Assessment of the Design Spectrum with Aggravation Factors by 2D Nonlinear Numerical Analyses: A Case Study in Gemlik Basin, Turkey

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Abstract

The response spectra of multidimensional analyses are compared with the one-dimensional (1D) local models to couple the irregular soil stratification effect in a site. In recent studies, the surface motion spectra ratios of 2D/1D or 3D/1D are defined as spectral aggravation factors for each region in a site. Particularly in alluvial basins, where the soil media is typically formed by fault ruptures or topographic depressions filled with sediments, the inclination of the rock outcrop in the edge of the basin has a considerable effect on the site response, and such effect has not yet been taken into consideration of recent seismic building codes and general engineering applications. In this study, the natural alluvial basin near the North Anatolian Fault in Gemlik, Maramara Region, Turkey, was investigated by 40 seismic site tests and 4 validation borings. The 2D and 1D nonlinear response history analyses in north-south and east-west directions of the Gemlik basin were performed by numerical model on finite difference scheme considering nonlinear elasto-plastic material behaviors and geometric discontinuities. 22 strong ground motions recorded on rock site are excited vertically as SH waves. The numerical results exhibited the narrow basin effects are derived not only by reflection, refraction, and shifting behavior but also by focusing and superposition of the seismic waves propagating from both opposite basin edges. As a result, the site-specific spectral aggravation factors, SAF 2D/1D defined by the ratio between the 2D and 1D acceleration response spectra for each period and any location on the site, were proposed for the Gemlik basin. The variations of the aggravation factors were observed as increasing values to 1.2-2.2 on the near edge and basin center.

Keywords: Aggravation factor; Nonlinear analysis; Basin effect; 2D Site response; Gemlik basin; Finite Difference Method

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1. Introduction

The design of earthquake-resistant structures in living quarters is one of the significant objectives of civil engineering. The prediction of the hazards on all living facilities to protect lives by reducing the destructive effect of earthquakes has always been crucial in terms of engineering. To estimate a design earthquake is the main phenomenon in the design of a building and has a crucial role in examining the existing building performance to minimize the loss of life and property. In the site response investigations performed last decades, the acquirements have pointed to four main aspects that shape the strong ground motions. The first is the amplification of displacement which exists when a seismic wave progresses through an interface to lower rigidity layers from the higher rigidity ones. The second is the resonance of the flat layers developed mechanically at specific frequencies. The third originates from the nonlinearity of soil stress-strain behavior and the nature of inhomogeneity and anisotropy in the material. The last effect derives from the wave propagation variation in the soil half-space, which has multilayered site conditions with stratigraphic heterogeneities.

In early studies, to reveal the effect of plane-incident wave in soft two-dimensional basin Aki and Larner (1970), Wong and Trifunac (1974) estimated surface motion of valleys with perfectly elastic material for incident plane SH rays by semi-analytical method. Smith (1975) performed analyses using finite difference and finite element techniques to study the effects of irregular layer interface systematically. Bard and Bouchon (1980a) examined the formation of surface waves and edge effects. King and Tucker (1984) noted significant differences in surface peak horizontal accelerations at the center and near the edges of the valleys. Yamanaka (1989) performed in situ investigation and numerical analysis to study the propagation of the seismic waves within the deep sedimentary layers of southwestern Kanto district, Japan. Papageorgiou and Kim (1991) investigated the effect of bedrock slope on amplifications by establishing a 2D model in the Caracas basin in Venezuela during the Caracas earthquake on 29 July 1967. Zhang and Papageorgiou (1996) worked at the Marina basin in San Francisco, California, to predict the impact of the Loma Prieta earthquake on 18 October 1989. Pei and Papageorgiou (1996) studied the surface waves created by motions traveling up from the basin bottom on the records from Gilroy seismograph arrays placed on the surface along the Santa Clara basin in California. Kawase (1996) used the 2D Finite Element model to analyze the basin edge effect along the damaged zone in the Kobe basin observed in the Hyogoken Nanbu earthquake on 17 January 1995. Graves et al. (1998) suggested that amplification increase was directly caused by surface waves generated from the basin edge. Bielak et al. (1999) evaluated soil amplification and structural damage together in a small valley in Kirovakan depending on basin conditions. The performed 1D wave propagation analyses could not provide sufficient results for the existence and spatial distribution of the 1988 Armenia (Spitak) earthquake damage over a wide area in Kirovakan.

In the progressive stages of the multidimensional response studies, the concept of aggravation factor was proposed by Chávez-Garcia and Faccioli (2000) Bakir et al. (2002) analyzed the strong ground motion in the Dinar district, where the 2D model was established on the edge of an alluvial basin in Southeastern Anatolia during the Dinar earthquake. Somerville et al. (2003) asserted that commonly used empirical approaches do not reflect the additional effects of sedimentary basins. In relevant studies, Semblat et al. (2005), Bard et al. (2010), Iyisan and Hasal (2011), Iyisan and Khanbabazadeh (2013), Abraham et al. (2016), Khanbabazadeh et al. (2016), Riga et al. (2016), Makra and Chávez-Garcia (2016), Chávez-Garcia FJ et al. (2018), Cipta et al. (2018), Moczo et al. (2018), Zhu et al. (2018), the results
found in the two-dimensional numerical analyses support this situation. In the current researches, Hasal et al. (2018), Khanbabazadeh et al. (2018; 2019), Ozaslan et al. (2020), it has been studied by time-domain dynamic analyses in idealized 2D models of Dinar and Düzce basins.

This study presents the effects of the heterogeneities in both vertical and lateral directions to the local seismic response by concentrating on the earthquake response analysis of the basin which is laterally confined and in the form of filled-sediment. In this aspect, a nonlinear soil behavior was examined under the risk-targeted levels consisted of design and maximum considered earthquakes. For this purpose, the proposed procedure is based on the correlation between detailed numerical modeling solutions to extract the contributions due to 2D effects additional to 1D soil behavior. The impacts of a multiaxial stress state in the soil, formed by a 2D-3D decomposed shallow stratigraphy, need to be investigated since they can play a remarkable role in the resulting nonlinear strains. In this study, comprehensive in-situ investigation and numerical analyses were performed to study the site response and propagation of the seismic waves within the shallow sedimentary basin in the Gemlik district of western Marmara Sea, Turkey. In the scope of the study, the effects of lateral discontinuities defined as basin geometry and slope of the bedrock on accelerations of design earthquake were investigated by nonlinear numerical analyses on natural basin conditions.

2. Site Investigations

2.1. Seismic array tests and borings

Gemlik basin in Bursa, located between latitudes of 40°26’24N-40°25’16N and the longitudes of 29°9’0E-29°11’20E, has been determined as the research area. The basin, which is around 1.7 km width in the north-south direction and 2.5 km long in the east-west direction, is enclosed by two fault segments on the southwestern branch of the North Anatolian Fault. The alluvium deposit and bedrock form in the basin have been assessed by a large number of microtremor array measurements and validation borings. Microtremor array measurements have been made at 40 different research points in total with two instrument sets. The recorded microtremors have been analyzed by performing the SPAC method using Fortran codes to create a detailed underground model of the basin (Okada and Suto 2003; Yamanaka 2005). Besides, confirmatory drillings and Standard Penetration Tests have been performed in 4 sites. The coordinates of the all site investigations are presented with emphasis on the methodologies and tools employed in Figure 1 and Figure 2, respectively. In the seismic experiments, a circular array consisting of seven equally spaced stations around the central station has been preferred because of its feasibility in the highly intensive residential areas. In this method, the phase velocities of Rayleigh waves are computed on cross-correlations between each station pairs on the circular array by analyzing vertical components of microtremors. The array consists of three stations located on the corners of two equilateral triangles and a three-components station at the center of the circle. The observed phase velocities are employed to the estimation of the shear wave velocities and layer thicknesses by performing inversion with Genetic Algorithm and Simulated Annealing method, which generates optimization in mutations, crossover, and selection of individuals in a population (Yamanaka and Ishida 1996)
The applied sizes of the array circle have ranged from 20m to 50m in radius. The phase velocities of Rayleigh waves have been estimated by 20-30 minutes long microtremor data, and the shear wave velocities of the soil layers have been determined by the inversion method. It has been revealed that the soil structure consists of two layers lied on the bedrock whose mean S-wave velocities are around 180 m/s and 500 m/s in Figure 3. Microtremor measurements have been completed by using the test set of Tokyo Institute of Technology with the frequency range from 3 to 100 Hz and
the test set Bursa Metropolitan Municipality with Guralp CMG-DM24S24 Laptop integrated 24 channel portable seismic recorder system with 1-500 Hz vertical feedback geophones.

![Microtremor array method](image)

**Figure 3.** Array measurement plan, data set, phase velocity and SPAC coefficients and Vs profile

The accuracy of the soil section models created in the study was controlled by the consistency of the data collected in both seismic and penetration tests. For this purpose, BH1, BH2, BH3, and BH4 investigation borings and penetration tests were performed at the same points with the microtremor measurements, MA BBB-5, MA BBB-6, MA BBB-7, and MA BBB-10, which were made in the basin center and near the edges.

In the borings, BH1 and BH2 medium to high plasticity clay and silty clay unit are observed to the depth of 8m-12m. The Triassic Abadiye formation belongs to the Karakaya group, which consists of meta claystone, metasiltstone, metasandstone, and metalimestone. The unit in question is generally very weathered, very weak, and fragmented because it is in the fault zone. It is fully weathered residual soils (sandy, silty clay), and the weathering degree decreases towards the end of the boring. In the third and fourth boreholes, a silty clay unit with a fine sand band, grayish color, and medium to high plasticity have been passed between 2m-30m. These units are Quaternary-aged alluvium. Under the alluvial layers, there is a residual soil (sandy, gravelly clay) deeper than 35m until the end of the borings, and it is predicted to have formed after the complete weathering of the Triassic Abadiye formation belonging to the Karakaya group. In the 4th borehole, shells belonging to sea creatures are observed in the boring sample after 20m in the formation (Okay and Güncüoğlu 2004). The alluvial deposits in the main lithological units detected in boring logs BH1-BH4 in the Gemlik basin are given in Figure 4.
The alluvial deposits in the main lithology discovered by borings BH1-BH4 in Gemlik basin

The correlations suggested by Imai (1977), JRA (1980), Iyisan (1996) were used to calculate the variation of shear wave velocity of the Standard Penetration Test data. The mean of the three different equations $V_s = 102N^{0.292}$, $V_s = 100N^{0.33}$, $V_s = 51.5N^{0.516}$ (respectively, where $N$ is the number of blows) was compared with the results of the seismic tests in Figure 5. In the borings, BH3 and BH4, which are closer to the basin center, the Standard Penetration Test blow counts (SPT-N) take lower values ranging from 5 to 15 in the upper layer with 10-40 m depth while these values increase to 20 to 40 after 35 m depth. In the borings located on the near edges of the basin, the mentioned increase starts from 10-20 m depth.

Figure 5. Comparison of the shear wave profiles of seismic test and SPT
2.2. The soil condition and basin geometry

Geographical Information System (GIS) was used to integrate all collected data on storing, evaluating, and presenting the field investigations. It provided the visualization of the layers created with the interpolation analysis of the spatial locations of the whole field study in 2D and 3D maps and helped the preparation of sections by establishing relations between data. The variation of bedrock depth along the basin base detected in simultaneous microtremor array measurements is presented in Figure 6. In addition, the resultant 2D layered basin models in the north-south direction (section A-A) and east-west direction (section B-B) are given by shear wave velocity intervals in Figure 7.

Figure 6. Bedrock depth by shear wave velocities in Gemlik basin

The Gemlik basin is a relatively shallow sedimentary structure with gentle slopes in the East-West direction, while the North-South direction has high inclined rock outcrops on the edges. The basin has a complicated geometry, where the depth of the soil deposit is estimated as 140m-180m at the center, overlie a rigid bedrock, and surrounded by an inclined rock outcrop. The 2D models of shear wave velocity exhibited the bedrock inclinations of the basin edges which extend with α₁=18°, α₂=10° in North direction and α₃=16°, α₄=32° in South direction and the 400m length flat interface between the bedrock and the soil deposit. On the East-West direction, there is the broader flat base with 1400m and the edge slopes with α₅=9°, α₆=10° in Figure 7.
Figure 7. The variation of the bedrock and layers inclinations in A-A and B-B sections and the positions of the site investigations
3. Site Response Analyses by Numerical Models

3.1. Description of Finite Difference-based nonlinear method

In the nonlinear 1D and 2D response history analyses of the basin, the explicit finite difference method has been performed by FLAC3D code (Fast Lagrangian Analysis of Continua 3D). Contrary to previous studies, this method provides elastoplastic soil nonlinearity under shear and compressional wave propagation by considering strain-dependent nonlinear constitutive rule and yielding criteria at any time during dynamic excitation in the same simulation. In this way, the averaged strain rate calculations over all subzones on the soil space mesh are performed before any calls of computation step by regarding constitutive model functions of the soil materials without any other damping requirement.

In the numerical analysis, the soil constitutive model needs to reflect nonlinear elastoplastic material properties under cyclic load in the time domain. In the applied constitutive model, energy dissipation is provided by the hysteretic model with the degradation of shear modulus (G/G\text{max}) and cyclic damping (D) at small strain levels. Furthermore, the plastic deformations of soil materials at high strain levels are by the Mohr-Coulomb model. The combination of strain-dependent damping ratio and secant modulus functions are derived by given equations and illustrated by Figure 8 (a). A loop tracked on an initial cycle of unloading/reloading is illustrated in Figure 8 (b).

![Figure 8](image)

**Figure 8.** (a) Shear modulus and damping with shear strain of three different cases, (b) Shear stress-strain loop, Mohr-Coulomb model with hysteretic damping (Itasca, 2017)

Subsequently, the yielding level of the hyperbolic rule must be higher than the Mohr-Coulomb yield stress. The state is provided with the condition given in the equation below (Cundall 2001).

\[ \gamma_{\text{ref}} > \gamma_m \]  \hspace{1cm} (1)

Where \( \gamma_{\text{ref}} \) is the ultimate value of \( \tau_m/G_{\text{max}} \) in hysteretic function, \( G_{\text{max}} \) is maximum shear modulus, \( \tau_m, \gamma_m \) are constant yield stress and shear strain, respectively.

\[ \gamma_m = \frac{\tau_m}{G_{\text{max}}} \]  \hspace{1cm} (2)
In the elastic range, $\gamma_c < \gamma_m$, the modulus reduction factor is defined by Equation 3,

$$ G / G_{\text{max}} = \frac{1}{1 + \frac{\gamma_c}{\gamma_m}} $$

In the plastic range, $\gamma_c \geq \gamma_m$.

$$ G / G_{\text{max}} = \frac{1}{1 + \frac{\gamma_m}{\gamma_{\text{ref}}} - \frac{\gamma_m}{\gamma_{\text{ref}}}} $$

The energy dissipation in a cycle, $\Delta W$, is expressed as the total of contributions from elastic $\Delta W_H$, and plastic $\Delta W_{MC}$ ranges.

$$ \Delta W = \Delta W_H + \Delta W_{MC} $$

$$ \Delta W_H = 4G_{\text{maks}} Y_{\text{ref}}^2 \left( 2 \left[ \frac{\gamma_m}{\gamma_{\text{ref}}} - \ln \left( 1 + \frac{\gamma_m}{\gamma_{\text{ref}}} \right) \right] \left( 1 + \frac{\gamma_m}{\gamma_{\text{ref}}} \right) / \left( 1 + \frac{\gamma_m}{\gamma_{\text{ref}}} \right)^2 \right) $$

$$ \Delta W_{MC} = \left( \frac{G_{\text{maks}}}{1 + \frac{\gamma_m}{\gamma_{\text{ref}}} \gamma_m} \right) \frac{\gamma_c}{\gamma_m} \left( \frac{\gamma_c}{\gamma_m} - 1 \right) $$

The maximum stored energy, $W$ and the damping ratio, $D$ in a cycle are given by equations below.

$$ W = \frac{1}{2} \tau_m y_c $$

$$ D = \frac{1}{4\pi} \frac{\Delta W_H + \Delta W_{MC}}{W} $$

$$ D = \frac{2}{\pi} \left( 2 \left[ \frac{\gamma_{\text{ref}}}{\gamma_{\text{ref}}} + \ln \left( 1 + \frac{\gamma_{\text{ref}}}{\gamma_{\text{ref}}} \right) \right] \frac{1}{\gamma_{\text{ref}}} + \frac{\gamma_c - \gamma_m}{\gamma_c} \right) $$

### 3.2. Boundary conditions and damping

The entire range of problems potentially encountered in geotechnical engineering exist in semi-infinite soil space, and the solutions are performed on the simulated finite media by discretizing with numerical methods. Thus the boundary conditions on the solution of the problem are crucial and need to be provided suitable with the surveyed wave propagation on infinite site conditions. In the dynamic analysis, advanced boundary conditions, improved to prevent the waves from being trapped inside the model without reflections that exist at the finite model borders, have been executed and given in Figure 9. The effectiveness of this type of energy-absorbing boundary has been demonstrated in both finite-difference and finite-element models (Cundall 2001).
At the bottom of the models, the quiet boundaries which prevent the reflection of outward propagating waves back into the model were assigned. On the lateral boundaries of the models, the nonreflecting free field boundaries were settled in a continuum finite difference scheme by coupling the main grid to the free-field grid by viscous dashpots. The assigned dashpots produce viscous normal and shear stress tractions along the model boundaries by given Equation 11-12.

\[ t_n = -\rho C_p v_n \]  
(11)

\[ t_s = -\rho C_s v_s \]  
(12)

Where, \( t_n, t_s \) normal and shear stresses traction, \( \rho \) is the mass density, \( C_p \) and \( C_s \) are the pressure (\( P \)) and shear (\( S \)) wave velocities, \( v_n \) and \( v_s \) are the normal and shear components of velocity at the quiet boundary.

\[ F_x = -\rho C_p (v_x^{m} - v_{x,ff})A + F_{x,ff} \]  
(13)

\[ F_y = -\rho C_s (v_y^{m} - v_{y,ff})A + F_{y,ff} \]  
(14)

\[ F_z = -\rho C_s (v_z^{m} - v_{z,ff})A + F_{z,ff} \]  
(15)

Where, \( F_x, F_y, F_z \) are gridpoint tractions of the free-field boundary, \( v_x^{m}, v_y^{m}, v_z^{m} \) are \( x, y, z \) velocity of gridpoint in the main grid at side boundary, \( v_x^{ff}, v_y^{ff}, v_z^{ff} \) are \( x, y \) velocity of gridpoint inside free field, \( A \) is the area of influence of free-field gridpoint, \( F_{x,ff}, F_{y,ff}, F_{z,ff} \) are free-field gridpoint force with contributions stresses of the free-field zones around the gridpoint (Itasca 2017).

### 3.3. 2D and 1D models of the Gemlik basin

The soil material in the basin has low shear wave velocity (\( V_s \)) ranging from 160 m/s to 220 in the near-surface, and the underlying layers get more rigid with increasing \( V_s \) from 300 m/s to 550 m/s by the influence of the effective stress until the bedrock which has \( V_s \) more than 750 m/s at the base. The basin consists of alluvium deposits, old river and sea sediments composed of sandy-silty clay layers with medium to high plasticity index (PI) value change between 15–25%. The significant \( V_s \) contrast between deposits and bedrock is not very common in real basin conditions, even if the sediments are poorly consolidated soils overlying on bedrock, because of the increasing overburden pressure on the deeper layers (Bard and Bouchon 1980a; Zhu and Thambiratnam 2016). The layer descriptions, soil formations, shear strength parameters, and maximum modulus considered in sublayers are given in Table 1.
Table 1. Specifications of the sub-layers and bedrock in the considered model.

| Layers description                  | Formation                  | c (kPa) | ϕ (°)  | Vₜ (m/s) | G (MPa) | K (MPa) | γ (kN/m³) |
|-------------------------------------|----------------------------|---------|--------|----------|---------|---------|-----------|
| Silt clay (medium plasticity)       | Quaternary Alluvial        | 50-80   | 5      | 160-220  | 40-90   | 165-280 | 17.0-18.0 |
| Sandy silty clay (medium to high plasticity) | Triassic Abadiye Formation | 100-150 | 10     | 300-550  | 170-600 | 475-1300| 19.0-20.0 |
| Bedrock                             | -                          | -       | -      | 750-1300 | 1050-3650| 1700-6100| 21.0-22.0 |

c: cohesion, ϕ: shear strength angle, Vₜ: shear wave velocity, G: shear modulus, K: bulk modulus, γ: unit weight

Following the research topic, the preferred software allows investigating the wave propagation produced by such multiple physical phenomena as refraction, reflection, and resonance in infinite soil media and nonlinear soil behavior. In 2D and 1D analysis, when performing this type of analysis, the sizes of the discriminated element in the soil media needs to permit the transition of the applied seismic waves. Similarly, the smallest time step required for each calculation needs to ensure the highest frequency input motion propagation in the model. Only in these conditions, the motion can be transferred accurately to the surface throughout the defined finite media. Figure 10 presents free field boundary zones and the Finite Difference scheme of the Gemlik basins in 2D plane strain models, which are built and investigated by numerical analyses. In this way, plane waves propagate upward and sustain no distortion at the boundary because the free-field zones supply identical to those in an infinite model.

Figure 10. Finite Difference scheme of the numerical model of Gemlik basin in two directions

3.4. Verification of the wave propagation

In the study, a trapezoidal model proved by Kawase and Aki (1989) has been produced identically to validate the wave propagation. The same acceleration pattern has been captured across the model surface and given in Figure 11. The result of the Kawase and Aki (1989) analysis also has been tested by Iyisan and Khanbabazadeh (2014), Gil-Zepeda et al. (2003). The properties of the materials in the model have been defined by 1000-2500 m/s shear wave velocities (Vₜ), and the unit weight of materials have been taken as the same to provide a constant impedance value concerning the compared model. The shear and bulk modulus have been considered by taking the Poisson ratio as 1/3. The propagation and distortion of the wave, the change of the wave characteristics in the material environment, and the
spectrum distribution have been investigated using the wavelet. As the input motion, the Ricker pulse has provided the opportunity to definitely examine wave propagation for different frequency ($f_c$) ranges and defined amplitudes $u(t)$. Consequently, it was settled that the numerical method and boundary conditions provide wave propagation, distraction, and reflection properly the same as the verification model.

![Wavelet diagram](image)

**Figure 11.** The specifications of the verification model and input motion (a) surface motion on the validation model in the study and (b) Kawase and Aki (1989)

In 2D plane strain and 1D soil column models of the Gemlik basin have been built and investigated by the same numerical method. When generating the finite difference scheme, the maximum defined zone size ($l_{max}$) has been restricted to be equal to or less than $1/10$ or $1/8$ of the lowest wavelength ($\lambda_{min}$) defined in the model. In other words, it depends on the wavelength of the wave with maximum frequency ($f_{max}$) transmitted in the softest layer of the media. The zone dimensions are determined by the equations given below (Cundall 2001; Itasca 2017)

$$\lambda_{min} \leq \frac{V_{min}}{f_{max}} \quad (16)$$

$$\Delta l_{max} \leq \frac{\lambda_{min}}{10} \quad (17)$$

In the models, considering the assumed relations, the maximum zone sizes ($\Delta l_{max}$) have been regulated as $1m$ to $5m$ by fitting to the shear wave velocity of the soil layers by seeing the earthquake records have the highest frequency components up to $15$ Hz.

### 3.5. Input motions

Computing response spectra for several varying strong ground motions and averaging them will lead to a smoother target spectrum. Producing such smoothed spectra is a significant step in improving a design spectrum. The ground motions are defined probabilistically as risk-targeted spectra by current seismic code provisions through the seismic hazard analyses. The results of hazard analyses are used to determine the site-specific Maximum Considered
Earthquake (MCEₚ) and the Design Earthquake (DEₚ) spectrum, and also site-specific design acceleration parameters of short-period, SDS and 1-second period, SD1. The underlying methods of a site-specific ground motion analysis are necessarily highly technical and require a unique combination of geotechnical, earth science, and probabilistic expertise (FEMA 2020).

Across the basin models, both 1D and 2D site response analyses were performed under the sets of ground motion data which were selected by matching to the level of MCEₚ spectrum and the level of DEₚ spectrum defined by exceedance probability of 2% in 50 years and 10% in 50 years, respectively in Figure 12. The input motion selection also considers near-fault and transition regions with given distances in the specification of the earthquakes in Table 2. In total, 22 earthquakes have been selected by matching to two levels of target spectra. Each strong ground motion set contains 11 earthquakes filtered by a 25 Hz low-pass filter and baseline corrected (Bommer and Acevedo 2004; Katsanos et al. 2010).

Table 2. The specifications of the selected earthquakes for DEₚ and MCEₚ levels

| Event  | Station, Year | Record                | Component | Magnitude Mₚ | Epicentral Distance (km) | Hypocentral Distance (km) | PGA (g) |
|--------|---------------|-----------------------|-----------|--------------|--------------------------|---------------------------|---------|
| E1     | San Fernando, 1971 | Pacoima Dam           | 164       | 6.6 (Mₚ)    | 11.9                     | 17.6                      | 1.22    |
| E2     | San Fernando, 1971 | Pacoima Dam           | 254       | 6.6 (Mₚ)    | 11.9                     | 17.6                      | 1.24    |
| E3     | Tabas, Iran, 1978 | Tabas                 | T         | 7.4 (Mₚ)    | 55.2                     | 55.5                      | 0.86    |
| E4     | Loma Prieta, 1989 | Gilroy Array #1       | 0         | 6.9 (Mₚ)    | 28.6                     | 33.6                      | 0.42    |
| E5     | Northridge, 1994  | Pacoima Dam (Upper left) | 194       | 6.7 (Mₚ)    | 20.4                     | 26.9                      | 1.29    |
| E6     | Duzce, Turkey, 1999 | IRIGM 496            | N-S       | 7.1 (Mₚ)    | 24.3                     | 28.1                      | 1.03    |
| E7     | 1999 Kocaeli Earthquake | Kocaeli M. Station | E-W       | 7.4 (Mₚ)    | 7.4                      | 17.5                      | 0.23    |
| E8     | Parkfield, 2004  | Parkfield-Fault Zone 11 | 90       | 6.0 (Mₚ)    | 9.3                      | 12.3                      | 0.60    |
| E9     | Parkfield, 2004  | Parkfield-Gold Hill 3W | 90       | 6.0 (Mₚ)    | 4.8                      | 9.4                       | 0.79    |
| E10    | Parkfield, 2004  | Parkfield-Fault Zone 11 | 360       | 6.0 (Mₚ)    | 9.3                      | 12.3                      | 1.13    |
| E11    | L'Aquila, Italy, 2009 | Aterno -Colle Grilli | 30       | 6.3 (Mₚ)    | 4.5                      | 10.3                      | 0.48    |
| E12    | Nahanni, Canada, 1985 | Site 1              | 280       | 6.8 (Mₚ)    | 6.5                      | 10.3                      | 1.2     |
| E13    | Nahanni, Canada, 1985 | Site 1              | 10        | 6.8 (Mₚ)    | 6.8                      | 10.5                      | 1.11    |
| E14    | San Fernando, 1971 | Pacoima Dam           | 164       | 6.6 (Mₚ)    | 11.9                     | 17.6                      | 1.22    |
| E15    | Palm Springs, 1986 | Anza-Red Mountain     | 360       | 6.1 (Mₚ)    | 46.2                     | 47.5                      | 0.12    |
| E16    | Loma Prieta, 1989 | Gilroy Array #1       | 0         | 6.9 (Mₚ)    | 28.6                     | 33.6                      | 0.42    |
| E17    | Northridge, 1994  | Pacoima Dam (Upper left) | 194       | 6.7 (Mₚ)    | 20.4                     | 26.9                      | 1.29    |
| E18    | Landers, 1992    | Lucerne              | 260       | 7.3 (Mₚ)    | 44.0                     | 44.6                      | 0.73    |
| E19    | Landers, 1992    | Lucerne              | 345       | 7.3 (Mₚ)    | 44.0                     | 44.6                      | 0.79    |
| E20    | Parkfield, 2004  | Parkfield-Fault Zone 11 | 360       | 6.0 (Mₚ)    | 9.3                      | 12.3                      | 1.13    |
| E21    | Parkfield, 2004  | Parkfield-Fault Zone 11 | 90       | 6.0 (Mₚ)    | 9.3                      | 12.3                      | 0.60    |
| E22    | Parkfield, 2004  | Parkfield-Gold Hill 3W | 90       | 6.0 (Mₚ)    | 4.8                      | 9.4                       | 0.79    |
4. Results and Discussion

4.1. Comparison of 2D and 1D basin response

Two seismic code-based levels of excitations have been performed into models to obtain the seismic response of the Gemlik basin in two directions. Thus, not only 2D and 1D models have been compared but also the maximum accelerations in input motions. In the stage of the assessment of the processed data, SH wave propagation in one-dimensional soil columns created for each point that was selected with 50m intervals throughout the basin surface was investigated by 1D analysis. In established 2D basin models, the effects of the stratigraphic two-dimensional discontinuity produced by refraction and reflection of SV waves were analyzed. Furthermore, the surface motions on 2D models have been recorded by synthetic seismographs located with equal intervals on the surface, similar to 1D analyses as 50m intervals. Consequently, the spectral aggravation factors, SAF2D/1D=Sae(T)2D/Sae(T)1D, have been defined as the ratio between the response spectrums of 2D and 1D models by considering locations and periods. A total of almost 1000 dynamic time history analyses has been performed and all results presented in detail.

On the other hand, the effect of rock outcrop distance to each other in the narrow section has also been studied by the earthquake waves produced on the surface. Accordingly, the ratio of the depth (H) of the basin to its width (L) is smaller than H/L=1/10 in contrary to the prior recent studies carried by Riga et al. (2016), Khanbabazadeh et al. (2018), and Zhu et al. (2018), in which the edge effect was examined.

The response spectra ratios calculated on the surface for different periods across the basin in 2D and 1D models are given in Figures 13-14. The surface acceleration spectra obtained in the artificial recorders s10, s16, s20, s24, s30, placed on the surface in the projections of the points where the changes of the edge slope at the bottom and the points of basin center, have been compared. In the A-A direction where the opposite sides of the basin are closer to each other in the E7 earthquake at the DE level, the aggravation factor increases to 2 in the basin center, especially at T=1s period, while at the larger period of T=1.2s reaches 1.5 values above the edge regions. In the earthquake E20 at MCE level, the aggravation factor reaches the maximum value at the center in the T = 1s period, and similarly to DE level
shifts to the edge region in a larger period. In lower periods, in $T=0.2s$ and $T=0.4s$, it is observed with multiple peak values of the highest aggravation factors occurring close to the edges of the basin in Figure 13.

Figure 13. The variation of the response spectra and aggravation factors in 1D and 2D analyzes across the A-A section for levels of $\text{DE}_R$ and $\text{MCE}_R$.

On the other hand, in the site response analysis conducted in the East-West direction, where the basin is wider than the other direction, and the edge slopes are approximately 10 degrees, smaller aggravation factors were obtained than the narrow section's values.

In Figure 14, the artificial recorders $s_{11}$, $s_{21}$, $s_{31}$, $s_{38}$, $s_{45}$ have been used to compare the results of earthquakes E7 and E20. In the B-B section, where the inclination of the outcrop is relatively lower, the aggravation at high frequencies remains under 1.2, and the 1D and 2D response history analysis results in the center of the basin are almost the same, contrary to the A-A section. The highest aggravation factor values were obtained between 1.25-1.5 in the region close
to the eastern edge. This situation confirms that in accordance with wave propagation phenomena previously described in the Semblat et al. (2005), Riga et al. (2016), and Zhu et al. (2016) studies, the increase in the width of the basin reduces the interference of the waves dispersed into the basin independently of the earthquake levels. In addition, the decreasing edge slope reduces the effects of the refraction, reflection, and shifting produced at high frequencies in the regions close to the edge.

**Figure 14.** The variation of the response spectra and aggravation factors in 1D and 2D analyzes across the B-B section for levels of DE<sub>R</sub> and MCE<sub>R</sub>

### 4.2. Maximum spectral aggravation factors for DE<sub>R</sub> and MCE<sub>R</sub>

The parameters need to satisfy the minimum requirements of the current seismic codes and must be those at the design earthquake or maximum considered earthquake shaking intensity. Site response in a basin can be significantly different
from a result of the 1D analyses. Because the reflection, refraction, and resonance of the seismic waves in the basin soil media alter the surface motion or seismic demands on a structure, it is essential to represent the free field motion by further investigations accurately. The primary step of improving a design spectrum is developing a response spectrum with the average of the spectra obtained by taking into account sufficient diversities of earthquakes in site-specific dynamic analyzes will lead to a smoother set of spectra. Comprehensive field experiments need to be done to investigate the basin structure to carry out such advanced analysis in 2D models. To select the suitable parameters of the soil constitutive models and examine wave propagation requires expertise in geotechnical and earthquake engineering, and evaluation of all analyzes results together requires numerical analysis and coding skills.

For this reason, in addition to the seismic code provisions, it is assumed that aggravation factor charts grouped according to the primary variables defining the soil classes and the geometric structure of a basin by making some assumptions with the procedure followed in several engineering calculations will help the application. Thus, it is considered that 1D analyzes that are currently used can be included in the basin response analyses by developing them with additional aggravation factors. In Figure 15-18, it is aimed to present the highest values of spectral aggravation by grouping to the strong ground motion levels consisting of 11 real earthquakes. Thus, both the effects of the differences in earthquake levels and lateral discontinuities that will cause to change in the aggravation factor depending on the location in the model are tried to be explained. It is illustrated in figures 15-16 that the narrow section of the Gemlik basin in the north-south direction increases the amplifications under the input motions at both levels that cannot be neglected. Considering the results given in Figures 17-18 in the east-west direction, this difference is interpreted as the superposition of the refracted and reflected seismic waves due to the interface which has a higher slope depending on location and frequency.

![Figure 15. The maximum aggravation factors for MCE_r level in section A-A](image)

Figure 15. The maximum aggravation factors for MCE_r level in section A-A
Figure 16. The maximum aggravation factors for $DE_R$ level in section A-A

Figure 17. The maximum aggravation factors for $MCE_R$ level in section B-B
On the other hand, it is seen that the maximum aggravation factors are between 1.25-1.5 values on the eastern edge of the B-B section. It is clear that the higher slope reflects the edge effects dominantly to the surface motions, and the behavior is getting similar to the 1D analysis in case the bedrock outcrop does not reach the surface regardless of earthquake level.

As the spectral aggravation factor charts unique for the Gemlik basin, it is intended the maximum values of the response spectra ratios are calculated by 1D and 2D time history analyses for 22 earthquakes in both directions at all points and each period. This spatial distribution of the maximum aggravation across the narrow direction can be seen in Figure 19.

At 1s period, a restricted region in the center has aggravation factors greater than 2-2.25, and the second peak is observed as 1.50-1.75 at 1.2s period while on the basin edges, the aggravations are noticed as 1.75-2.00 between 1s and 1.2s with narrowing width of the basins. In Figure 20, totally different from the A-A section, it can be noticed that in section B-B, the maximum aggravations increase to 1.25-1.5 in periods between 1s-1.5s only in the region close to the east edge.
Figure 19. Maximum spectral aggravation factors for north-south direction in Gemlik basin

Figure 20. Maximum spectral aggravation factors for east-west direction in Gemlik basin
5. Conclusion

In the Gemlik basin models created in the north-south and east-west directions by seismic tests and boring investigations, the aggravation factors have been defined by site-specific response analyzes performed considering the DE_e and MCE_e earthquake levels. The progression of the surface waves which are derived from both edges in the narrow direction of the Gemlik basin into the center of the basin, create higher amplification particularly at lower frequencies in 2D plane strain analyses with respect to results of the 1D soil column method. On the contrary, in the wider direction where the bedrock slope is lower, it is recognized that the aggravations take lower values only near the edge region, and the 2D and 1D analysis results get similar as they move away from the edge.

Consequently, the dynamic property of the soil deposit and earthquake characteristics have a remarkable effect on the strong ground motion. In contrast, in basins, the discontinuities of soil layers and distracted-interacted propagation of seismic waves have a more significant role in distributing the peak ground accelerations.

Particularly in alluvial sites, different regions along the basin surface are affected to various degrees. Therefore, the aggravation coefficients proposed for each specified region in the basin could be used more feasibly by calibrating to the 1D design spectrum. The multidimensional response analysis methods are necessitating a technical combination of geotechnical, earth science, and time-consuming process. Hence, the study method can develop basin-specific aggravating factors, and the suggested charts can be used with seismic code provisions. Finally, it is asserted that further studies that were numerically defining the regions where the basin affects mainly emerge and which periods are critical will significantly contribute to revealing the uncertainties about the subject.

Declaration of Conflicting Interests and Data Availability Statement

On behalf of all authors, the corresponding author states that there is no potential conflict of interest with respect to the research, authorship, and/or publication of this article. All data of the analyses and models or codes that clarify this investigation's findings are available from the corresponding author upon reasonable request.

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