

Seismic fragility analysis of railway embankment structures considering shallow bedrock ground conditions

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ABSTRACT

Railway embankment structures are a key component of linear infrastructure and may affect the operation of the railway network during earthquakes due to structural damage and safety-related operational restrictions. Accordingly, seismic fragility analysis is essential for evaluating the seismic response and damage potential of railway embankment structures.

In this study, the seismic fragility of railway embankment structures was evaluated under ground conditions characterized by relatively shallow bedrock, with Korea considered as a representative example of such conditions. To account for these characteristics, site classification criteria (S_1 – S_5) defined in the Korean Design Standard (KDS 17 10 00) were applied. Bedrock depth, shear wave velocity of the underlying soil layer, and embankment height were considered as key parameters, and seismic fragility functions were derived using permanent vertical displacement obtained from nonlinear dynamic time-history analyses. The fragility analysis results indicate that seismic fragility increases with increasing bedrock depth and decreasing shear wave velocity of the soil layer. In addition, the influence of embankment height varies depending on ground conditions, with ground amplification effects exerting a stronger influence on the seismic response under soft ground conditions, whereas inertial effects associated with increased embankment mass become more influential under relatively stiff ground conditions. Overall, this study provides fundamental data for the seismic performance evaluation and seismic design of railway embankment structures located on ground characterized by shallow bedrock.

1. Introduction

Railway infrastructure is a key component of critical social infrastructure that supports large-scale population mobility and freight transportation, and it plays an important role in the transportation network. As a linear infrastructure network composed of bridges, tunnels, and embankments, railway infrastructure may experience network-wide operational impacts when operational limitations arise in a localized section [1]. In particular, when natural hazards such as earthquakes occur, railway infrastructure may experience structural damage due to the effects of seismic loading [2]. Regardless of whether structural damage occurs, operational limitations may be imposed following an earthquake to ensure safety, which can lead to substantial earthquake-induced impacts on railway infrastructure [3]. These earthquake-induced impacts have been verified through earthquake case histories in the form of diverse damage patterns observed in railway

infrastructure, clearly demonstrating the need to assess its seismic fragility.

A wide range of damage has been observed across individual railway infrastructure components in earthquakes worldwide. During the 1999 Chi-Chi earthquake in Taiwan, the Sanyi railway tunnel, constructed using the NATM method, experienced structural damage induced by seismic loading, including damage to the tunnel lining [4]. During the 2011 Tohoku earthquake in Japan, seismic loading caused various forms of structural damage in railway embankments and bridge foundations, including settlement and foundation deformation [5]. In the 2022 Mw 6.9 Menyuan earthquake in China, severe damage to the girders and rails was observed in a high-speed railway bridge located in the near-fault region [6]. During the 2023 Mw 7.7 Pazarcik earthquake in Türkiye, a surface fault rupture induced by the earthquake cut through a railway tunnel, causing severe structural damage, including partial tunnel collapse [7]. Accordingly, as diverse forms of damage have been

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observed in railway infrastructure during earthquakes, numerous dynamic numerical analysis studies have been conducted to investigate the response to seismic loading and the associated damage characteristics. These dynamic numerical analysis studies have provided a basis for seismic fragility assessment based on seismic response characteristics and damage measures.

On this basis, studies have been proposed that utilize results from dynamic numerical analyses to evaluate the seismic fragility of structures comprising railway infrastructure. For bridges, seismic fragility analyses have been conducted for simply supported and continuous girder bridges, focusing on the seismic responses of the girders and bridge piers [8–10]. Seismic fragility was also evaluated for cut-and-cover and circular tunnels by using damage indices based on member bending-moment responses and lining deformations obtained from dynamic numerical analyses [11–13]. In contrast, embankments, unlike bridges and tunnels, are continuum structures composed of geomaterials rather than discrete structural members such as girders, piers, and linings, and therefore their structural characteristics limit the direct applicability of conventional fragility analysis methods based on member bending-moment responses. Accordingly, prior studies on embankments have performed dynamic numerical analyses and seismic fragility analyses by using dynamic responses induced by earthquakes, such as permanent vertical displacement and horizontal displacement, as damage indices rather than the responses of structural members.

Considering these structural characteristics, numerous numerical studies have been conducted to analyze the seismic behavior of embankments. Lin et al. [14] developed a full-scale numerical model of an embankment using FLAC3D and analyzed its seismic behavior. Based on the analysis of response accelerations and dynamic displacement responses at 18 locations within the embankment, acceleration amplification was greater at the embankment crest than at the lower portion, and it was confirmed that, as the input horizontal peak acceleration increased, residual deformation at the embankment crest increased more rapidly than that in the lower portion. Namdar [15] developed two embankment models with different heights and performed numerical analyses for each model under both unidirectional and multidirectional seismic loading. As a result, both embankment models exhibited larger strains under multidirectional ground motions, indicating that multidirectional seismic loading can reproduce actual earthquake responses more realistically.

Using the results of dynamic numerical analyses, seismic fragility analyses have been conducted to evaluate the probability of exceeding damage states for embankments. Hübner and Mahler [16] performed two-dimensional nonlinear analyses using PLAXIS 2D by varying the embankment height while keeping the underlying ground conditions identical, and derived a seismic fragility function based on horizontal displacement. The analysis results confirmed a tendency for horizontal displacement at the embankment crest to increase as the embankment height increased, suggesting that embankment height is a key variable influencing fragility. Argyroudis and Kaynia [17] performed two-dimensional numerical analyses considering embankment height as a variable under ground conditions classified as C and D according to Eurocode 8, and derived a seismic fragility function by using permanent vertical displacement as the damage index. As a result, seismic fragility was found to increase with softer underlying ground and with increasing embankment height, indicating that the effects of ground conditions and embankment geometry are reflected in the fragility. Shinoda et al. [18] derived a seismic fragility function for an embankment designed in accordance with Japanese railway design standards by using the permanent sliding displacement of the embankment slope, estimated using the Newmark method, as the damage index. As a result, seismic fragility was shown to increase with increasing embankment height and decreasing soil friction angle, whereas increasing reinforcement tensile strength reduced fragility. Mohammadi et al. [19] derived a seismic fragility function for railway embankments by using permanent vertical displacement, estimated from nonlinear time-history analyses, as the

damage index. As the results of a parametric investigation considering the embankment aspect ratio (H/B), soil friction angle and cohesion, and characteristics of near-field and far-field ground motions as key variables, fragility increased as H/B increased and as soil friction angle and cohesion decreased, and near-field ground motions in particular tended to increase the probability of embankment collapse.

Additionally, to analyze the seismic behavior of embankments under various ground conditions, some numerical and seismic fragility analysis studies have been conducted for embankments on liquefiable soils. Oblak et al. [20] proposed individual seismic fragility functions for each variable influencing the response of embankments on liquefiable soils. It was confirmed that seismic fragility increased with increasing thickness of the liquefiable layer, whereas fragility decreased as the embankment width and the density of the liquefiable layer increased. Zhou et al. [21] derived seismic fragility functions according to whether liquefaction mitigation strategies were considered, and suggested that the effectiveness of each mitigation strategy can be quantitatively evaluated at the engineering design stage. Based on a comparative evaluation of soil densification, gravel berm, and sheet-pile enclosure, sheet-pile enclosure was identified as the most effective mitigation strategy for suppressing lateral displacement.

Although studies evaluating the seismic behavior and fragility of embankments during earthquakes have been continuously conducted worldwide, limited attention has been given to the influence of shallow bedrock conditions on the seismic performance of railway embankments. In particular, under conditions where bedrock is relatively shallow, differences in soil layer thickness can result in more pronounced variations in seismic response characteristics. Understanding these effects is important for improving the seismic resilience of railway infrastructure systems located in regions characterized by shallow bedrock conditions.

Korean ground conditions represent a typical case characterized by relatively shallow bedrock depth. In this context, shallow bedrock refers to bedrock depths less than approximately 30 m, which corresponds to the depth commonly used in V_{s30} -based site classification. Geological investigations in Korea frequently report bedrock depths within this range. Therefore, the bedrock depths considered in this study (1–28 m) represent typical shallow bedrock conditions observed along Korean railway corridors. Accordingly, this study applied the site classification criteria specified in the Korean Design Standard for Seismic Design (KDS 17 10 00) to account for such conditions. Fifteen analysis cross-sections were defined, and the seismic fragility of railway embankments was evaluated through dynamic numerical analyses. Based on these results, this study aims to provide fundamental reference data to support the seismic performance assessment and seismic design of railway embankments located on shallow bedrock sites.

2. Numerical analysis model and analysis conditions

This study constructed a two-dimensional finite element model under plane-strain conditions using OpenSees [22] to perform seismic fragility analyses of railway embankments. Plane-strain conditions are applied by assuming that the structure is continuous in the longitudinal direction, and three-dimensional behavior is simplified to a two-dimensional cross-section by setting the longitudinal strain to zero. This method is well-suited for modeling the behavior of structures that are continuous in the longitudinal direction, such as railway embankments. Since the cross-sectional geometry of embankment structures is uniform along the longitudinal direction, the dominant deformation occurs within the transverse cross-section, and thus two-dimensional plane-strain analysis can be considered an appropriate approach for developing seismic fragility functions of embankment structures. This method also offers the advantage of improving computational efficiency, and the same methodology has been adopted in previous studies on seismic fragility analysis of embankment structures [17,19]. The finite element method discretizes the structure into a finite number of

elements, numerically evaluates the mechanical behavior of each element, and assembles the results to approximate the global structural response. Based on this numerical modeling framework, this study developed a numerical analysis model to perform seismic fragility analyses of railway embankment structures, and defined the analysis conditions and input ground motions.

2.1. Numerical analysis cases

In this study, the Korean Design Standard for Seismic Design was adopted as the site classification criteria for the analysis. In general, site classification is internationally based on the average shear wave velocity of the upper 30 m of the ground (V_{s30}). However, under site conditions characterized by relatively shallow bedrock, even sites with the same V_{s30} may exhibit different dynamic ground behavior depending on bedrock depth and the resulting soil layer thickness. According to previous studies, even when sites share the same V_{s30} , differences in bedrock depth and bedrock properties can lead to variations in inherent site characteristics and seismic response. This effect is particularly pronounced under shallow bedrock conditions (bedrock depth <30 m), where significantly different site response characteristics have been reported [23].

In Korea, bedrock is commonly distributed within 30 m from the ground surface, and under such ground conditions, seismic response characteristics may vary even for sites classified under the same V_{s30} -based site classification criteria. Accordingly, in this study, the site classification criteria S_1 to S_5 specified in the KDS 17 10 00 [24] were applied by considering conditions characterized by relatively shallow bedrock depth, and the numerical analysis cases were defined based on these criteria. The site classification criteria for Korean ground conditions specified in KDS 17 10 00 are summarized in Table 1.

The numerical analysis cases presented in Table 2 consist of a total of 15 cases, with embankment height, bedrock depth, and shear wave velocity of the soil layer selected as the primary variables. The embankment height was defined considering that the average embankment height along the Honam High-Speed Railway embankment sections is approximately 6 m. The Honam High-Speed Railway, along with many other high-speed railway lines in Korea, employs a concrete slab track system in its embankment sections. Accordingly, the analysis conditions and damage thresholds in this study were defined with reference to the design and maintenance criteria applicable to slab track systems. Based on this consideration, three embankment height conditions of 3 m, 6 m, and 8 m were adopted for comparative analysis.

The underlying ground conditions were also defined by applying the site classification criteria (S_1 – S_5) specified in KDS 17 10 00, based on ground conditions representative of the Honam High-Speed Railway embankment sections. For each site classification category, the shear wave velocity and bedrock depth were determined using representative average values to adequately reflect the characteristics of the corresponding classification criteria. In this process, the underlying soil layer was modeled as a uniform sandy soil condition for each numerical analysis case. This simplification was adopted to establish a parametric framework for independently evaluating the effects of the primary variables, namely bedrock depth, shear wave velocity of the underlying

Table 1
Site classification criteria [24]

Soil Type	Description of Soil	Classification Criteria	
		Depth to Bedrock, H (m)	Shear Wave Velocity (m/s)
S_1	Rock	Less than 1	-
S_2	Shallow, stiff soil	1–20	≥ 260
S_3	Shallow, soft soil		≤ 260
S_4	Deep, stiff soil	Greater than 20	≥ 180
S_5	Deep, soft soil		≤ 180

Table 2
Analysis cases.

Case No.	Embankment Height (m)	Bedrock Depth (m)	Shear Wave Velocity (m/s)	Soil Type	V_{s30} (m/s)	IBC Site Class
1	3	1	330	S_1	784	SB
2	3	13	330	S_2	596	SC
3	3	13	200	S_3	540	SC
4	3	28	280	S_4	315	SD
5	3	28	160	S_5	203	SD
6	6	1	330	S_1	784	SB
7	6	13	330	S_2	596	SC
8	6	13	200	S_3	540	SC
9	6	28	280	S_4	315	SD
10	6	28	160	S_5	203	SD
11	8	1	330	S_1	784	SB
12	8	13	330	S_2	596	SC
13	8	13	200	S_3	540	SC
14	8	28	280	S_4	315	SD
15	8	28	160	S_5	203	SD

soil layer, and embankment height, on seismic fragility. For reference, Table 2 also presents the V_{s30} values calculated for each numerical analysis case, along with the corresponding site classifications based on the International Building Code [25].

For the embankment, the crest width was set to 16 m for all numerical analysis cases, and the embankment material was modeled as a uniform sandy soil with a shear wave velocity of 290 m/s. The embankment slope was defined as 1:1.8 in accordance with the standard embankment slope for railway earthworks specified in the Korean Design Standard for Earthworks—Embankment and Cut [26], which corresponds to the high-speed railway design criterion. The standard slopes for railway embankments specified in KDS 11 70 05 are summarized in Table 3.

2.2. Input ground motions

The input ground motions were selected based on earthquake records observed in Korea and recorded ground motions included in the NGA-West2 database [27]. In this study, real recorded ground motions were used as input motions to evaluate the seismic response of the embankment while reflecting the time-history characteristics of actual earthquake records. The earthquake records observed in Korea were obtained from the recorded acceleration data provided by the National Earthquake Comprehensive Information System (NECIS) [28] of the Korea Meteorological Administration. Among the earthquake records from 2007 to 2017, those with available acceleration time histories suitable for analysis were collected, resulting in a total of 9 original ground motion records. In addition, 11 original ground motion records were selected from the NGA-West2 database, focusing on earthquakes with moment magnitudes in the range of 5.0 to 7.5.

Both the EW and NS horizontal components of each original ground motion record were used as independent input ground motions. Accordingly, the EW and NS horizontal components of the 20 original ground motion records, consisting of 9 Korean records and 11 NGA-West2 records, were used to construct a total of 40 input ground motions. This was done to account for the fact that the acceleration time-

Table 3
Standard railway embankment slopes [26].

Height to Construction Surface, H		Conventional Railway	High-Speed Railway
Conventional Railway	High-Speed Railway		
$H < 5.0$ m	$H < 3.0$ m	1:1.5	1:1.8
$5.0 \text{ m} < H < 10.0$ m	$3.0 \text{ m} < H < 9.0$ m	1:1.8	1:1.8
$10.0 \text{ m} < H < 15.0$ m	$9.0 \text{ m} < H < 15.0$ m	1:2.0	1:2.0
$H > 15.0$ m	$H > 15.0$ m	1:2.3	1:2.3

history characteristics may differ depending on the horizontal component direction, even for records obtained from the same earthquake event.

To derive seismic fragility functions, it is necessary to obtain embankment responses over various seismic intensity levels and to include analysis results reaching the defined damage states. Accordingly, each original ground motion was used without scaling and was additionally scaled by factors of 1.5 and 2.0 to construct input motions covering different seismic intensity levels. As a result, a total of 120 input ground motions were constructed. The scaling was performed using a constant amplitude scaling method, which can reflect various seismic intensity levels while preserving the time-history and frequency characteristics of the real recorded ground motions.

Regarding the selection of the scaling factors, Watson-Lamprey and Abrahamson [29] noted that commonly recommended limits on ground motion scaling typically range from factors of approximately 2 to 4. In addition, Bommer and Acevedo [30] summarized previous recommendations indicating that records requiring scaling factors of 4 or greater may be rejected, and that a more conservative scaling limit of approximately 2 has been suggested for some analyses in which nonlinear ground response is important. Based on these previous studies, the maximum scaling factor in this study was conservatively limited to 2.0 to minimize possible distortion of the characteristics of the original ground motions due to excessive scaling. In addition, a scaling factor of 1.5 was adopted to provide an intermediate seismic intensity level between the unscaled motions and the maximum scaling case.

The constructed input ground motions include both Korean recorded motions and real earthquake records from the NGA-West2 database, allowing various ground motion characteristics to be reflected in the analyses. The peak ground acceleration (PGA) of the selected ground motions ranges from 0.05 g to 1.03 g for the records observed in Korea, and from 0.13 g to 2.06 g for the records obtained from the NGA-West2 database. The collected ground motion records were converted in terms of units and defined as time-history loading for use as analysis input. The constructed input ground motions were then applied in dynamic time-history analyses to evaluate the dynamic response of the embankment under different ground conditions.

As representative examples, acceleration time-history plots of two earthquake records observed in Korea (Gyeongju_YOCB_EW and Pohang_PHA2_EW) and three earthquake records from international databases (Kobe_KBU000_EW, Loma Prieta_GIL067_EW, and Northridge_CHL070_EW) used in this study are presented in Fig. 1.

2.3. Numerical analysis model

The height of the underlying ground was set to 40 m. Depending on the bedrock depth defined for each numerical analysis case, the upper portion of the model consisted of soil layers and the lower portion consisted of bedrock.

For the boundary conditions, the same conditions were applied to all numerical analysis cases to ensure consistency of the analysis results. Seismic waves were applied in the horizontal direction. Accordingly, the boundary nodes at the bottom of the model were assigned free conditions in the horizontal direction and fixed conditions in the vertical direction. Through this configuration, the transmission of seismic loading was allowed while unrealistic behavior associated with gravity-induced vertical displacements was restricted. A Lysmer and Kuhlemeyer [31] dashpot was installed at the base of the model. Since applying only a fixed boundary condition at the base can cause seismic waves to reflect at the boundary and produce unrealistic responses, the dashpot was incorporated to dissipate energy at the base boundary, thereby enabling the representation of behavior close to actual subsurface boundary conditions. The dashpot coefficient was determined as the product of the mass density and shear wave velocity of the underlying bedrock layer, following the method of Joyner and Chen [32]. In addition, for the left and right lateral boundaries, nodes located at the same depth were

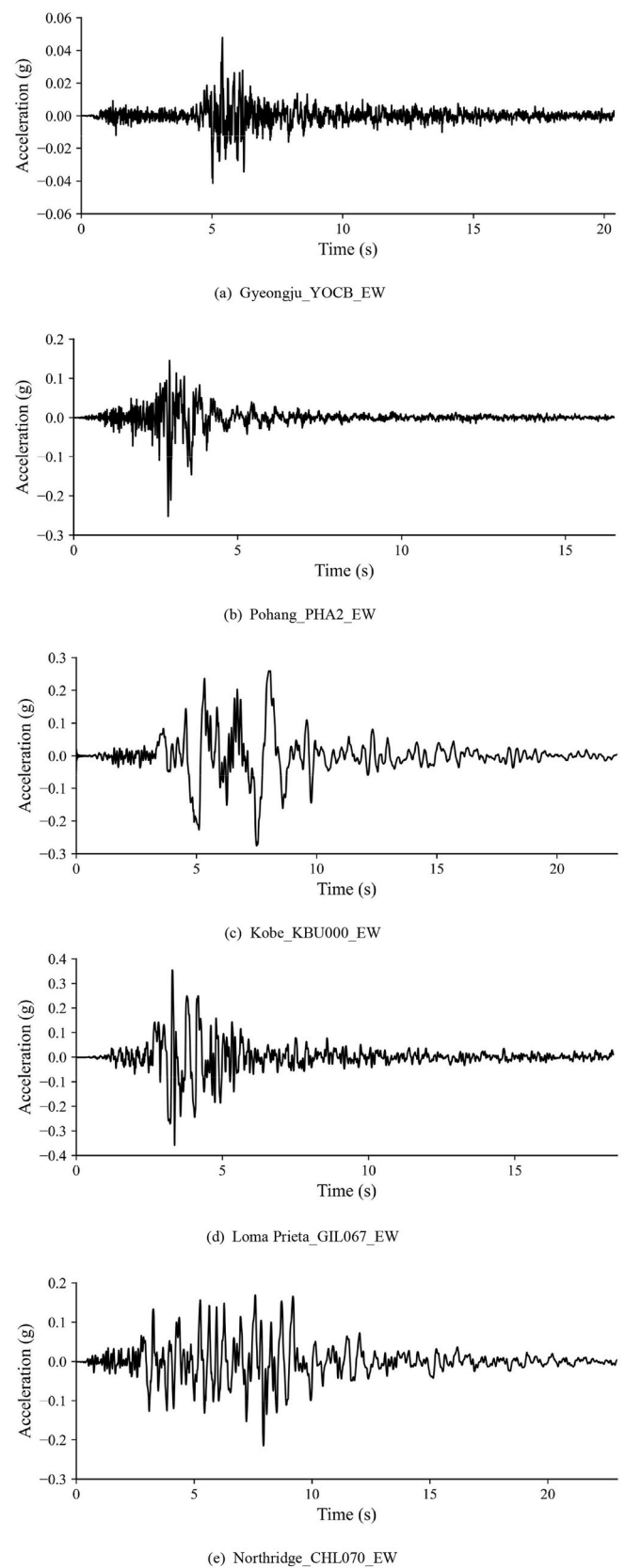


Fig. 1. Representative acceleration time histories of input ground motions.

connected using the equal degrees of freedom [33] constraint, assigning identical horizontal and vertical degrees of freedom. Through this approach, unrealistic deformations that could occur at the lateral boundaries were suppressed, thereby enabling the representation of behavior close to infinite ground conditions within a finite numerical domain. For all other internal nodes, free conditions were assigned in both the horizontal and vertical directions to allow the ground and structural responses to develop adequately under seismic loading.

To quantitatively examine the effect of the boundary conditions on the analysis results, a sensitivity analysis was performed for Case 15, representing the softest ground condition. In this analysis, the representative Kobe_KBU000_EW ground motion record was used, and the model width was varied as 60 m, 100 m, 140 m, 180 m, and 220 m to compare the permanent vertical displacement at the central point of the embankment crest. The sensitivity analysis results are presented in Table 4. The results confirmed that the permanent vertical displacement converged at model widths of 140 m or greater. However, a model width of 180 m was adopted for conservative design. Through this sensitivity analysis, a sufficient model width was secured to minimize the effects of reflected waves at the lateral boundaries. Therefore, the influence of the adopted boundary conditions and model width on the embankment crest response was found to be limited within the considered analysis conditions. The modeled analysis cross-section is shown in Fig. 2.

All numerical analysis cross-sections were discretized using four-node, two-degree-of-freedom quadrilateral elements under plane-strain conditions for the embankment, underlying soil layer, and bedrock layer. The element size (h) was determined to prevent unrealistic seismic wave responses by first calculating the wavelength based on the shear wave velocity (V_s) and the target maximum frequency ($f_{\max}$), as expressed in Equation (1), and then defining the element size according to the criterion proposed by Kuhlemeyer and Lysmer [34], as given in Equation (2). The target maximum frequency ($f_{\max}$) was set to 10 Hz, and for a conservative design, the minimum shear wave velocity of 160 m/s was considered, resulting in a maximum allowable element size of 1.6 m. Accordingly, an element size of $1 \text{ m} \times 1 \text{ m}$ was adopted for all numerical analysis cases.

$$\lambda = V_s / f_{\max} \quad (1)$$

$$h \leq \lambda / 10 \quad (2)$$

An appropriate model was adopted to account for the nonlinear dynamic behavior of the soil material. In this study, both the embankment and the underlying soil layer were assumed to consist of a uniform sandy soil, and the PressureDependMultiYield (PDMY) model [35] was adopted. The PDMY model is a nonlinear model that accounts for stiffness variation and yielding behavior associated with confining pressure, and it can effectively simulate the nonlinear behavior of sandy soils under seismic loading.

The primary input parameters of the PDMY model include density, shear modulus, bulk modulus, and internal friction angle. Among these, the shear modulus and bulk modulus were calculated and applied based on the shear wave velocity and Poisson's ratio of the soil layer defined for each numerical analysis case. This approach allowed the stiffness variations of the ground associated with changes in shear wave velocity to be appropriately reflected. The remaining parameters related to shear and volumetric behavior were assigned by considering the relative

density characteristics of the soil. The underlying soil layers under the S_1 and S_2 conditions were classified as dense sand, the embankment and the underlying ground under the S_4 condition as medium-dense sand, the underlying ground under the S_3 condition as medium sand, and the underlying ground under the S_5 condition as loose sand. Accordingly, the parameters related to the shear and volumetric behavior were defined based on the recommended values provided in the OpenSees manual [36]. In particular, among the key parameters defining dilatancy, the Phase Transformation Angle (PTAng) defines the stress state at which the volumetric behavior transitions from contraction to dilation during shearing. Contrac is a parameter defining the rate of shear-induced volume decrease, where a larger value indicates a faster contraction rate. Dilat1 and Dilat2 are parameters defining the rate of shear-induced volume increase, where larger values indicate stronger dilation. The complete set of input properties for the sandy soil used in this study is summarized in Table 5.

The underlying bedrock layer was modeled using the linear elastic ElasticIsotropic model [36], as it was considered to have sufficiently high stiffness such that deformations induced by seismic loading would be relatively small, and nonlinear behavior was therefore neglected. Elastic modulus, Poisson's ratio, and density corresponding to a shear wave velocity of 800 m/s were applied, and the same properties were used for all numerical analysis cases. The material properties of the bedrock adopted in this study are summarized in Table 6.

In addition, Rayleigh damping was applied during the dynamic analysis to account for the energy dissipation characteristics of the ground material. Since the PDMY constitutive model directly simulates hysteretic damping in the medium-to-large strain range through elastoplastic behavior, Rayleigh damping serves a supplementary role in providing energy dissipation at low strain levels. A damping ratio of 5% was applied to the underlying soil layer and the embankment based on previous studies [14], while a lower damping ratio of 0.5% was assigned to the underlying bedrock to reflect its relatively higher stiffness. To this end, the underlying soil layer, embankment, and bedrock were defined as separate domains, and the first natural frequency of each domain was estimated using the shear wave velocity and layer thickness. Subsequently, the first natural frequency of each domain and a frequency equal to five times this value were selected as the reference frequencies for calculating the Rayleigh damping coefficients, which were then applied to each domain to reflect damping characteristics corresponding to the ground conditions.

2.4. Verification of the numerical model

To verify the validity of the numerical model developed in this study, the results of OpenSees analyses conducted under free-field conditions were compared with those obtained from DeepSoil [37], a widely used program for one-dimensional free-field site response analysis. Through this comparison, the ability of the ground modeling approach and boundary condition settings adopted in this study to reasonably simulate the dynamic ground response was examined.

For the free-field time-history analyses conducted for verification, three ground motions, namely Gyeongju_YOCB_EW, Pohang_PHA2_EW, and Kobe_KBU000_EW, were used as input motions. The free-field analyses were performed considering only the underlying ground, excluding the embankment. The underlying ground conditions consisted of a soil layer with a shear wave velocity of 330 m/s and a thickness of 13 m, underlain by a bedrock layer with a thickness of 27 m. These ground conditions are identical to those applied in Cases 2, 7, and 12 among the numerical analysis cases considered in this study. The OpenSees and DeepSoil results were compared based on the acceleration time-history responses at the ground surface, and the results are presented together in Fig. 3.

A comparison of the acceleration time-history responses under free-field conditions shown in Fig. 3 indicates that, for all three ground motions, the ground-surface acceleration responses obtained from

Table 4
Sensitivity analysis results.

Model Width (m)	Permanent Vertical Displacement (m)
60	0.132
100	0.153
140	0.159
180	0.159
220	0.159

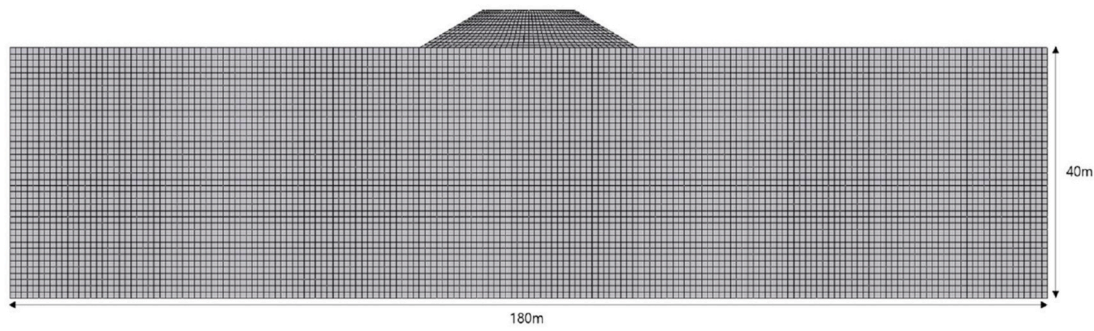


Fig. 2. Numerical analysis cross-section of the railway embankment.

Table 5

Input parameters of sandy soil for the PDMY model.

Parameters	Vs = 160 m/s	Vs = 200 m/s	Vs = 280 m/s	Vs = 290 m/s	Vs = 330 m/s
Density (ton/m ³)	1.7	1.75	1.9	1.91	1.95
Poisson's ratio	0.38	0.35	0.3	0.29	0.27
ShearModulus (kPa)	43,520	70,000	148,960	160,631	212,355
BulkModulus (kPa)	166,826.7	210,000	322,746.7	328,911.1	390,856.3
FrictionAng (°)	30	31	33	33	35
PeakShearStrain	0.1	0.1	0.1	0.1	0.1
PressDependCoe	0.5	0.5	0.5	0.5	0.5
PTAng	29	27	27	27	27
Contrac	0.21	0.07	0.05	0.05	0.03
Dilat1	0	0.4	0.6	0.6	0.8
Dilat2	0	2	3	3	5

Table 6

Input material properties of bedrock.

Parameters	Value
Elastic modulus	3,600,000
Poisson's ratio	0.2
Density	2.34

OpenSees and DeepSoil exhibit similar trends in terms of overall amplitude and waveform shape. In particular, the timing of peak acceleration and the amplification intervals were found to be similar, confirming that the ground response characteristics under free-field conditions were reasonably simulated. These results indicate that the ground modeling approach and boundary conditions adopted in this study appropriately simulate seismic wave propagation and the dynamic response of the ground.

In addition to the acceleration time-history comparisons for the three input ground motions, Fast Fourier Transform (FFT) analyses were performed for the same three input ground motions to further examine the frequency-domain response characteristics. The comparisons of the FFT amplitudes of the free-field ground-surface acceleration responses obtained from OpenSees and DeepSoil are presented in Fig. 4.

The FFT analysis results shown in Fig. 4 indicate that the dominant frequency components and overall FFT amplitude trends obtained from OpenSees and DeepSoil were reasonably similar in the major frequency range for the three input ground motions. Although some discrepancies were observed in the higher-frequency range, these differences may be attributed to differences in the analysis procedures and damping treatment adopted in OpenSees and DeepSoil. These frequency-domain comparison results further support that the OpenSees model can reasonably simulate the free-field dynamic ground response.

Moreover, the permanent vertical displacement induced by earthquake loading in sandy soils is closely associated with the accumulation of volumetric strain caused by cyclic shear deformation [38]. In this study, cyclic shear deformation is also induced in the embankment and underlying ground by horizontal seismic loading, and the volumetric

deformation accumulated through this process appears as permanent vertical displacement at the embankment crest. The permanent vertical displacement can be expressed by accumulating the volumetric strain over the deforming soil layer, as shown in Equation (3).

$$\delta_v = \int_0^H \varepsilon_v(\gamma_{cyc}, z) dz \quad (3)$$

where δ_v is the permanent vertical displacement, $\varepsilon_v(\gamma_{cyc}, z)$ is the volumetric strain induced by cyclic shear strain at depth z , γ_{cyc} is the cyclic shear strain, and H is the thickness of the deforming soil layer. This relationship indicates that the permanent vertical displacement is directly related to the accumulation of volumetric strain induced by cyclic shear deformation. Therefore, the reasonable agreement of the free-field ground-surface acceleration time-history and frequency-domain responses with the DeepSoil results indicates that the seismic wave propagation process and horizontal ground response were reasonably simulated, which suggests that the dynamic response characteristics required for evaluating permanent vertical displacement were appropriately reflected in the numerical model. Based on these verification results, the same numerical modeling approach was applied in this study to perform dynamic time-history analyses considering the interaction between the embankment and the underlying ground.

3. Dynamic analysis results

For all defined analysis cases, gravity analyses were first conducted to establish the initial stress distributions in the underlying ground and the embankment, followed by dynamic analyses using the input ground motions. The gravity analysis was performed by first establishing the initial stress distribution through an elastic gravity analysis in which unit weights were applied vertically to each element, followed by an additional plastic gravity analysis to transition the model to nonlinear material behavior. Based on the established initial stress distribution, dynamic time-history analyses were conducted, and the acceleration response and permanent vertical displacement of the embankment were extracted as the primary dynamic response indices.

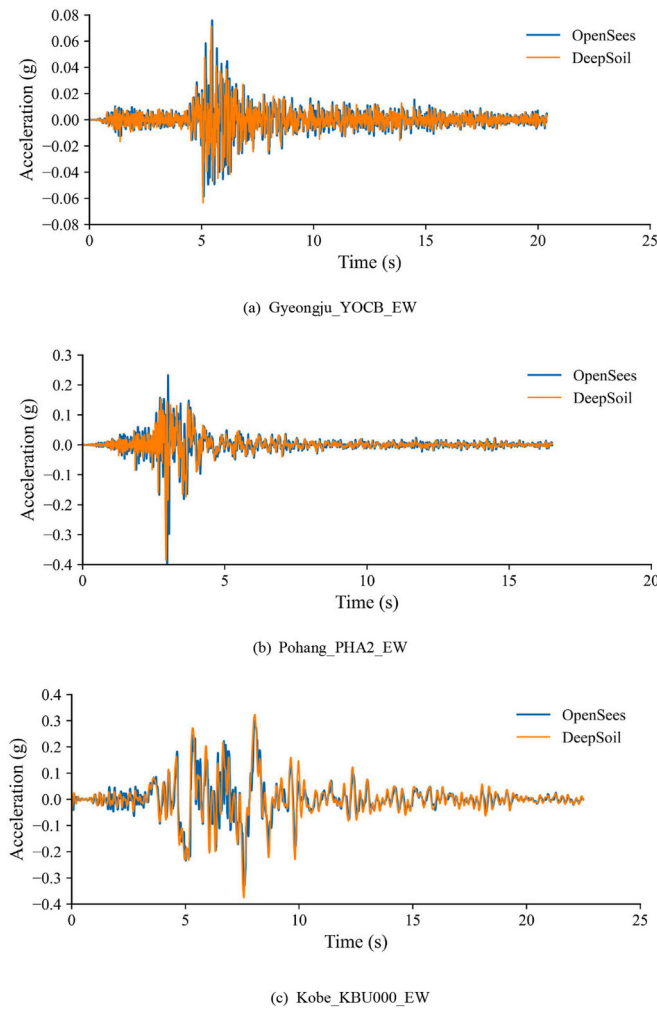


Fig. 3. Comparison of free-field ground-surface acceleration responses from OpenSees and DeepSoil.

3.1. Acceleration responses

To analyze the dynamic behavior characteristics of the embankment, the acceleration response at the central point on the crest of the embankment was examined. The acceleration response is a key response indicator that reflects the magnitude of inertial forces transmitted to the embankment during seismic loading, as well as the amplification characteristics of the underlying ground.

As a representative example, Fig. 5 presents the acceleration time-history responses at the central point on the crest of the embankment obtained by applying the Kobe_KBU000_EW ground motion to Cases 11 and 15, which have identical embankment heights but differ in bedrock depth and underlying soil conditions. Cases 11 and 15 were selected to compare the acceleration response characteristics at the crest of the embankment associated with differences in subsurface conditions, where Case 11 has the shallowest bedrock depth and the highest stiffness of the underlying soil layer, whereas Case 15 has the deepest bedrock depth and the lowest stiffness of the underlying soil layer.

In Case 11, the peak acceleration at the crest of the embankment reached 0.344 g compared to the peak acceleration of the input ground motion (0.276 g), indicating an amplification of approximately 25%. In Case 15, the peak acceleration increased to 0.445 g, corresponding to an amplification of approximately 61%, demonstrating a pronounced amplification effect. These results are considered to reflect the ground amplification characteristics associated with increasing bedrock depth and decreasing stiffness of the underlying soil layer, which are

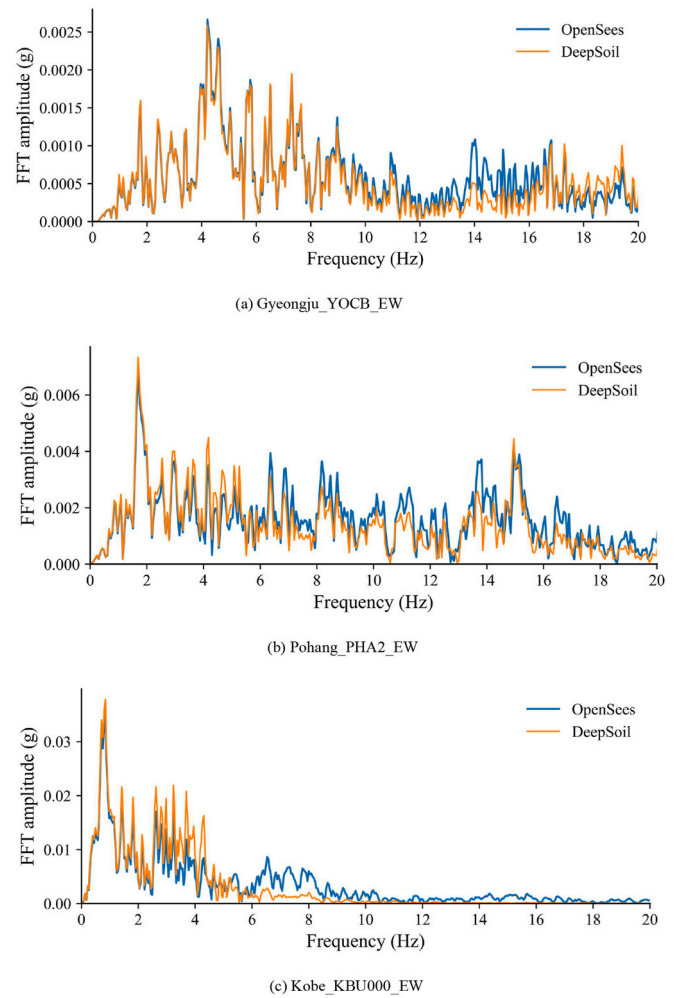


Fig. 4. FFT amplitude comparison of free-field responses for three input ground motions.

manifested in the acceleration response.

Meanwhile, a comparison of the acceleration responses for different site classes S_3 (Case 13), S_4 (Case 14), and S_5 (Case 15) under the same embankment height condition showed that the acceleration responses varied depending on the underlying ground conditions. In particular, the difference in acceleration response between S_4 and S_5 , which are both classified as SD sites, was found to be relatively small.

3.2. Permanent vertical displacement responses

The permanent vertical displacement at the central point on the crest of the embankment was extracted as the primary response parameter to evaluate the behavior induced by seismic loading. In this study, the permanent vertical displacement was defined as the difference between the displacement immediately after the gravity analysis and that at the end of the dynamic analysis, thereby allowing the displacement induced solely by seismic loading to be evaluated. Downward vertical displacement was defined as negative.

As a representative example, Fig. 6 presents the time-history responses of vertical displacement at the central point on the embankment crest obtained by applying the Kobe_KBU000_EW ground motion to Cases 11–15 with different underlying ground conditions. The vertical displacement responses shown in Fig. 6 represent the seismic-induced displacement after excluding the initial displacement caused by the gravity analysis. From these time-history responses, the permanent vertical displacement remaining at the embankment crest after the end

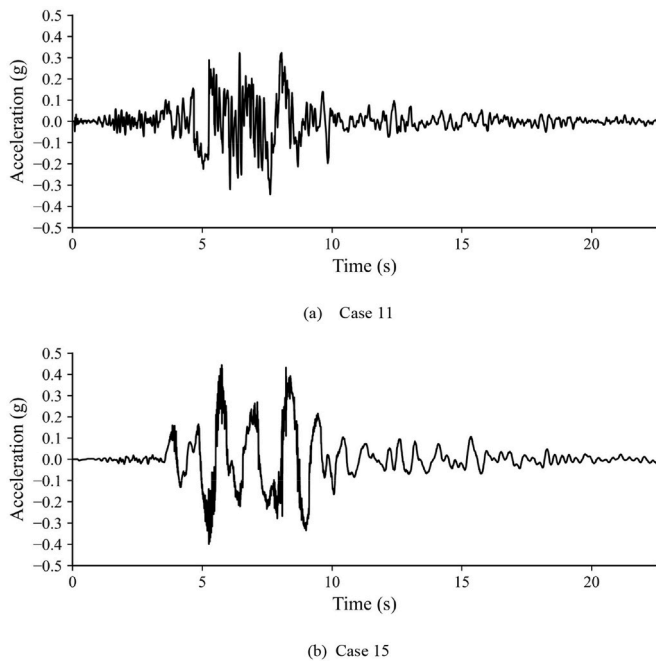


Fig. 5. Acceleration time-history responses at the embankment crest under the Kobe_KBU000_EW ground motion (Cases 11 and 15).

of seismic excitation was evaluated, and the magnitude of permanent vertical displacement was found to vary depending on the underlying ground conditions across the different cases.

Under conditions with a shallow bedrock depth and high stiffness of the underlying soil layer, the permanent vertical displacement at the embankment crest after the earthquake was relatively small. In contrast, under conditions with increased bedrock depth and reduced shear wave velocity of the underlying soil layer, displacement accumulation continued during seismic loading, resulting in larger permanent vertical displacements. These differences reflect variations in ground deformation characteristics resulting from the increased thickness and reduced stiffness of the underlying soil layer.

In particular, a comparison of the permanent vertical displacement responses for S_3 (Case 13), S_4 (Case 14), and S_5 (Case 15) showed that the difference between S_3 and S_4 was relatively small, whereas a pronounced difference was observed between S_4 and S_5 , which are both

classified as SD sites according to the V_{s30} -based site classification criteria. This result differs from the trend observed in the acceleration responses, in which the difference between S_4 and S_5 was relatively small despite both being classified as SD sites.

This difference can be interpreted as resulting from the fact that, unlike acceleration responses, permanent vertical displacement responses directly reflect the nonlinear behavior and cumulative deformation characteristics of the ground. As demonstrated above, the permanent vertical displacement response can be used as an indicator reflecting the damage level of embankment structures, and it was confirmed that substantial differences in response can occur even within the same V_{s30} -based site classification. The permanent vertical displacement derived in this study is used as a damage index for the seismic fragility analysis of the embankment. In particular, under conditions where bedrock is relatively shallow, there is a need to develop seismic fragility functions based on site classification criteria that can adequately reflect these damage characteristics.

4. Development of seismic fragility functions

A seismic fragility function expresses the probability that a structure reaches or exceeds a specified damage state during an earthquake as a function of seismic intensity. Seismic fragility functions are developed by comprehensively accounting for various sources of uncertainty, including seismic intensity measures, structural characteristics, and ground conditions, and are used for seismic risk assessment and evaluation of structural seismic performance. Through this approach, the seismic performance of structures can be quantitatively evaluated probabilistically, enabling reasonable prediction of the expected damage levels and potential losses during earthquakes.

Under ground conditions with shallow bedrock, it was observed that the damage characteristics of embankment structures can differ even when the sites are classified under the same V_{s30} -based site classification criteria. Accordingly, this study applied site classification criteria that reflect ground conditions with shallow bedrock and presented the corresponding seismic fragility functions.

For the derivation of the seismic fragility functions, this study adopted the permanent vertical displacement at the central point on the crest of the embankment, obtained from dynamic numerical analysis results, as the damage index. As the seismic intensity measure, the PGA evaluated under free-field conditions was used. The free-field PGA reflects the intrinsic dynamic characteristics of the ground by excluding the influence of the interaction between the embankment and the

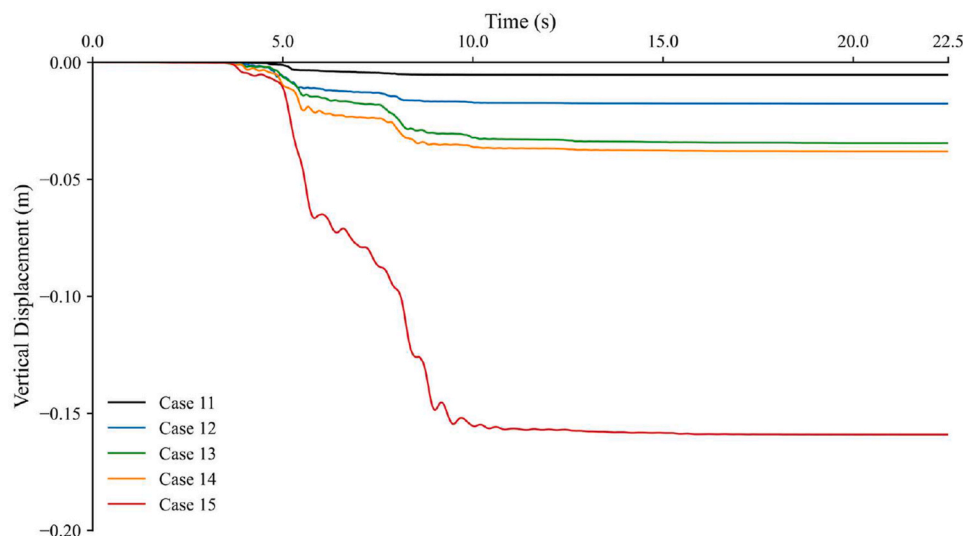


Fig. 6. Example of vertical displacement time-history responses of an embankment structure (Cases 11–15).

underlying ground, and can be considered a suitable intensity measure for this study, which aims to compare seismic fragility across multiple numerical analysis cases with varying ground conditions. In addition, PGA has been consistently adopted as the intensity measure in previous seismic fragility studies of embankment structures [17,19], enabling direct comparability with existing fragility functions. For each input ground motion, the PGA was first calculated under free-field conditions, after which dynamic time-history analyses were performed using a numerical model that accounts for the interaction between the embankment and the underlying ground. From the analysis results, the permanent vertical displacement at the central point on the crest of the embankment was extracted, and based on this response, the damage states and damage index were defined. Finally, the analysis results corresponding to each seismic intensity measure were evaluated using linear regression to estimate the mean and standard deviation, based on which seismic fragility functions following a lognormal cumulative distribution were derived.

4.1. Definition of damage index and damage thresholds

For the derivation of seismic fragility functions, it is necessary to define a damage index that can distinguish damage states, along with threshold values corresponding to each damage state. In this study, the damage states of embankment structures were classified into three categories, namely minor damage, moderate damage, and extensive damage. The damage index was defined as the permanent vertical displacement (PVD) at the central point on the crest of the embankment obtained from the dynamic analysis, normalized by a reference displacement, and was calculated as shown in Equation (4).

$$DI = PVD / 0.01 \quad (4)$$

The reference displacement of 0.01 m corresponds to the minimum threshold of the caution level among the vertical alignment irregularity management stages for high-speed railway specified in the Track Maintenance Guidelines [39], and was used as the reference displacement for calculating the damage index. The lower bound of the caution level was adopted as a conservative reference value because even relatively small permanent settlements can adversely affect track alignment and operational safety in railway systems. Therefore, the reference displacement was defined to reflect the onset of serviceability-related performance degradation rather than severe structural damage. Although the damage thresholds were defined based on the Track Maintenance Guidelines, the PVD used as the damage index is directly related to the operational safety and speed restriction criteria of high-speed railway embankments and therefore has a widely accepted and internationally relevant significance in seismic fragility analysis. This relationship is particularly relevant for the concrete slab track system considered in this study. Unlike ballasted track, where the ballast layer can partially redistribute differential settlements, the rigid slab track transmits PVD at the embankment crest directly to the track level with minimal attenuation. Accordingly, the PVD at the embankment crest can be considered a reasonable approximation of the resulting vertical track irregularity for the slab track system. Furthermore, since the PVD evaluated in this study represents the absolute permanent displacement at a single measurement point rather than the relative differential settlement between two points along the track, the use of absolute PVD as the damage index provides a conservative estimate of track irregularity. This conservative approach is appropriate for a seismic fragility assessment and is consistent with previous studies that adopted PVD as the primary damage index for embankment fragility analysis [17,19]. A more rigorous evaluation explicitly accounting for differential track irregularity along the longitudinal direction will be pursued in future studies incorporating three-dimensional modeling of the embankment-track system.

The threshold values corresponding to each damage state were defined with reference to the vertical alignment irregularity

management stages for high-speed railway specified in the Track Maintenance Guidelines. The damage thresholds for the minor, moderate, and extensive damage states were determined according to the caution, maintenance, and speed restriction levels, respectively, of the high-speed railway vertical alignment irregularity management stages specified in the Track Maintenance Guidelines. The threshold values of vertical alignment irregularity corresponding to the management stages for high-speed railway specified in the Track Maintenance Guidelines are summarized in Table 7.

In the Track Maintenance Guidelines, an upper threshold value corresponding to the speed-restriction level is not explicitly specified. Accordingly, to clearly distinguish the damage states, the upper threshold of the extensive damage state was defined as 300 mm by considering an existing damage classification in which displacements exceeding 300 mm are classified as a collapse state [40]. Based on the high-speed railway vertical alignment irregularity management stages and the defined damage states, the threshold values of vertical alignment irregularity corresponding to each damage state were converted into damage index values. The classification of damage states according to the converted damage index ranges is presented in Table 8.

4.2. Derivation of seismic fragility functions

Based on the defined damage index and damage thresholds, seismic fragility functions for embankment structures were derived in this study. The seismic fragility functions were formulated as a standard lognormal cumulative distribution function, as shown in Equation (5).

$$P_f(ds \geq ds_i | S) = \Phi \left(\frac{\ln(IM/IM_{mi})}{\beta_{tot}} \right) \quad (5)$$

In this formulation, P_f denotes the probability of exceeding a given damage state ds_i at a specified seismic intensity measure, and $\Phi(\cdot)$ represents the standard normal cumulative distribution function. IM_{mi} denotes the median value of the seismic intensity measure corresponding to the onset of the i -th damage state, and β_{tot} represents the logarithmic standard deviation of the lognormal distribution.

The median seismic intensity IM_{mi} for each damage state was estimated from dynamic time-history analyses using the 120 input ground motions considered in this study. For each input ground motion, the seismic intensity measure and the damage index were transformed into logarithmic form, and the relationship between the two variables was quantified through linear regression analysis. Subsequently, the damage index thresholds defined for each damage state were substituted into the regression equation to estimate the seismic intensity levels corresponding to each damage state, from which the median values for the damage states were derived.

Fig. 7 presents an example of the distribution of the damage index as a function of seismic intensity level, along with the results of linear regression. The dashed line in the figure represents the trend line derived using linear regression, while the data points indicate the analysis results for individual input ground motions. For some damage states, such as the extensive damage state, where the analysis results did not reach the damage index threshold, the median value for the corresponding damage state was estimated by extrapolation based on the linear relationship between the damage index and the seismic intensity

Table 7

Threshold values of vertical alignment irregularity according to management stages.

Management Stage	Vertical Alignment Irregularity Threshold (mm)
Caution level	$10 \leq N_{all} < 18$
Maintenance level	$N_{all} \geq 18$
Speed restriction level (speed limit = 170 km/h)	$N_{all} \geq 30$

Table 8

Damage index corresponding to damage states.

Damage State	Management Stage	Damage Index (DI)
Minor	Caution level	$1 \leq DI < 1.8$
Moderate	Maintenance level	$1.8 \leq DI < 3$
Extensive	Speed restriction level (speed limit = 170 km/h)	$3 \leq DI < 30$

measure. Through this procedure, fragility functions were derived for all defined damage states, and seismic fragility was compared across all numerical analysis cases with different bedrock depths, shear wave velocities of the underlying soil layer, and embankment heights using the same damage state criteria.

The logarithmic standard deviation of the seismic fragility function, β_{tot} , was estimated by considering the total variability of the seismic response. The total variability consists of the uncertainty in structural response due to ground motion variability (β_D), the uncertainty associated with damage state definition (β_{ds}), and the uncertainty in the evaluation of the seismic capacity of the structure (β_c), and is defined as shown in Equation (6).

$$\beta_{tot} = \sqrt{\beta_D^2 + \beta_{ds}^2 + \beta_c^2} \quad (6)$$

β_D was estimated by dividing the standard deviation of the residuals of $\ln$ (DI) from the linear regression between $\ln$ (PGA) and $\ln$ (DI) by the regression slope, thereby converting the uncertainty from the $\ln$ (DI) space to the $\ln$ (PGA) space. In contrast, β_{ds} was set to 0.4 based on values commonly adopted in previous fragility studies and relevant guidelines [41], while β_c was assigned a value of 0.3 based on engineering judgment [17]. The median seismic intensity values, IM_{mi} , and the lognormal standard deviation, β_{tot} , estimated for each damage state were applied to derive the seismic fragility functions for each case, and the results are presented in Table 9.

4.3. Analysis of seismic fragility results

The seismic fragility of embankment structures is determined by the combined effects of embankment geometry and the characteristics of the underlying ground. In this study, seismic fragility responses were examined with respect to bedrock depth using a site classification approach that accounts for shallow bedrock conditions, and comparisons were further performed for variations in the shear wave velocity of the underlying soil layer under the same bedrock depth conditions. In addition, seismic fragility responses were compared for different embankment heights under identical underlying ground conditions to evaluate the influence of structural geometry.

The variation in seismic fragility associated with each variable was compared using representative cases, and seismic fragility comparisons were conducted based on the median values of the fragility functions. The median represents the seismic intensity at which the probability of reaching a given damage state equals 50%.

4.3.1. Effect of bedrock depth on seismic fragility

In this study, bedrock depth is a key parameter that determines the thickness of the underlying soil layer and influences the dynamic behavior of the ground under shallow bedrock conditions. To compare the effect of bedrock depth on the seismic fragility of embankment structures, two cases with different bedrock depths were compared while keeping the shear wave velocity of the underlying soil layer and the embankment height constant. Fig. 8 presents representative seismic fragility functions for Case 6, with a bedrock depth of 1 m, and Case 7, with a bedrock depth of 13 m, under the embankment height condition of 6 m.

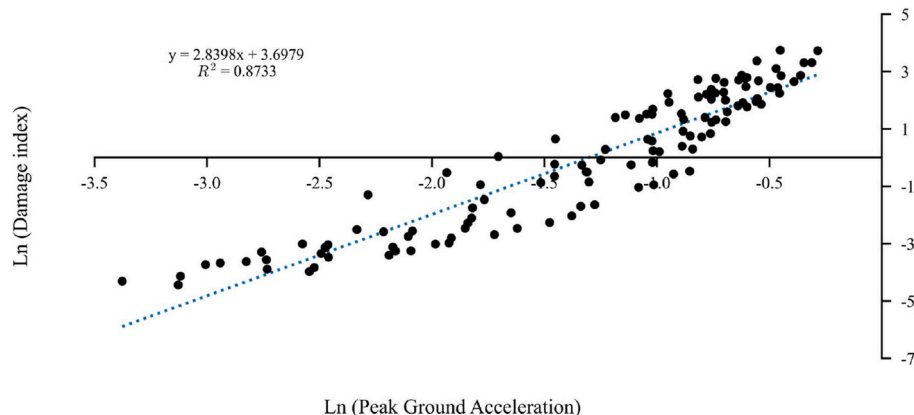
For the moderate damage state, the median PGA was 1.374 g for Case 6 and 0.628 g for Case 7. As the bedrock depth increased from 1 m to 13 m, the median PGA required to reach the same damage level decreased by approximately 54%. A similar decreasing trend was observed for the minor damage state, with median PGA values of 0.971 g for Case 6 and 0.507 g for Case 7. For the extensive damage state, a more pronounced reduction was observed, with the median PGA decreasing from 4.742 g to 1.352 g.

These results can be interpreted as indicating that, as bedrock depth increases, the thickness of the underlying soil layer increases, leading to

Table 9

Parameters of seismic fragility functions for each analysis case.

Case No.	Median Seismic Intensity (IM_{mi})			Lognormal Standard Deviation (β_{tot})		
	Minor	Moderate	Extensive	Minor	Moderate	Extensive
1	2.639	3.874	15.299	0.850	0.850	0.850
2	0.698	0.918	2.446	0.607	0.607	0.607
3	0.532	0.626	1.119	0.565	0.565	0.565
4	0.493	0.602	1.229	0.560	0.560	0.560
5	0.330	0.394	0.742	0.572	0.572	0.572
6	0.971	1.374	4.742	0.696	0.696	0.696
7	0.507	0.628	1.352	0.594	0.594	0.594
8	0.467	0.549	0.984	0.572	0.572	0.572
9	0.408	0.494	0.982	0.559	0.559	0.559
10	0.318	0.383	0.745	0.580	0.580	0.580
11	0.697	0.967	3.117	0.667	0.667	0.667
12	0.447	0.547	1.134	0.589	0.589	0.589
13	0.432	0.509	0.919	0.574	0.574	0.574
14	0.377	0.453	0.873	0.560	0.560	0.560
15	0.306	0.370	0.729	0.581	0.581	0.581

**Fig. 7.** Example of linear regression results (Case 15).

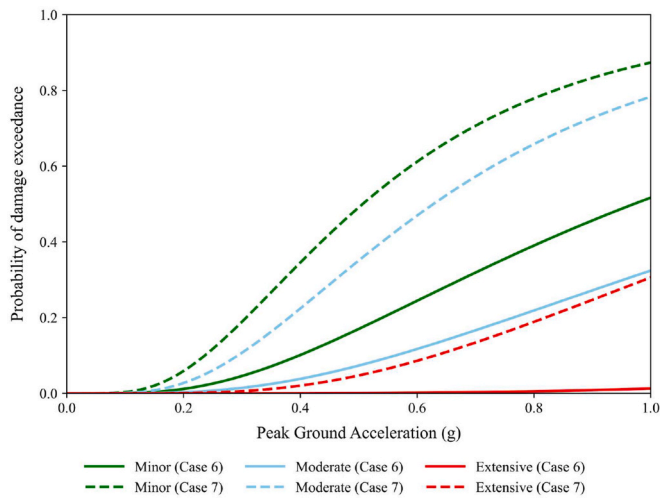


Fig. 8. Seismic fragility functions for Cases 6 and 7 (Shear wave velocity = 330 m/s, embankment height = 6 m; Case 6: bedrock depth = 1 m, Case 7: bedrock depth = 13 m).

corresponding changes in the dynamic characteristics of the ground. As the thickness of the soil layer increases, the fundamental frequency of the ground decreases, which in turn increases the likelihood of resonance with low-frequency components contained in the input ground motions. As a result, the potential for amplification of seismic response in the underlying ground increases, and the amplified response is transmitted to the embankment crest, leading to an increased likelihood of damage to the embankment structure.

A consistent decreasing trend in the median PGA with increasing bedrock depth was also observed for embankment heights of 3 m and 8 m, indicating that bedrock depth is a ground condition parameter that can reasonably explain differences in the seismic fragility response of embankment structures under shallow bedrock conditions.

4.3.2. Effect of shear wave velocity of the underlying soil layer on seismic fragility

The shear wave velocity of the soil layer is a material property that represents ground stiffness and exerts a dominant influence on ground amplification characteristics. To evaluate the effect of changes in soil layer stiffness associated with variations in shear wave velocity on the seismic fragility of embankment structures, two cases with different shear wave velocities were compared while fixing the bedrock depth at 28 m and keeping the embankment height constant. Under the embankment height condition of 6 m, Case 9 with a shear wave velocity of 280 m/s and Case 10 with a shear wave velocity of 160 m/s were selected as representative cases, and the seismic fragility functions for the two cases are presented in Fig. 9.

For the moderate damage state, the median PGA was 0.494 g for Case 9 and 0.383 g for Case 10, indicating an approximate 22% reduction in the median PGA as the shear wave velocity decreased. For the minor damage state, the median PGA decreased from 0.408 g to 0.318 g, and a similar decreasing trend was observed for the extensive damage state, with the median PGA reducing from 0.982 g to 0.745 g.

This trend can be interpreted as a result of changes in ground response characteristics caused by a reduction in ground stiffness associated with decreasing shear wave velocity. As a result, the seismic response developed within the underlying soil layer is transmitted to the embankment crest, leading to a relatively increased likelihood of damage to the embankment structure.

Even under the same bedrock depth condition, a consistent increase in seismic fragility with decreasing shear wave velocity was observed for embankment heights of 3 m and 8 m. This confirms that the shear wave velocity of the soil layer is also a key dynamic ground parameter

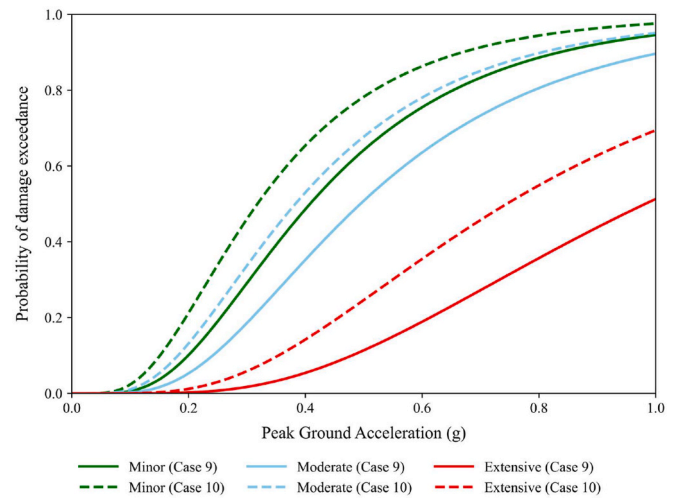


Fig. 9. Seismic fragility functions for Cases 9 and 10 (Bedrock depth = 28 m, embankment height = 6 m; Case 9: shear wave velocity = 280 m/s, Case 10: shear wave velocity = 160 m/s).

influencing the seismic fragility of embankment structures.

4.3.3. Effect of embankment height on seismic fragility

When the underlying ground conditions are identical, embankment height is a key parameter that determines the mass and geometry of the embankment structure and directly influences the magnitude of inertial forces generated during seismic loading. To evaluate the effect of embankment height on seismic fragility, cases with embankment heights of 3 m, 6 m, and 8 m were compared while keeping the underlying ground conditions identical. As representative examples, Cases 2, 7, and 12 corresponding to the S_2 ground condition were compared, and the seismic fragility functions of the three cases are presented in Fig. 10.

For the moderate damage state, the median PGA was 0.918 g for an embankment height of 3 m (Case 2), 0.628 g for 6 m (Case 7), and 0.547 g for 8 m (Case 12). Overall, the median PGA decreases progressively as the embankment height increases, indicating that an increase in embankment height leads to an increase in seismic fragility. For the minor damage state, the median PGA decreased from 0.698 g to 0.507 g and further to 0.447 g. A similar trend was observed for the extensive damage state, with the median PGA decreasing from 2.446 g to 1.352 g

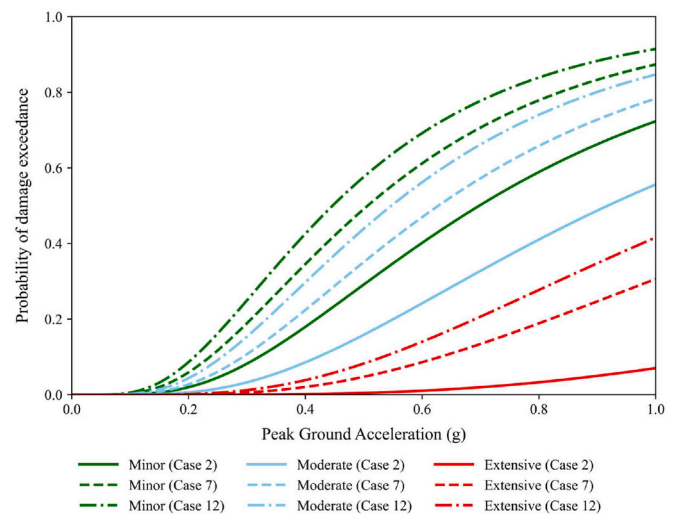


Fig. 10. Seismic fragility functions for Cases 2, 7, and 12 (Shear wave velocity = 330 m/s, bedrock depth = 13 m; Case 2: embankment height = 3 m, Case 7: embankment height = 6 m, Case 12: embankment height = 8 m).

and then to 1.134 g.

This trend can be interpreted as resulting from the increase in inertial forces induced within the embankment during earthquakes due to the increase in structural mass associated with greater embankment height under identical underlying ground conditions. In other words, when ground conditions remain constant, the seismic response of embankment structures tends to be more strongly influenced by the geometry and mass of the structure.

In contrast, under the S_5 condition, which represents the weakest underlying ground condition (Cases 5, 10, and 15), the median PGA for the moderate damage state was 0.394 g, 0.383 g, and 0.370 g, respectively, indicating that the variation in seismic fragility with increasing embankment height was relatively small. This suggests that, under soft ground conditions, the response characteristics of the underlying ground may exert a greater influence on the seismic response of embankment structures than changes in inertial forces associated with embankment height.

4.3.4. Effect of site classification (S_1 – S_5) on seismic fragility

To compare seismic fragility according to the site classification criteria adopted in this study under identical embankment height conditions, the seismic fragility for the moderate damage state was compared using five cases with an embankment height of 8 m. The seismic fragility functions for the moderate damage state of Cases 11 through 15 are presented in Fig. 11.

The comparison results indicate that, as the site classification changes from S_1 to S_5 , the median PGA of the fragility functions decreases progressively. This confirms that, even under identical embankment height conditions, the PGA required to reach the same damage state varies depending on the site classification.

These differences in seismic fragility can be attributed to variations in the underlying ground characteristics reflected in the adopted site classification criteria. A comparison between the S_2 and S_3 site conditions, which share the same bedrock depth, indicates that the seismic fragility of embankment structures increases as the shear wave velocity of the underlying soil layer decreases. This suggests that differences in the stiffness of the underlying soil layer incorporated in the site classification criteria may influence the seismic response of embankment structures and the likelihood of damage occurrence.

In addition, a comparison between the S_1 and S_2 site conditions, which have the same shear wave velocity, shows that seismic fragility

increases with increasing bedrock depth. These results indicate that bedrock depth reflected in the site classification criteria is an important variable influencing the likelihood of damage occurrence in embankment structures. Overall, it can be confirmed that differences in underlying ground conditions according to the site classification criteria applied in this study are effectively reflected in the seismic fragility of embankment structures.

4.3.5. Comparison of seismic fragility functions with previous studies

In the previous study by Argyroudis and Kaynia [17], seismic fragility functions for railway embankments were developed based on the Eurocode 8 site classification criteria, and the results showed that seismic fragility increases as the site classification shifts toward softer ground conditions. Fig. 12 presents the seismic fragility functions for Soil types C and D corresponding to an embankment height of 6 m as reported in a previous study. It is observed that, as the site condition shifts from SC to SD, the median PGA required to reach the same damage state decreases.

For comparison with previous studies, this study examined seismic fragility functions by applying the V_{s30} -based site classification criteria of the IBC, which are widely used internationally. Fig. 13 presents the seismic fragility functions for Case 7, corresponding to the IBC SC site class, and Case 9, corresponding to the IBC SD site class, among the analysis cases considered in this study. The comparison results indicate that seismic fragility under the SD site condition is greater than that under the SC site condition, which is qualitatively consistent with the general trend reported in previous studies that the seismic fragility of embankment structures increases as site classification shifts toward softer ground conditions.

In addition, seismic fragility was compared for Cases 7 and 8, which are both classified as SC site class according to the IBC criteria. Fig. 14 presents the seismic fragility functions for the two cases, and despite being classified under the same site class, the median PGA required to reach the same damage state differs between them. This indicates that, even under the same V_{s30} -based site classification criteria, differences in underlying ground conditions may result in variations in the seismic response and fragility characteristics of embankment structures.

5. Discussion

The analysis results of this study indicate that the seismic fragility of embankment structures is determined not by a single variable, but by the interaction between embankment geometry and the characteristics of the underlying ground. In particular, variations in bedrock depth and shear wave velocity of the soil layer act as key parameters that modify

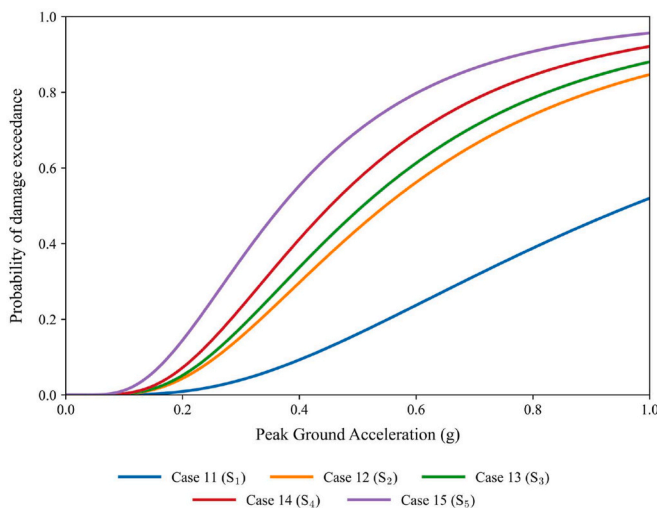


Fig. 11. Seismic fragility functions for Cases 11 through 15 (Embankment height = 8 m; Case 11: bedrock depth = 1 m, V_s = 330 m/s; Case 12: bedrock depth = 13 m, V_s = 330 m/s; Case 13: bedrock depth = 13 m, V_s = 200 m/s; Case 14: bedrock depth = 28 m, V_s = 280 m/s; Case 15: bedrock depth = 28 m, V_s = 160 m/s).

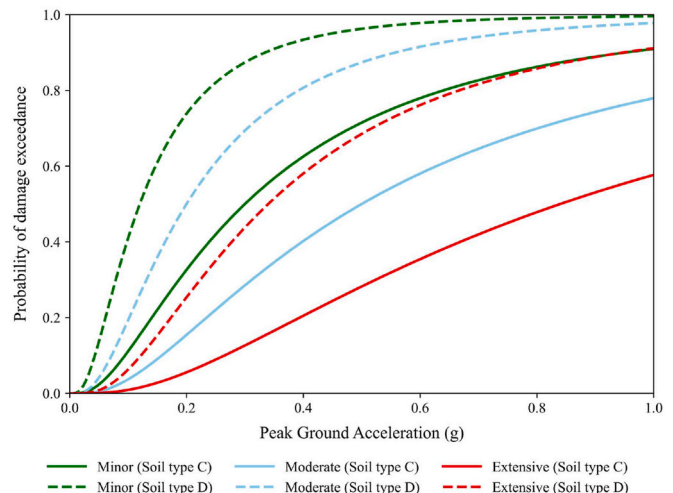


Fig. 12. Seismic fragility functions for Soil types C and D [17].

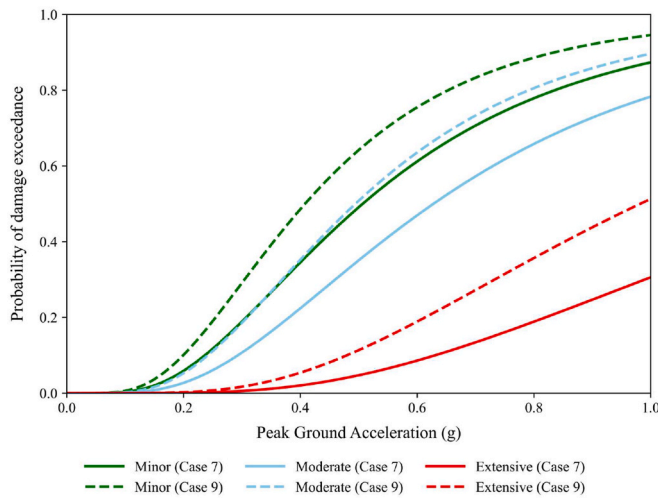


Fig. 13. Seismic fragility functions for Cases 7 and 9 corresponding to IBC SC and SD site classes (Embankment height = 6 m; Case 7: bedrock depth = 13 m, shear wave velocity = 330 m/s; Case 9: bedrock depth = 28 m, shear wave velocity = 280 m/s).

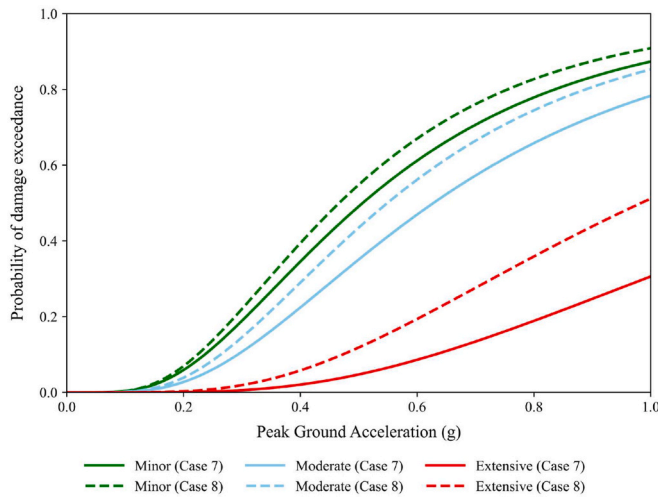


Fig. 14. Seismic fragility functions for Cases 7 and 8 classified as IBC SC (Embankment height = 6 m, shear wave velocity = 330 m/s; Case 7: bedrock depth = 13 m, Case 8: bedrock depth = 13 m, shear wave velocity = 200 m/s).

the thickness and stiffness of the underlying soil layer, thereby altering the fundamental frequency characteristics of the ground. As the bedrock depth increases or the shear wave velocity decreases, the fundamental frequency of the soil layer tends to shift toward the low frequency range, which may increase the potential for resonance with low frequency components of the input ground motions. Such changes in frequency characteristics can be considered as factors influencing the level of seismic response of embankment structures.

These dynamic ground behavior characteristics suggest that, when applying site classification criteria to shallow bedrock conditions, it is necessary to consider both the stiffness and thickness of the soil layer. In particular, under ground conditions where bedrock is relatively shallow, variations in soil layer thickness can directly influence the fundamental frequency and amplification characteristics of the ground, thereby acting as an important factor in determining structural response during earthquakes. From this perspective, site classification criteria that simultaneously consider shear wave velocity and bedrock depth can reasonably reflect ground response characteristics observed under shallow bedrock conditions.

Based on the analysis results considering these ground conditions, the fragility functions derived in this study show a consistent decrease in the median PGA with decreasing shear wave velocity and increasing bedrock depth. This indicates that the seismic intensity required to reach the same damage state is reduced, resulting in increased seismic fragility of embankment structures. This trend can be interpreted as a result of the interaction between the fundamental frequency characteristics of the ground and the frequency components of the input ground motions being reflected in the fragility functions. Accordingly, the dynamic characteristics of the ground can substantially alter the seismic intensity required to induce damage in embankment structures.

When the underlying ground conditions are identical, embankment height is a key variable that determines the mass and geometry of the embankment structure and, consequently, the magnitude of inertial forces acting on the structure during earthquakes. Under the S_2 ground condition, the median PGA exhibits a clear decreasing trend as embankment height increases. This suggests that the increase in structural mass and inertial forces associated with greater embankment height can amplify the structural response and exert a relatively strong influence on seismic fragility. In contrast, under the S_5 ground condition, which represents the weakest soil condition, the reduction in the median PGA with increasing embankment height is relatively small. This implies that, under soft ground conditions, the response characteristics of the underlying ground may have a greater influence on the seismic response than changes in embankment height. These differences indicate that the influence of embankment height may vary depending on ground conditions.

In addition, the fragility functions derived in this study were compared with those reported in a previous study to examine both common trends and distinctive features. Previous studies reported that seismic fragility increases as site classification shifts toward softer ground conditions. A consistent trend was also observed in this study, where the seismic fragility of embankment structures increased with progressively softer ground conditions. This indicates that the fragility functions derived in this study, based on the adopted site classification criteria, reasonably reflect the seismic response characteristics associated with different site classifications. Meanwhile, even within the same V_{s30} -based site classification, the median PGA of the fragility functions differed when site classification criteria that also consider bedrock depth were applied. This suggests that the seismic response of embankment structures on shallow bedrock is influenced by the combined effects of average shear wave velocity and underlying ground conditions, including soil layer thickness.

Overall, the results of this study demonstrate that, under ground conditions characterized by relatively shallow bedrock, site classification criteria that consider bedrock depth in addition to the shear wave velocity of the underlying soil layer are important for the rational assessment of the seismic fragility of embankment structures. By applying site classification criteria that simultaneously consider shear wave velocity and bedrock depth to reflect these ground condition characteristics, it was confirmed that seismic response characteristics associated with the stiffness of the underlying ground and soil layer thickness can be effectively incorporated into the fragility functions. In addition, the seismic response of embankment structures was found to be influenced by both the characteristics of the underlying ground and the increase in mass associated with embankment height, as well as the resulting inertial effects. It was further confirmed that the influence of embankment geometry can vary depending on ground conditions. These results suggest that a fragility assessment that simultaneously accounts for the interaction between underlying ground conditions and embankment geometry can reasonably reflect the actual seismic behavior of embankment structures.

6. Conclusion

In this study, the site classification criteria (S_1 – S_5) specified in the

KDS 17 10 00 were applied to analyze the seismic fragility of railway embankment structures. Through this approach, the effects of underlying ground characteristics and embankment geometry on seismic fragility were evaluated under ground conditions characterized by relatively shallow bedrock. For the seismic fragility analysis, a two-dimensional finite element numerical model under plane-strain conditions was developed using OpenSees, and nonlinear dynamic time-history analyses were performed for a total of 15 analysis cases by considering bedrock depth, shear wave velocity of the underlying soil layer, and embankment height as the primary variables. The PVD at the central point on the embankment crest was defined as the damage index, and seismic fragility functions were derived by establishing damage states and corresponding damage thresholds based on the vertical alignment irregularity management criteria specified in the Track Maintenance Guidelines.

The analysis results indicate that the seismic fragility of embankment structures is determined not by a single variable, but by the interaction between embankment geometry and the characteristics of the underlying ground. Under conditions characterized by greater bedrock depth and lower shear wave velocity of the underlying soil layer, the median PGA required to reach the same damage state consistently decreased, indicating that variations in underlying ground conditions may have a meaningful influence on the seismic fragility of embankment structures. In addition, the influence of embankment height varied depending on ground conditions. Under relatively stiff ground conditions, the increase in inertial forces associated with greater embankment height exerted a stronger influence on seismic fragility, whereas under soft ground conditions, the influence of ground amplification effects appeared relatively more pronounced. These results suggest that the influence of embankment height may vary depending on ground conditions.

In addition, the comparison of seismic fragility according to the applied site classification criteria under the same embankment height condition showed that the median PGA of the fragility functions gradually decreased as the site class changed from S_1 to S_5 . This trend indicates that the site classification criteria considering both shear wave velocity and bedrock depth effectively reflect the seismic response characteristics and differences in seismic fragility of embankment structures. In particular, the present study quantitatively demonstrated that ground conditions classified into the same site class according to the conventional V_{s30} -based site classification criteria can exhibit different median PGA values of the fragility functions when bedrock depth is considered through the S_1 – S_5 site classification criteria. This finding provides a quantitative basis for emphasizing the importance of considering both shear wave velocity and bedrock depth when evaluating the seismic fragility of embankment structures under shallow bedrock conditions.

This study evaluated the seismic fragility of railway embankment structures located on sites characterized by shallow bedrock and analyzed the effects of key parameters, including bedrock depth, shear wave velocity of the soil layer, embankment height, and site classification, on seismic fragility of embankment structures. The analysis results and fragility functions presented in this study can be used to quantitatively evaluate the seismic fragility of embankment structures and estimate the exceedance probability of each damage state within the considered analysis conditions, thereby providing fundamental reference data for the seismic performance evaluation and seismic design of railway embankment structures. In addition, the PVD at the embankment crest was adopted as the damage index, which is consistent with previous fragility studies on embankment structures [17,19]. For the concrete slab track system considered in this study, the rigid track structure transmits PVD at the embankment crest directly to the track level, supporting the use of PVD as a reasonable proxy for vertical track irregularity. The use of absolute PVD rather than relative differential displacement also provides a conservative basis for damage assessment.

Meanwhile, this study was conducted based on two-dimensional plane-strain conditions with a uniform sandy soil assumption.

Although two-dimensional plane-strain analysis can be considered an appropriate approach for developing seismic fragility functions of embankment structures, it has limitations in fully capturing the three-dimensional characteristics of seismic loading and the behavior of sections where ground conditions or structural geometry vary along the longitudinal direction. In practice, natural ground profiles often show heterogeneity such as layered deposits with varying stiffness, which may affect the dynamic response characteristics of the ground and the permanent vertical displacement at the embankment crest. However, the primary objective of this study is to independently evaluate the effects of bedrock depth, shear wave velocity of the underlying soil layer, and embankment height on seismic fragility, and the uniform ground condition can be considered as a condition for systematically isolating the effect of each variable. In future studies, more comprehensive evaluations will be conducted by incorporating three-dimensional analysis, multilayered or heterogeneous ground conditions, and actual track structures. In addition, track irregularity will be more explicitly evaluated through three-dimensional embankment–track system modeling. The effectiveness of various seismic intensity measures, including PGV and Arias intensity, will also be evaluated to identify an optimal IM for the seismic fragility assessment of embankment structures. Furthermore, based on additional analysis results, a fragility curve equation applicable to practical seismic design will be developed.

CRediT authorship contribution statement

Hyunmin Song: Investigation, Methodology, Writing – original draft. **Ji Hyeon Kim:** Methodology, Project administration, Visualization. **Mintaek Yoo:** Conceptualization, Supervision, Writing – review & editing.

Declaration of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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Data availability

Data will be made available on request.

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