress underwater, the hardened cement paste rapidly comes apart, resulting in gravelization and disintegration (Maekawa and Fujiyama 2013). In this case, variance in pore pressure and Parameter $K$ are calculated to estimate the remaining life of RC bridge deck slabs as the concrete undergoes gravelization by Eq. (1). In this study, we attempt to project the bifurcation of shear localization based on the assumption that the transition of the degree of cementation of low-strength concrete has the same mechanism as that of the fatigue disintegration of bridge deck slabs as stated above.
The constitutive laws of soil materials included in Eq. (1) are an elasto-plastic model based on certain yield criteria of angles of internal friction. Shear dilatancy no longer progresses under high confining pressure. Then, we proceed with the above-stated model to simply ignore volumetric changes. The confining pressure effects are determined by the angle of internal friction.

Figure 17 shows analysis when the transition model is applied to the weak layer by Eq. (1) where the sandy soil’s stress is computed by the multi-yield surface plastic model (Towhata and Ishihara 1985; Towhata 2008; Soltani and Maekawa 2016) with the internal friction angle of 45 degrees and cohesive strength of zero. The soil model built into Eq. (1) was applied to the entire history of forces applied to the weak layer, and numerical analysis was used to confirm an increase in load bearing capacity. This is understood as the effect of the soil material having a higher dependency on confining pressure than the concrete.

When used with the concrete model of firm cementation of aggregates as discussed in Fig. 12, the weak layer reflected the highly anisotropic nature associated with the cracks, accurately simulating the bifurcation of shear cracks in the analysis. However, the subsequent deterioration of cementation was not reflected in the concrete model solely, and it was not very responsive to confining pressure. Thus, we suppose that the result was a low estimation of bearing stress. When the proposed transition model was applied to the weak layer, both the initial high anisotropy of the deformation and the high dependency on confining pressure after major deformation of gravelization were considered. As a result, both the load-displacement curves and the progression and bifurcation of shear localization areas were simulated.

The weak layer’s effect of dispersing and bifurcating the damage is derived from the anisotropic nature of the multidirectional cracking peculiar to concrete. The flow of load bearing capacity after the progression of damage based on dependency on the confining pressure of the soil likely corresponds almost exactly to the state of the damage to the weak layer.

There is room for future improvement of the transition model used in this study because the simple, perfect elasto-plastic model represented the friction material model that preceded the transition. Additionally, the homogeneity with the fatigue gravelization of bridge deck slabs assumed in this study is viewed as being in the stage of estimation based on conditions. We view this as yielding a point worthy of attention in the evolution of a unified understanding of low-strength concrete, cement-modified soil, and other intermediary materials.

5. Conclusions

The localization of shear deformation is a failure mode in the same way as soil materials, low-strength concrete, and normal-strength one, and the interaction between shear cracks and faults is the main viewpoint of this paper. The following are the main conclusions drawn from this study.

(1) Experiments have shown that delamination and shearing in interfaces containing parts with sudden changes in rigidity cause brittle failure across entire structures. In this case, establishing boundary elements for the interfaces makes it possible to simulate failure modes quantitatively using numerical analysis.

(2) Successfully demonstrated was a case in which inserting a weak intermediate layer (a weak layer) at parts with sudden changes in rigidity causes shear
cracks to bifurcate from the weak layer, transitioning to ductile damage modes.
(3) Multiple scattered cracks were observed at the outset of shear localization of the low-strength concrete that formed the weak layer. In areas of major principal strains, the pulverization of the hardened cement paste, and gravelization in which the cementation between the aggregates was lost, were confirmed.
(4) Although it was possible to simulate the bifurcation of shear localization by using nonlinear analysis based on existing constitutive laws of concrete, the simulated bifurcation did not progress to the successive strong layers. The ultimate bearing stress was successfully reproduced by using the constitutive laws of soil, but it was unable to simulate the bifurcation that developed in the weak layer.
(5) Based on the results of experiments and observations, the authors applied a collapsing cementation model to the constitutive law of low-strength concrete and incorporated behavior that approximates granular soil material undergoing great shear deformation. This enabled us to accurately simulate the bifurcation of shear localization and shear cracks as well as load bearing capacity.
Transient gravelization process from solidified concrete anisotropy to more isotropic assembly of particles is not fully clarified in this study. Thus, the authors will investigate this issue with much lower strength composites in future. This transient process may play some role in the interface of brick masonry structures.

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References
An, X., Maekawa, K. and Okamura, H., (1997). “Numerical simulation of size effect in shear strength of RC beams.” *Doboku Gakkai Ronbunshu*, 564/V-35, 297-316. (in Japanese)
Anastasopoulos, I., Gerolymos, N., Drosos, V., Georgarakos, T., Kourkoulis, R. and Gazetas, G., (2008). “Behavior of deep immersed tunnel under combined normal fault rupture deformation and subsequent seismic shaking.” *Bulletin of Earthquake Engineering*, 6, 213-239.
Ariga, Y., (2007). “Fundamental study on analysis method for behavior of concrete dam against fault displacement.” *JSCE Journal of Earthquake Engineering*, 29, 690-697. (in Japanese)
Fadaee, M., Anastasopoulos, I., Gazetas, G., Jafari, M. K. and Kamalian, M., (2013). “Soil bentonite wall protects foundation from thrust faulting: Analyses and experiment.” *Earthquake Engineering and Engineering Vibration*, 12(3), 473-486.
IAEA, (2016). “*IAEA Safety standards for protecting people and the environment: Site evaluation for nuclear installations, No. NS-R-3 (Rev. 1): Safety requirements.*” Vienna, Austria: International Atomic Energy Agency.
IAEA, (2019). “*IAEA Safety standards for protecting people and the environment: Site evaluation for nuclear installations, No. SSR-1: Specific safety requirements.*” Vienna, Austria: International Atomic Energy Agency.
JSCE, (2017). “*Standard specification of concrete structures (Design).*” Tokyo: Japan Society of Civil Engineers.
Konagai, K., Mikami, A., Katagiri, T., Ahsan, R. and Maruyama, D., (1999). “Report of the damage caused by the mid-north Iwate earthquake of September 3.” *Bulletin of Earthquake Resistant Structure Research Center, University of Tokyo*, 32, 3-13.
Kurihara, H., Kikuchi, K. and Fukazawa, E., (1994). “Experimental study on durability of artificial soft rock.” *Doboku Gakkai Ronbunshu*, 486/VI-22, 85-94. (in Japanese)
Lertsrisakulrat, T., Niwa, J., Yanagawa, A. and Matsuo, M., (2002). “Concepts of localized compressive failure of concrete in RC deep beams.” *Doboku Gakkai Ronbunshu*, 697/V-54, 215-225. (in Japanese)
Li, B., Maekawa, K. and Okamura, H., (1989). “Contact density model for stress transfer across cracks in concrete.” *Journal of the Faculty of Engineering, The University of Tokyo (B)*, 40(1).
Lin, A., Ren, Z., Jia, D. and Wu, X., (2009). “Co-seismic thrusting rupture and slip distribution produced by the 2008 Mw 7.9 Wenchuan earthquake, China.” *Tectonophysics*, 471(3-4), 203-215.
Maekawa, K. and Fukuura, N., (2014). “Nonlinear modeling of 3D structural reinforced concrete and seismic performance.” In: T. T. C. Tsu, C. Wu and J. Li, Eds. *Infrastructure Systems for Nuclear Energy*. New York: John Wiley and Sons.
Maekawa, K. and Fujiyama, C., (2013). “Rate-dependent model of structural concrete incorporating kinematics of ambient water subjected to high-cycle loads.” *Engineering Computations*, 30(6), 825-841.
Maekawa, K., Ishida, T. and Kishi, T., (2009). “Multi-scale modeling of structural concrete.” London: CRC Press, Taylor and Francis Group.
Maekawa, K., Fukuura, N. and Soltani, M., (2008). “Path-dependent high cycle fatigue modeling of joint interfaces in structural concrete.” *Journal of Advanced Concrete Technology*, 6(1), 227-242.
Maekawa, K., Pimanmas, A. and Okamura, H., (2003). “Nonlinear mechanics of reinforced concrete.” London: CRC Press, Taylor and Francis Group.
Matsuoka, H. and Sun, D., (1995). “Extension of spatially mobilized plane (SMP) to frictional and cohesive materials and its application to cemented sands.” *Soils and Foundations*, 35(4), 63-72.
Ohmachi, T., (2000). “On damage to dams in Taiwan due
to the 1999 Chichi earthquake.” Journal of Japan Society of Dam Engineers, 10(2), 138-150. (in Japanese)
Sato, R. and Kawakane, H., (2008). “A new concept for the early age shrinkage effect on diagonal cracking strength of reinforced HSC beams.” Journal of Advanced Concrete Technology, 6(1), 45-67.
Soltani, M. and Maekawa, K., (2016). “Numerical simulation of progressive shear localization and scale effect in cohesionless soil media.” International Journal of Non-Linear Mechanics, 69, 1-13.
Sutton, M. A., Orteu, J. J. and Schreier, H., (2009). “Image correlation for shape, motion and deformation measurements: Basic concepts, theory and applications.” New York: Springer US.
Tazawa, E. and Miyazawa, S., (1995). “Influence of cement and admixture on autogenous shrinkage of cement paste.” Cement and Concrete Research, 25(2), 281-287.
Towhata, I. and Ishihara, K., (1985). “Modeling soil deformation undergoing cyclic rotation of principal stress axis.” Soils and Foundations, 25(2), 135-147.
Towhata, I., (2008). “Geotechnical earthquake engineering.” Berlin and Heidelberg: Springer-Verlag.
Yamanoi, Y., Aoki, H. and Maekawa, K., (2019). “Interaction of shear band of rock/soil foundation and failure of underground RC ducts.” In: H. Barros, C. Ferreira, J. M. Adam and J. Delatte, Eds. Proc. 3rd International Conference on Recent Advances in Nonlinear Design, Resilience and Rehabilitation (CoRASS2019), Coimbra, Portugal 16-18 October 2019. Barcelona, Spain: European Community on Computational Methods in Applied Sciences (ECCOMAS), 44-53.
Yoneda, T., Ishida, T., Maekawa, K., Gebreyouhannes, E. and Mishima, T., (2013). “Simulation of early-age cracking due to drying shrinkage based on a multi-scale constitutive model.” In: H. Christian, P. Bernhard and A. Dietmar, Eds. Proc. 5th Biot Conference on Poromechanics (Poromechanics V), Vienna, Austria 10-12 July 2013. Reston, Virginia, USA: American Society of Civil Engineers, 579-588.
Yonezawa, K., Higuchi, S., Anabuki, T., Watanabe, N. and Itoh, G., (2016). “Damage estimation of underground RC-structure subjected to bedrock displacement by non-linear 3D FE-analysis.” JSCE Journal of Structural Engineering A, 62(A). (in Japanese)
Zhang, N. and Tan, K. H., (2007). “Size effect in RC deep beams: Experimental investigation and STM verification.” Engineering Structures, 29, 3241-3254.