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Insights into Liquefaction-Triggered Lateral Spreading in Brest Pokupski During the 2020 Petrinja Earthquake

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Abstract

This paper presents a numerical analysis of liquefaction-induced lateral spreading at Brest Pokupski in Croatia, observed during the 2020 M_w 6.4 Petrinja earthquake. The site is composed of Holocene alluvial deposits consisting of loose to medium-dense sands, silty sands, and sandy silts with high groundwater levels, locally confined by low-permeability cohesive layers, which together created favourable conditions for excess pore pressure generation, liquefaction and lateral spreading. During the 2020 Petrinja earthquake, the area experienced large-scale ground failures, including widespread soil deformations, sand ejecta, ground cracking and settlement, making it one of the most prominent manifestations of earthquake-induced liquefaction in the affected region. The study applies dynamic analyses using the advanced constitutive models and soil parameters derived from laboratory tests and CPT investigations. The model simulates excess pore water pressure development and resulting ground deformations, with displacements evaluated at advanced stages of dissipation. During the dissipation phases, the model exhibits a high sensitivity to hydraulic conductivity, which may be attributed to slight residual numerical imbalances at the end of shaking. Consequently, scaling factors of 100 and 1000 result in horizontal displacements of approximately 20–30 cm and 60–70 cm, respectively, within the liquefiable layer. Trends of ground displacement with distance along the profile, however, remain consistent regardless of the applied scaling factors, and can therefore be used in conjunction with displacement measurements to assess the response. Numerical results are used in conjunction with InSAR observations from the European Ground Motion Service (EGMS) and geodetic benchmark measurements to assess the displacement field changes along the profile. The comparison from two points showed a shift in the displacement direction towards the river by about 14°.

Keywords: lateral spreading; liquefaction; dynamic numerical analysis; remote sensing



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1. Introduction

Among the various consequences of soil liquefaction during earthquakes, lateral spreading is recognized as one of the most damaging and hazardous phenomena. It is generally defined as the predominantly horizontal ground displacement toward a free face, such as a riverbank or excavation, as a result of strength loss in an underlying liquefied layer. Lateral spreading occurs when a relatively intact upper soil crust is transported over a liquefied substratum until a new equilibrium configuration is reached. This phenomenon typically develops in soils with mild ground slopes ranging approximately from 0.3%

to 5% [1], while steeper slopes tend to experience classical slope failure mechanisms. During earthquake-induced liquefaction, cyclic shearing causes rearrangement and collapse of the soil particle structure, particularly in contractive soils. This rearrangement reduces interparticle contact stresses and transfers load to the pore water, resulting in the generation of excess pore water pressures and a reduction in effective stress. Subsequent dissipation of excess pore pressures may lead to soil densification and settlement. Although the overall volumetric change within the layer may be limited, substantial variations in void ratio and pore water distribution can develop with depth. These processes result in localized zones where the soil behaves in a fluid-like manner, drastically reducing shear strength and enabling lateral displacement of the overlying non-liquefied crust. The movement of the upper crust during lateral spreading often occurs in the form of discrete blocks of varying sizes.

Research on liquefaction-induced lateral spreading has traditionally relied on case history analyses from past earthquakes, which have provided the empirical basis for widely used predictive models. Numerous well-documented seismic events have reported significant lateral spreading, including the 1999 Kocaeli earthquake [2], the 1999 Chi-Chi earthquake [3], the 2010 Maule earthquake [4], the 2011 Tohoku earthquake [5], the 2011 Christchurch earthquake [6], the 2014 Cephalonia earthquake [7], and more recently the 2020 Petrinja earthquake [8] and 2021 Maduo earthquake [9]. These case-specific investigations have contributed significantly to the understanding of the mechanisms and controlling factors governing lateral spreading. However, empirical approaches remain dependent on available case-history databases and are associated with uncertainties related to the definition of key parameters and limited representation of extreme seismic events. In particular, previous studies have highlighted the scarcity of observations from very large magnitude earthquakes ($M_w > 8$) and uncertainties related to source-to-site distance parameters [10,11].

To overcome some of these limitations, numerical modelling has become an increasingly important tool for investigating the complex mechanisms associated with liquefaction-induced lateral spreading. Advanced finite element and finite difference approaches enable detailed simulation of soil response under seismic loading through the incorporation of sophisticated constitutive models and site-specific conditions. For example, de la Maza et al. [12] investigated lateral spreading at Caleta Lo Rojas during the Maule earthquake using numerical analyses and compared the results with predictions obtained from the Youd et al. [13] methodology and post-earthquake field measurements. Similarly, extensive field observations following the Canterbury earthquake sequence revealed lateral displacements of up to 2–3 m along riverbanks, extending 200–250 m inland [14]. More recent numerical studies by Millen and Cubrinovski [15] successfully reproduced these observations, emphasizing the importance of geometric conditions, soil stratigraphy, and seismic intensity in controlling lateral spreading behaviour. The reliability of numerical approaches has been further demonstrated through comparisons with experimental and field observations. Valsamis et al. [16] employed a validated numerical methodology calibrated against centrifuge experiments to develop simplified relationships for predicting maximum ground displacement. Further, Araujo and Ledezma [17] investigated lateral spreading case histories from the 2010 Maule and 1964 Alaska earthquakes using advanced numerical tools, with additional validation performed against centrifuge experiments from the VELACS [18] and LEAP [19] projects. Their results demonstrated that numerical models can adequately reproduce pore water pressure generation and shear modulus degradation during strong seismic loading.

Beyond liquefaction-induced lateral spreading, the integration of numerical modelling with remote sensing techniques has become an increasingly valuable framework for evaluating permanent ground deformation caused by natural and anthropogenic hazards. Satellite- and UAV-based observations provide spatially distributed deformation measurements that

complement numerical predictions and improve understanding of complex ground failure mechanisms [20,21]. Additionally, data-driven approaches based on machine learning techniques, including artificial neural networks and conditional generative adversarial networks, have recently shown potential for predicting lateral spreading displacements using extensive case-history databases [22–25]. However, these methods primarily focus on prediction accuracy and do not explicitly represent the physical mechanisms governing deformation.

Despite these advances, relatively few studies have combined validated dynamic numerical modelling with detailed field observations and remote sensing data to investigate lateral spreading triggered by moderate-magnitude earthquakes. In particular, the lateral spreading induced by the 2020 M_w 6.4 Petrinja earthquake has not yet been examined using a numerical framework capable of reproducing observed deformation patterns. This study addresses this gap by integrating site-specific numerical modelling, field investigations, and observations based on InSAR and geodetic benchmarks to evaluate the mechanisms governing lateral spreading at Brest Pokupski and assess the capability of the adopted modelling approach to reproduce and predict ground displacements.

2. Case Study Description

One of the most significant manifestations of earthquake-induced ground failure during the 2020 Petrinja earthquake sequence was recorded in the area of Brest Pokupski, located along the Kupa River floodplain, approximately 3 km from the epicentre. The M_w 6.4 earthquake triggered widespread liquefaction phenomena across the central part of Croatia, particularly within young alluvial environments composed of saturated sandy and silty deposits (Figure 1) [8,26].

The geological setting of Brest Pokupski strongly influenced the observed ground response. The area is predominantly underlain by Holocene alluvial and floodplain deposits consisting of poorly graded sands, silty sands, and sandy silts with variable fine content. These deposits were formed in a dynamic fluvial environment and are typically characterized by loose to medium-dense packing and high groundwater levels, conditions highly susceptible to seismic liquefaction. The presence of laterally continuous saturated sandy strata beneath thin cohesive surface layers further contributed to the development of extensive lateral spreading during the earthquake shaking. Together with the local geomorphological conditions, these factors created highly favourable conditions for liquefaction occurrence at the study site. Similar geological and hydrogeological conditions were also present in other parts of the affected region, contributing to widespread liquefaction manifestations over an estimated area of approximately 600 km² following the 2020 Petrinja earthquake [27].

Post-earthquake field reconnaissance documented numerous manifestations of liquefaction and lateral spreading, including ground fissures, longitudinal and transverse cracking, differential settlements and sand ejecta. Among the observed geotechnical effects, lateral spreading represented one of the most damaging processes affecting both natural terrain and built infrastructure at the Brest Pokupski site. Significant damage was recorded on transportation infrastructure, including local roads and the main bridge crossing (Figure 2a), where permanent ground deformations caused cracking, settlements, and distortion of the pavement structure and bridge approaches. Severe damage was also observed along the flood protection levee in Brest Pokupski (Figure 2b), where liquefaction-induced deformations and longitudinal cracking developed in sections exposed to strong lateral ground movements. At Brest Pokupski, lateral spreading developed as a consequence of cyclic pore pressure build-up within loose, saturated sandy layers during strong ground shaking. The phenomenon was especially pronounced in low-lying floodplain areas characterized by

shallow groundwater conditions and recent fluvial sediments deposited by the Kupa River. The terrain gently slopes toward the Kupa River, with an average inclination of approximately 3%, creating favourable preconditions for downslope lateral ground displacement toward the river channel during liquefaction. The settlement of Brest Pokupski is located approximately 400 m from the Kupa River, where intense liquefaction manifestations with extensive sand ejecta were documented in the field (Figure 2c).

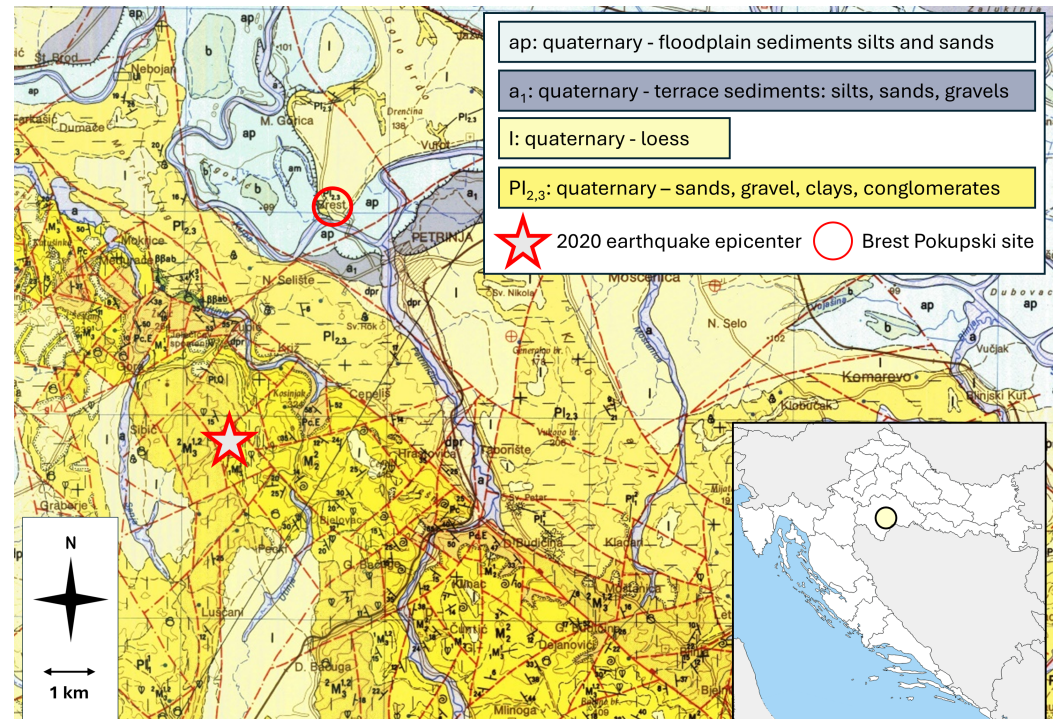


Figure 1. Location of the Brest Pokupski case—study site where liquefaction and lateral spreading occurred during the Petrinja 2020 earthquake, modified from Pikija [28].



Figure 2. Manifestations and consequences of liquefaction-induced lateral spreading at Brest Pokupski following the 2020 Petrinja earthquake: (a) damage to the bridge crossing and adjacent roadway infrastructure caused by permanent ground deformation (photograph: index.hr portal), (b) longitudinal cracking and deformation of the flood protection levee, and (c) sand ejecta and surface evidence of liquefaction documented in the Brest Pokupski site.

The Brest Pokupski case represents a well-documented example of liquefaction-induced lateral spreading associated with the 2020 Petrinja earthquake and provides valuable insight into the relationship between local geological conditions, groundwater levels, topography and seismic loading. The documented ground deformations and infrastructure damage clearly demonstrate the susceptibility of young alluvial deposits to liquefaction and lateral spreading processes, particularly in lowland river floodplain environments characterized by loose saturated sandy sediments.

3. Integrated Methodology for Lateral Spreading Analysis

The adopted methodology consists of several sequential steps aimed at evaluating liquefaction-induced lateral spreading at the Brest Pokupski site. The first stage involved the collection and interpretation of available geological, geotechnical, and topographical data required for the definition of the numerical model geometry and subsurface conditions. Based on the identified stratigraphy and results of CPT investigations, material parameters for the constitutive model were determined. Subsequently, dynamic numerical analyses were performed to simulate the liquefaction response and associated permanent ground deformations during the 2020 Petrinja earthquake. Finally, the obtained numerical results were compared with satellite-based ground displacement observations derived from InSAR data in order to validate the adopted modelling approach.

3.1. Terrain Geometry and Topographic Data Acquisition

The terrain geometry used in the numerical analyses was derived from high-resolution LiDAR data combined with bathymetric survey data of the Kupa River channel. The LiDAR dataset was acquired within the national project “Multisensor Aerial Survey of the Republic of Croatia for Disaster Risk Reduction Purposes”, coordinated by the State Geodetic Administration of Croatia. The digital terrain model has a spatial resolution of 1 m, with a minimum point density of 8 points/m² in urban areas and 4 points/m² in non-urban areas. The vertical and horizontal accuracies are ± 0.10 m and ± 0.20 m, respectively, which are sufficient for the geometric representation required in the present analyses [29]. Since the study site is located immediately adjacent to the Kupa River, the riverbed geometry was complemented using bathymetric survey data acquired within the VEPAR project [30] coordinated by Croatian Waters. The VEPAR programme included systematic bathymetric surveys of major Croatian river systems to support flood risk management and hydraulic analyses, providing high-resolution riverbed geometry that was integrated with the LiDAR-derived terrain model. The combined datasets were subsequently used to define the geometry of the numerical model.

3.2. Geotechnical Investigations

The soil stratigraphy, material properties, and field conditions at the Brest Pokupski site were assessed through an extensive post-earthquake geotechnical investigation campaign consisting of boreholes with accompanying laboratory testing, CPTu soundings, and geophysical field investigations, all conducted one to two months after the earthquake. The investigations were carried out following the 2020 Petrinja earthquake in order to evaluate subsurface conditions within the damaged settlement area and to support the definition of remediation priorities related to liquefaction-affected ground.

The investigated zone was located approximately 400 m from the free face of the Kupa River, which represents the primary direction of observed lateral spreading. Consequently, certain assumptions regarding the continuity of soil layering within the intermediate area included in the numerical model had to be adopted. Nevertheless, the available investigation data indicate a relatively uniform subsurface profile dominated by loose to medium-dense sandy deposits confined by low-permeability silty and clayey layers, creat-

ing favourable conditions for cyclic pore pressure build-up and liquefaction development during seismic loading.

As shown in Figure 3, the field investigation program included nine CPTu soundings distributed across the study area. In order to verify and calibrate the CPT-based soil interpretation, three boreholes with laboratory testing were additionally performed. Furthermore, MASW investigations were conducted along five profiles to determine the shear wave velocity distribution, while seismic refraction testing was performed along one selected profile. Figure 3 also presents the location of the representative cross-section adopted for the subsequent numerical analyses.

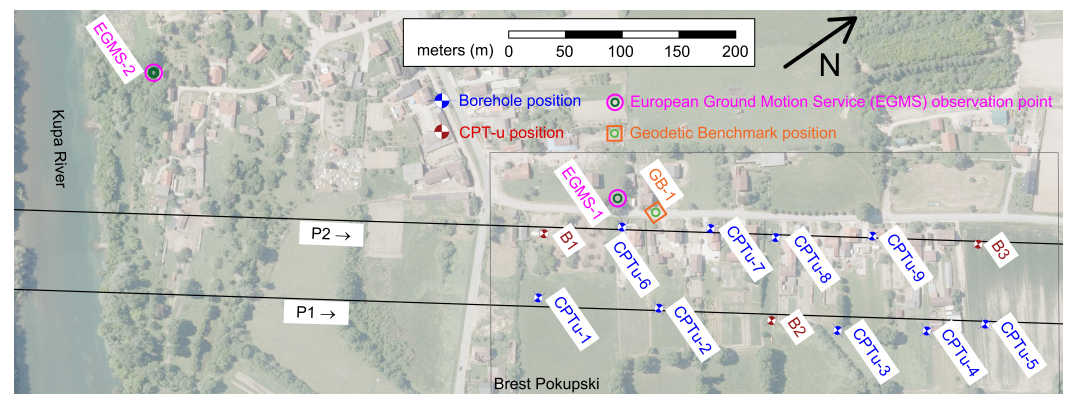


Figure 3. Layout of the post-earthquake geotechnical investigation campaign at the Brest Pokupski site, including CPTu soundings and boreholes together with the location of the representative cross-section adopted for numerical analyses.

3.3. Numerical Modelling and Non-Linear Dynamic Analysis

Fully non-linear dynamic-with-consolidation numerical analyses were performed using the finite element software PLAXIS 2D version 2024.3.0.95 developed by Bentley Systems (Bentley, 2024, Exton, PA, USA). The analyses were conducted within an effective stress framework in order to simulate the coupled seismic response of the soil, excess pore water pressure generation, and the development of permanent ground deformations associated with liquefaction-induced lateral spreading.

Since the investigated subsurface profile consists predominantly of liquefiable sandy deposits confined between low-permeability cohesive layers, different constitutive models were adopted to adequately represent the mechanical behaviour of individual soil units. The cohesive layers were simulated using the Hardening Soil model with small-strain stiffness (HSsmall), which enables improved representation of stress-dependent stiffness, small-strain soil behaviour, and cyclic degradation effects. The liquefiable sandy layer was modelled using the PM4Sand constitutive model, specifically developed for the simulation of cyclic mobility, pore pressure accumulation, and liquefaction response of saturated sands under earthquake loading.

The dynamic analyses incorporated earthquake ground motion time histories applied at the model boundaries. To capture the progressive development of lateral spreading and large permanent deformations, a large-deformation formulation was adopted, allowing continuous updating of the finite element mesh geometry throughout the calculation process. Coupled consolidation calculations were additionally performed following the peak seismic loading phase to simulate the dissipation of excess pore water pressures and the post-shaking stabilization of the soil mass. The final permanent displacements were evaluated at a stage corresponding to approximately 90% dissipation of excess pore water pressures, representing the stabilized post-earthquake ground condition.

3.4. Comparison with Remote Sensing Observations and Terrestrial Measurements

The numerical results were compared to ground displacement measurements obtained through the European Ground Motion Service (EGMS), as well as geodetic benchmark measurements presented and made available by Henriquet et al. [31]. The EGMS platform represents a major advancement in the operational application of Interferometric Synthetic Aperture Radar (InSAR) technology for large-scale ground deformation monitoring across Europe [32]. Developed within the Copernicus Earth observation programme, EGMS provides harmonized, standardized, and regularly updated ground motion products derived from Sentinel-1 SAR imagery.

Satellite-based remote sensing techniques have become increasingly important in modern geoscience due to their capability to provide continuous spatial coverage of ground deformation processes. Among these methods, InSAR has proven particularly valuable because it enables the detection of surface displacements with millimetre-scale precision by analysing phase differences between repeated radar acquisitions [33]. Developments in multi-temporal InSAR approaches, such as Persistent Scatterer Interferometry (PSI) and Small Baseline Subset (SBAS), have significantly improved the reliability and spatial consistency of deformation measurements [34,35]. These methods enable the detection of both slow, continuous deformation processes and rapid displacement events, providing a more comprehensive understanding of ground motion dynamics [36].

In this study, EGMS Level 3 orthorectified products were used, providing decomposed ground displacement time series in the east–west directions. The satellite-derived deformation data were compared with the numerically predicted permanent ground displacements in order to evaluate the capability of the adopted numerical model to reproduce the observed liquefaction-induced lateral spreading behaviour at the Brest Pokupski site. Although the interpretation of InSAR measurements may be affected by vegetation cover, atmospheric disturbances, and the line-of-sight nature of SAR observations, the EGMS dataset provides a valuable and independent basis for validating large-scale spatial patterns of earthquake-induced ground deformation.

On the other hand, geodetic benchmark measurements are independent of weather and vegetation conditions, and measure the true displacements in the direction of movement, but data is much more spatially and temporally sparse. One benchmark point placed for cadastral surveying is found close to the location where field investigations are performed (Figure 3), with measurements available prior to and after the earthquake. In combination the east–west InSAR measurements, a more complete and reliable assessment can be performed with regards to lateral spreading related ground movements.

4. Geotechnical Characterization and Numerical Model Development

4.1. Subsurface Conditions and Constitutive Model Calibration

The layout of the performed field investigations is presented in Figure 3. CPTu soundings 1–5 are located along profile P1, oriented approximately perpendicular to the Kupa River, while CPTu soundings 6–9 are positioned along profile P2, parallel to P1 and shifted approximately 80 m toward the northwest, where the terrain elevation is approximately 2–3 m higher.

The interpreted stratigraphy from the CPTu investigations, confirmed by borehole data, indicates relatively uniform alluvial depositional conditions across the site. Along profile P1, the subsurface generally consists of a near-surface cohesive clay layer overlying a sandy layer, beneath which alternating cohesive and mixed fine-grained deposits with increased coarse-grained content are present. The sandy layer identified below the upper cohesive unit is interpreted as the primary liquefiable stratum. Along profile P2, a similar stratigraphic sequence is observed; however, the uppermost part of the profile contains a

shallow sandy surface layer, likely representing anthropogenic fill associated with nearby roads and residential infrastructure. In addition, the identified stratigraphic boundaries generally follow the natural terrain morphology, exhibiting a gentle inclination from the elevated P2 profile toward the lower P1 profile closer to the Kupa River floodplain. This interpreted geometry is schematically shown in Figure 4.

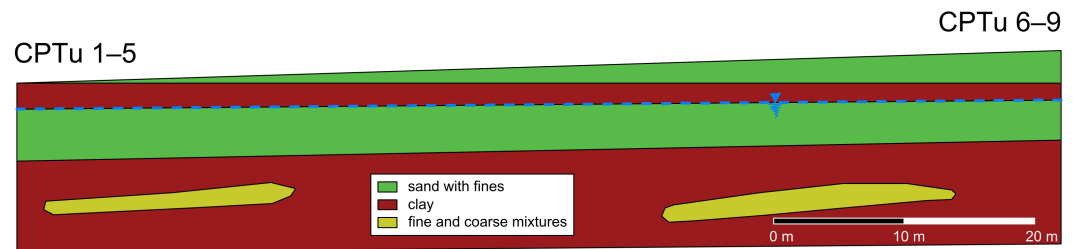


Figure 4. Generalised terrain morphology and subsurface stratigraphy in the direction perpendicular to the analysed profile.

One of the key factors controlling the occurrence of liquefaction in susceptible soil layers is the groundwater regime and degree of saturation. Based on borehole records and CPTu investigations, the groundwater table was identified at depths closely corresponding to the top of the liquefiable sandy layer. The presence of saturated sandy deposits confined beneath a low-permeability cohesive surface layer created favourable conditions for excess pore pressure generation during seismic loading. Considering the detected groundwater conditions and stratigraphy, only the deeper saturated sandy layer is considered susceptible to liquefaction. Consequently, the numerical and geotechnical analyses presented in this study primarily focus on the stratigraphic conditions identified along profile P1, where the most representative liquefaction-prone conditions were observed.

For the estimation of model parameters, laboratory test results obtained from boreholes B1, B2, and B3 were combined with data from the five CPTu soundings located along profile P1. The interpreted CPTu results, presented in Figure 5, consistently identify a distinct sandy layer at depths between approximately 2 and 6 m. The thickness of this layer varies between roughly 3 and 5 m across the investigated profile and is primarily identified based on the normalized friction ratio (F_r) response. Considering its grain-size characteristics, relative density, and saturated conditions, this layer was interpreted as the primary liquefiable stratum and was therefore adopted as the critical layer in the numerical analyses. It must be noted that considerable variability and locally high values are observed for the u_2 parameter, particularly within the fine-grained layers with low hydraulic conductivity. It is likely that cone penetration through these layers generates excess pore pressures that do not have sufficient time to dissipate during penetration. However, the locally high u_2 values may also be related to the measurement conditions and cannot be unequivocally attributed to this mechanism. In contrast, the pore pressures within the liquefiable sandy layer display an approximately hydrostatic trend, consistent with the water level detected in the boreholes. Despite the local and considerable variability in u_2 , the corrected tip resistance, $q_{t,c}$, is only slightly affected and remains very similar to the measured q_c . Consequently, the observed variability in u_2 does not have a significant effect on subsequent processing.

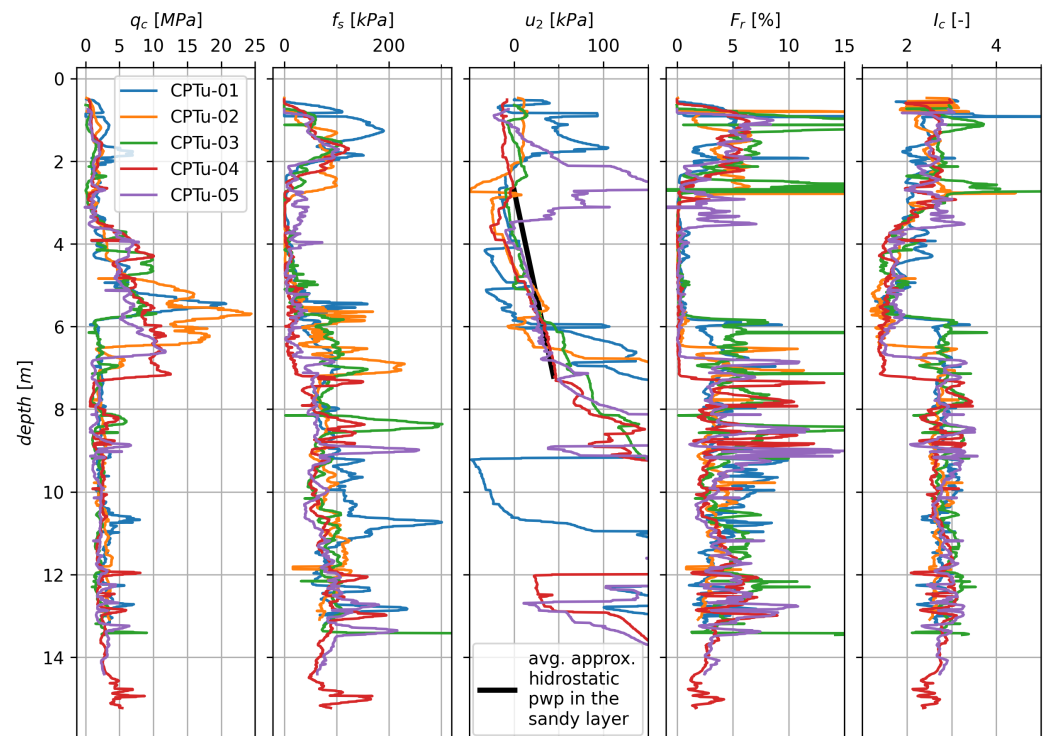


Figure 5. CPTu profiles showing the identified liquefiable layer at 2 to 6 m depth.

To further present the in-situ data, a combined interpretation of all five CPTu soundings is provided on the Robertson soil behaviour type chart [37], Figure 6. Each CPTu sounding is represented by a distinct symbol, while depth within each sounding is encoded using a continuous colour scale. This representation highlights the consistent occurrence of a sandy soil layer at depths between approximately 2 and 6 m, which aligns well with the interpreted stratigraphic model and confirms the presence of the identified liquefiable layer. The PM4Sand constitutive model parameters were selected within recommended ranges for soils with comparable density and grading characteristics, while particular attention was given to the calibration of the parameter h_{p0} based on site-specific cyclic resistance information. The cyclic resistance ratio (CRR) was evaluated from five CPTu profiles following the procedure of Boulanger and Idriss [38], resulting in an average CRR value of 0.192 for the identified liquefiable layer corresponding to a magnitude $M_w 7.5$ event. Although the calculated CRR values exhibit spatial variability, ranging from 0.147 to 0.345, representative average properties were adopted throughout the numerical model to investigate the global mechanism of liquefaction-induced lateral spreading rather than local effects associated with small-scale soil heterogeneity.

The parameter h_{p0} was then calibrated through a series of stress-controlled cyclic direct simple shear (CDSS) simulations, with the aim of reproducing the observed cyclic response inferred from CPTu-based resistance. The calibration was guided by matching the onset of liquefaction after approximately 15 uniform loading cycles under a shear stress amplitude (τ_{cyc}) consistent with $CSR \times \sigma'_{v0}$ (19.2 kPa for the initial vertical effective stress of 100 kPa), with CSR set equal to CRR. This number of cycles reflects the equivalent cyclic demand associated with a magnitude $M_w 7.5$ earthquake, as suggested by Idriss and Boulanger [39].

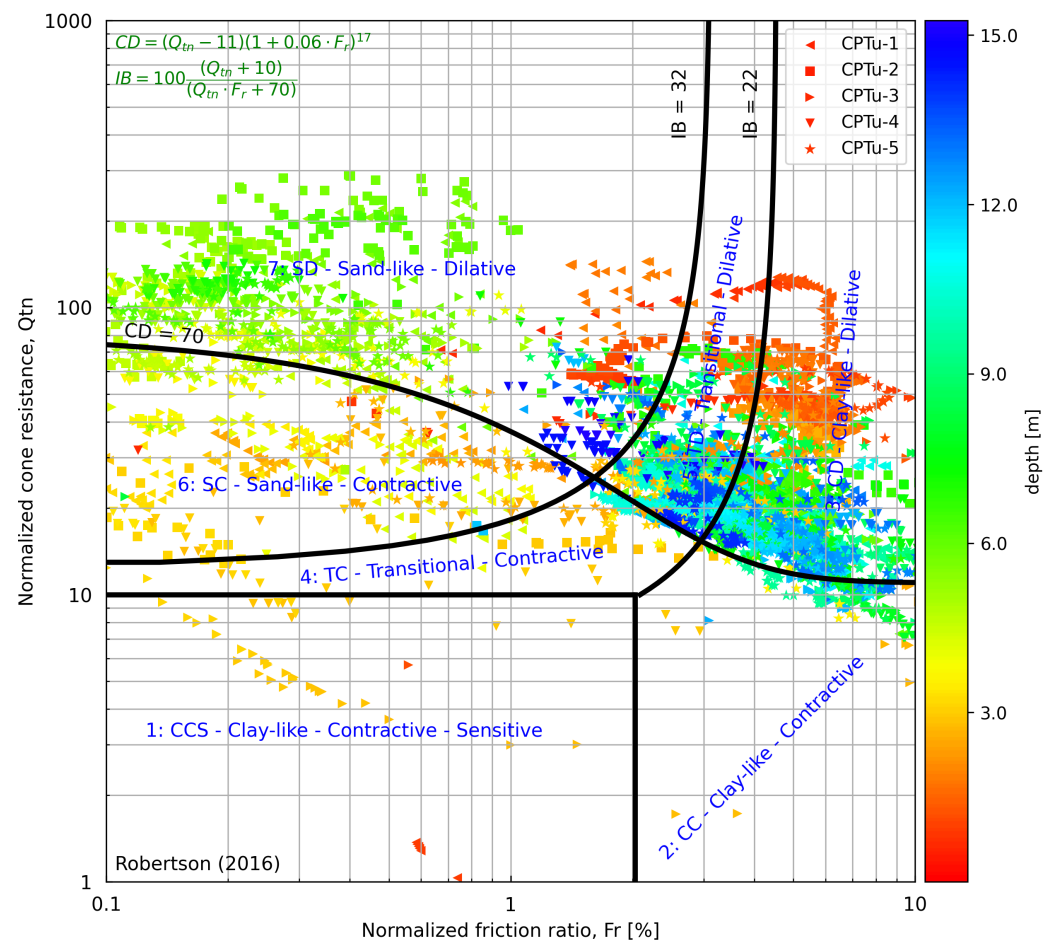


Figure 6. Robertson chart representation of CPTu soundings with depth variation, based on Robertson [37].

Within this framework, liquefaction was interpreted as the point at which shear strains reached approximately 3%, in line with the criterion proposed by Boulanger and Ziotopoulou [40]. The resulting calibrated response, shown in Figure 7, is obtained with $h_{p0} = 0.38$, providing a reasonable match between the model response and the CPTu-inferred cyclic resistance of the soil. All fine-grained materials identified throughout the soil profile are represented using a single material definition, for which a common set of parameters is adopted. The Hardening Soil with Small Strain Stiffness model is used for this material. The same constitutive model is also applied to the sandy units in the initial (non-dynamic) calculation phases, as well as in limited zones adjacent to the model boundaries during the dynamic analysis, ensuring numerical consistency across the model domain.

Accordingly, a unified parameter set is calibrated to represent the response of all relevant soil units. The calibration primarily focuses on the reference stiffness parameters (E_{ref} and $G_{0,ref}$) and the stress dependency exponent m . The parameter m is obtained by fitting a linear trend in a log–log space, based on the relationship between E_{oed} and the normalized stress level, expressed as $\ln(E_{oed})$ versus $\ln\left(\frac{\sigma'_1 + c \cdot \cot \varphi}{p_a + c \cdot \cot \varphi}\right)$. In this formulation, the major principal effective stress is approximated by the vertical effective stress, with each CPTu data point representing a distinct stress state.

The same procedure is applied consistently to both E_{oed} and G_0 across all CPTu profiles, and the resulting values of m are subsequently harmonized to ensure a consistent stress-dependent formulation. An illustrative example of this calibration is provided in Figure 8. Finally, Equation (1) is fitted to the CPTu-derived data using the calibrated m values in order to determine the corresponding reference stiffness parameters.

$$E_{oed} = E_{oed,ref} \left(\frac{\sigma'_1 + c \cdot \cot \varphi}{p_a + c \cdot \cot \varphi} \right)^m \quad (1)$$

The soil parameters adopted for the Hardening Soil with Small Strain Stiffness (HSSmall) model are summarized in Table 1, while the parameters for the PM4Sand model are given in Table 2. Parameters were mostly derived from CPTu-based correlations, complemented where appropriate by correlations with borehole SPT results and available index properties. Where multiple estimation approaches were available, the final parameter set was defined based on a best-estimate interpretation, ensuring consistency between the different data sources and the overall soil behaviour inferred from the in-situ and laboratory investigations. As the primary focus of this study is lateral spreading induced by liquefaction, which does not occur in clay materials, the clay layers were assumed to remain fully drained throughout the entire analysis. Consequently, no excess pore water pressures were generated in the clay, thereby reducing the computational cost.

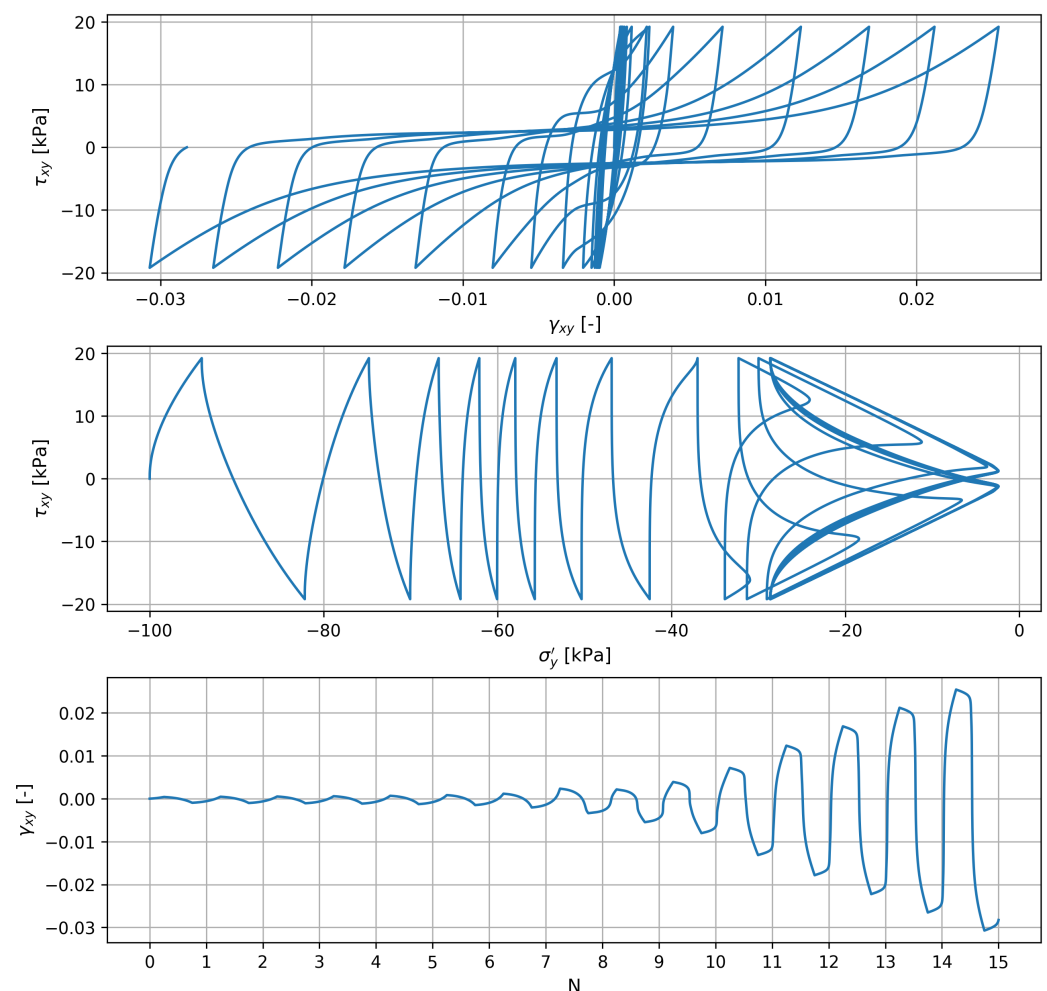


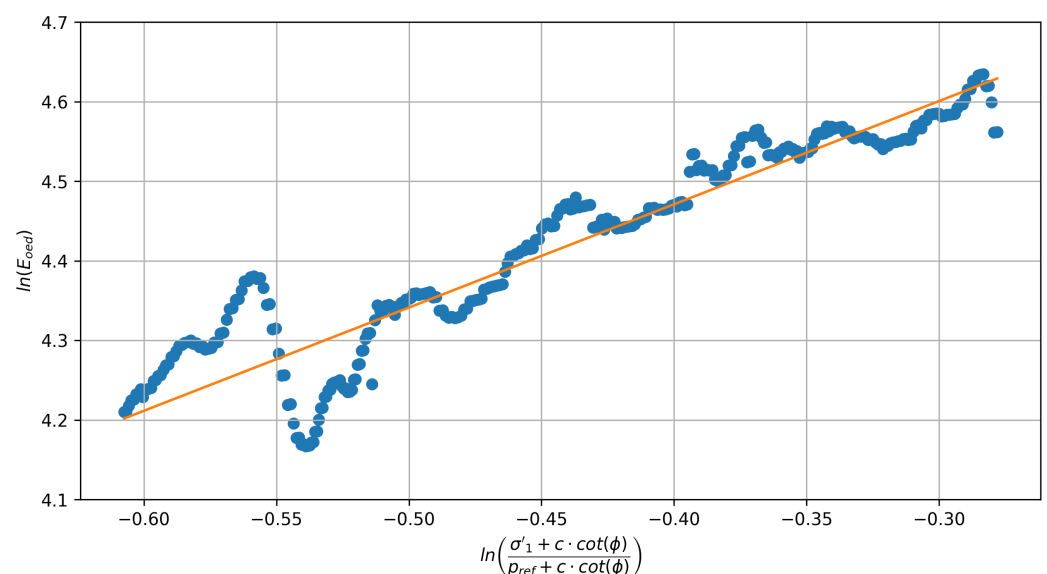
Figure 7. Simulated CDSS test for PM4Sand model calibration: **(top)** shear stress–shear strain (τ_{xy} – γ_{xy}); **(middle)** shear stress–vertical effective stress (τ_{xy} – σ'_y); **(bottom)** shear strain–number of loading cycles (γ_{xy} – N).

Table 1. Parameters of the HSSmall model.

Parameter	Clay	Sand
drainage type	drained	undrained (A)
e_0 [-]	0.67	0.54
$E_{oed,ref}$ [MPa]	6.9	21.4
$E_{50,ref}$ [MPa]	6.9	21.4
$E_{ur,ref}$ [MPa]	34.5	64.2
$G_{0,ref}$ [MPa]	69.8	119.1
m [-]	0.698	0.903
$\gamma_{0.7}$ [-]	2.7×10^{-4}	1.12×10^{-4}
φ [°]	26.9	37.5
c [kPa]	11.2	7.5
k [m/s]	2.4×10^{-8}	2.9×10^{-5}
unsaturated flow model	van Genuchten with USDA dataset	
OCR [-]	5	1

Table 2. Parameters of the PM4Sand model.

D_r [-]	G_0 [-]	n_b [-]	n_d [-]	Q [-]	R [-]	h_{p0} [-]	φ_{cv} [°]	e_{min} [-]	e_{max} [-]
0.636	1191	0.5	0.1	10	1.5	0.38	33°	0.5	0.8

**Figure 8.** Calibration of the stress dependency exponent m .

4.2. Numerical Model Setup

To capture the lateral spreading response, the numerical model was defined to represent the zone closest to the Kupa River, where the phenomenon is expected to be most pronounced due to the gentle ground slopes and proximity to the watercourse. The model domain extends approximately 300 m in the horizontal direction and 30 m in depth, and includes the riverbed area, discretized by 6805 elements (average size 2.054 m; maximum size 5.198 m; minimum size 0.4684 m) with 55678 nodes. Although site investigations are not available within the near-river zone, the stratigraphic profile was extrapolated based on the interpreted site conditions and the relatively uniform geological setting of the area. Considering the depositional environment associated with the Kupa river floodplain, the adopted extrapolation is considered a reasonable assumption. This interpretation is also consistent with geotechnical investigations carried out on both riverbanks for the design and rehabilitation of nearby infrastructure, which generally indicate a stratigraphic

sequence comprising cohesive soils overlying a sand layer underlain by cohesive soils. A 2 m thick surface clay layer is underlain by a 4 m thick saturated sandy layer identified as the liquefiable stratum, followed by a deeper clay deposit extending to the base of the model. Based on LiDAR survey data, the terrain surface and layer boundaries were assigned a uniform slope of 2.77% (1.59°) across the model. The adopted stratigraphy is shown in Figure 9. The groundwater level is positioned at the top of the liquefiable sand layer, consistent with the interpreted field data, and remains at a constant level in the river during the computations, at an elevation of 100.1 m.a.s.l. At the vertical model boundaries, free-field conditions are set, while at the bottom boundary the compliant-base condition is set. To ensure numerical stability, thin boundary zones are introduced at the model edges and at the ground surface above the liquefiable layer, where the Hardening Soil with Small Strain Stiffness model is used instead of PM4Sand.

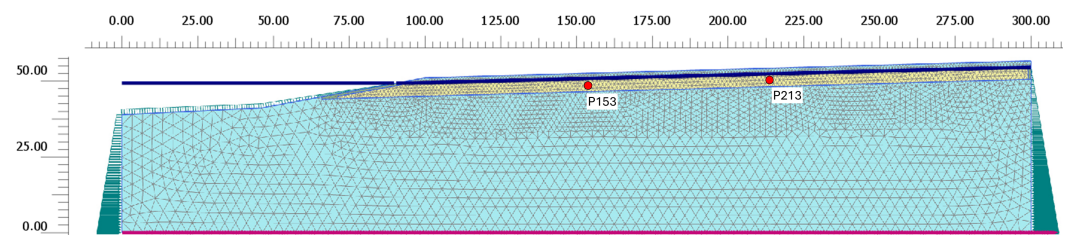


Figure 9. Computational model setup of P1 cross section, used for lateral spreading analysis.

The numerical analyses are performed to evaluate the evolution of excess pore pressures and the resulting ground displacements during seismic loading and subsequent dissipation. The modelling framework consists of five sequential phases, three of which represent dynamic shaking:

1. Initial stress state phase—establishment of in-situ stresses, with all soil units modelled using the HSSmall constitutive model;
2. Undrained adjustment phase (no external loading)—transition of the sandy layer to the PM4Sand model prior to dynamic excitation;
3. Dynamic phase (strong motion duration)—application of the main shaking record, with the PostShake formulation deactivated;
4. Dynamic phase (remaining earthquake duration)—continuation of the input motion with the PostShake formulation activated to capture post-liquefaction reconsolidation effects;
5. Post-shaking dissipation phase—no external excitation, used to simulate pore pressure dissipation and long-term consolidation.

To improve computational efficiency during the post-shaking dissipation phase and test the sensitivity of the model to total displacements and displacement gradients, the hydraulic conductivity of the sandy layer is increased using two scaling factors ($SF = 100$ and 1000), and the phase is analysed accordingly. The strong motion duration is defined up to 95% of the Arias intensity of the input signal.

As no recorded ground motion from the 2020 $M_w 6.4$ Petrinja earthquake was available for the study area, this study utilizes a ground motion recorded during the 2020 Zagreb $M_w 5.3$ earthquake. The record was selected due to its comparable near-source characteristics. It was subsequently scaled to the estimated peak ground acceleration (PGA) of 0.41 g at the study site using a Ground Motion Prediction Equation (GMPE). GMPEs are empirical models used to estimate expected ground shaking as a function of earthquake magnitude, source-to-site distance, local site conditions, and regional attenuation properties. The target PGA values applied in this study were derived from metadata reported by the US Geological Survey for the Petrinja earthquake sequence [41]. While in principle the magnitude of

an earthquake affects its ground motion parameters and frequency content, usage of a real near-source earthquake record is deemed better than utilizing synthetic accelerograms with the presumed characteristics for a given magnitude. The significant duration of the Zagreb earthquake is approximately 8 s, while for a far-source record of the Petrinja earthquake the significant duration is approximately 11.5 s. Given how the predictions made by various GMMs see [42] vary from 3 to 6 s for M_w5 – M_w6 , the Zagreb record is chosen as the most relevant record available. The scaled ground motion record for the Zagreb earthquake is shown in Figure 10.

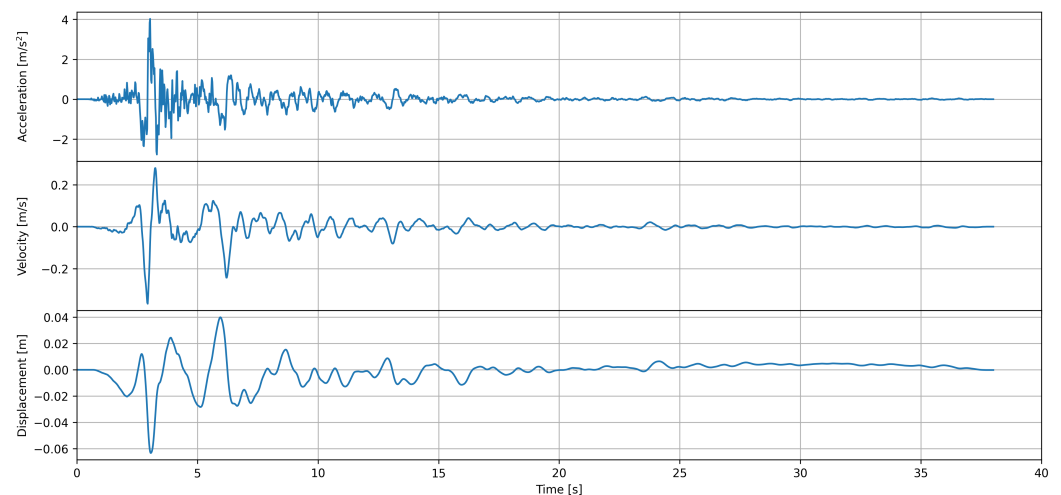


Figure 10. Scaled ground motion record of the Zagreb earthquake.

5. Results and Validation

The final horizontal displacements considered in this study are defined at the stage where 90% of the excess pore water pressure has dissipated. Because the dissipation process is spatially non-uniform, different locations within the model do not reach this criterion simultaneously. Figure 11 illustrates the spatial distribution of excess pore water pressures during the strong motion, weak motion, and dissipation phases of the numerical analysis. To provide a representative assessment of the dissipation behaviour, two observation points, P153 and P213 (Figure 9), were selected for detailed analysis. These locations were chosen because they are sufficiently far from the model boundaries and therefore minimally affected by boundary-induced numerical disturbances where significantly higher excess pore pressures develop toward the right side of the model. During the dissipation phase, the excess pore pressures gradually decrease due to drainage and consolidation processes within the soil mass.

Figure 12 presents the evolution of excess pore water pressure during the strong motion, weak motion, and dissipation phases of the numerical simulations at both observation points. During the shaking phases, excess pore pressures progressively increase due to cyclic loading, reaching peak values near the end of the seismic excitation. Following the termination of shaking, the dissipation phase is characterized by a gradual reduction in pore pressure as drainage occurs within the soil domain. To define the end of the dissipation process, a cut-off criterion corresponding to 10% of the peak excess pore water pressure was adopted. This threshold represents the point at which most of the generated excess pore pressure has dissipated and the soil response approaches a stabilized condition. Based on this criterion, the required dissipation duration was estimated to range between ~80–120 s for the SF1000 cases and ~340–440 s for the SF100 cases.

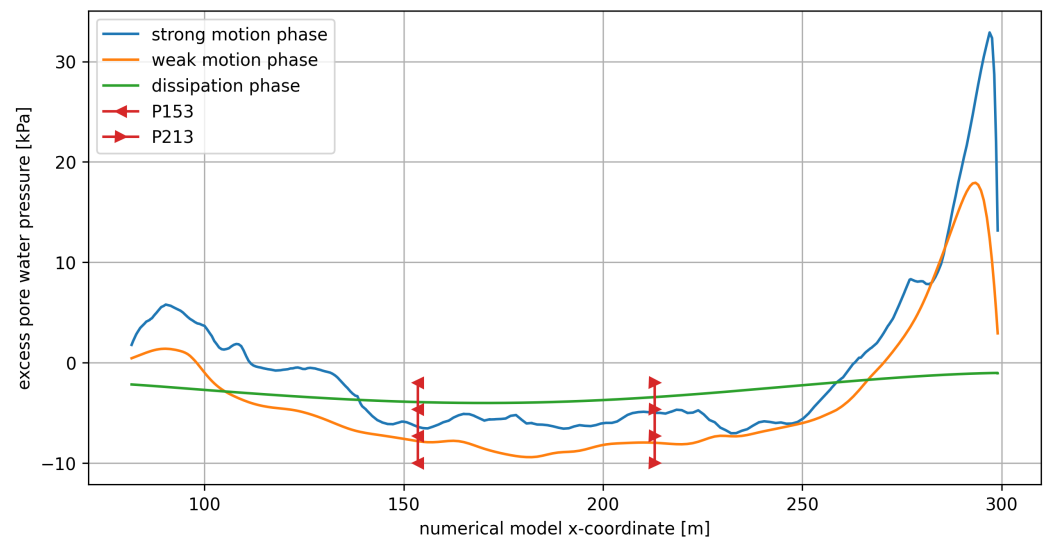


Figure 11. Excess pore water pressure evolution along the analysed cross-section during the strong motion, weak motion and dissipation phases of the numerical analyses.

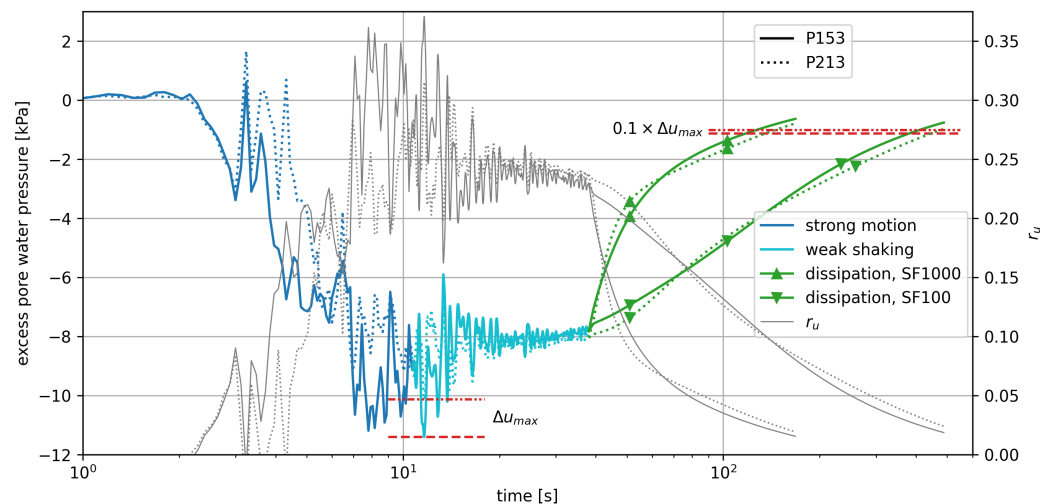


Figure 12. Excess pore water pressure development and dissipation at observation points P153 and P213.

The horizontal displacement contours across the whole model are shown for two points in time in Figure 13, one at the end of dissipation with SF100 and another at the end of dissipation with SF1000, while the corresponding horizontal displacement profiles at the mid-depth of the sandy layer, evaluated at the time steps corresponding to 90% excess pore pressure dissipation, are presented in Figure 14. The results indicate a noticeable scatter in the predicted horizontal displacements between the two scaling approaches. The SF1000 model predicts horizontal displacements ranging from approximately 0.25 m to 0.15 m along the model length, whereas the SF100 simulations produce significantly larger displacements, varying between approximately 0.7 m and 0.6 m.

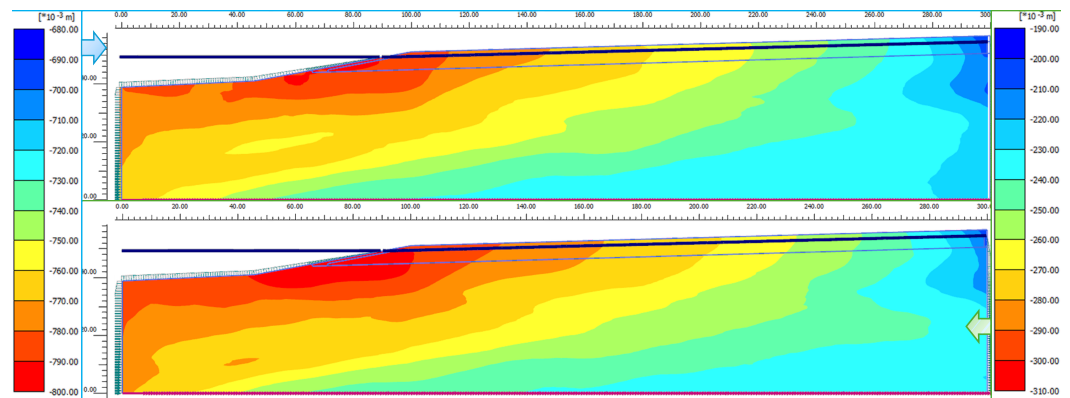


Figure 13. Contours of horizontal displacements after dissipation with SF100 (top) and SF1000 (bottom).

This behaviour highlights the sensitivity of the predicted horizontal ground deformations to the adopted hydraulic conductivity scaling factor and the associated excess pore pressure dissipation characteristics in the numerical analyses. Slight residual velocity imbalances at the end of shaking may propagate during the dissipation phase, potentially explaining why longer dissipation periods associated with lower scaling factors result in larger horizontal displacements.

Although the displacement magnitudes differ considerably, both scaling approaches exhibit a similar spatial trend characterized by a gradual increase in horizontal displacement toward the right side of the model. The displacement profiles are affected near the model boundaries due to the imposed boundary conditions, while the central portion of the model remains approximately linear, with an average displacement gradient of $\sim 3 \times 10^{-4}$ m/m. This trend suggests that significant lateral spreading displacements are expected within a zone of approximately 500–1500 m from the initiation area of lateral spreading, which is consistent with observations at the Brest Pokupski settlement, located approximately 400–900 m away, where pronounced lateral spreading-induced deformations were recorded.

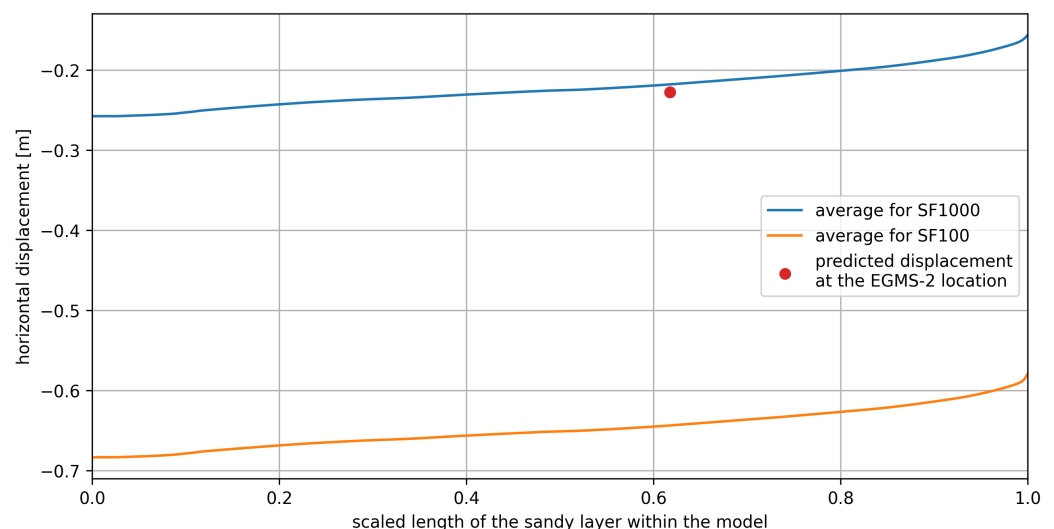


Figure 14. Horizontal displacement of the sandy layer along the model for different SF.

In the area where deformations occurred, the geodetic benchmark (GB-1) recorded a total displacement of 38.9 cm towards the southeast, corresponding to an azimuth of 133.96° . The numerical model represents a cross-section perpendicular to the river course, thus having an azimuth of approximately 30.19° . By resolving the benchmark displacement into

its components, the east–west displacement is 28.0 cm, while the component in the direction of the analysed profile is 9.3 cm. Figure 15 shows two EGMS points; one located near the benchmark (EGMS-1) and the other closer to the river (EGMS-2), within the modelled area. The east–west displacement of EGMS-1 is 28.8 cm, which is in good agreement with the benchmark measurement, thereby confirming its reliability. Similarly, EGMS-2 exhibits an eastward displacement of 26.5 cm, but the displacement component perpendicular to the river is not available. Using the derived displacement gradients, the displacement towards the river at the location of EGMS-2 was estimated to be 22.8 cm, for a projected distance of approximately 450 m between EGMS-2 and GB-1. Combined with the measured eastward displacement, this corresponds to a total displacement of 49.5 cm towards an azimuth of 147.62° . Expectedly, the displacement towards the river increases closer to the riverbank, causing the resultant displacement vector to rotate approximately 14° southwards. The approximate location of the EGMS-2 point within the model is also marked on Figure 14, showing the previously presumed displacement, for comparison with the model results.

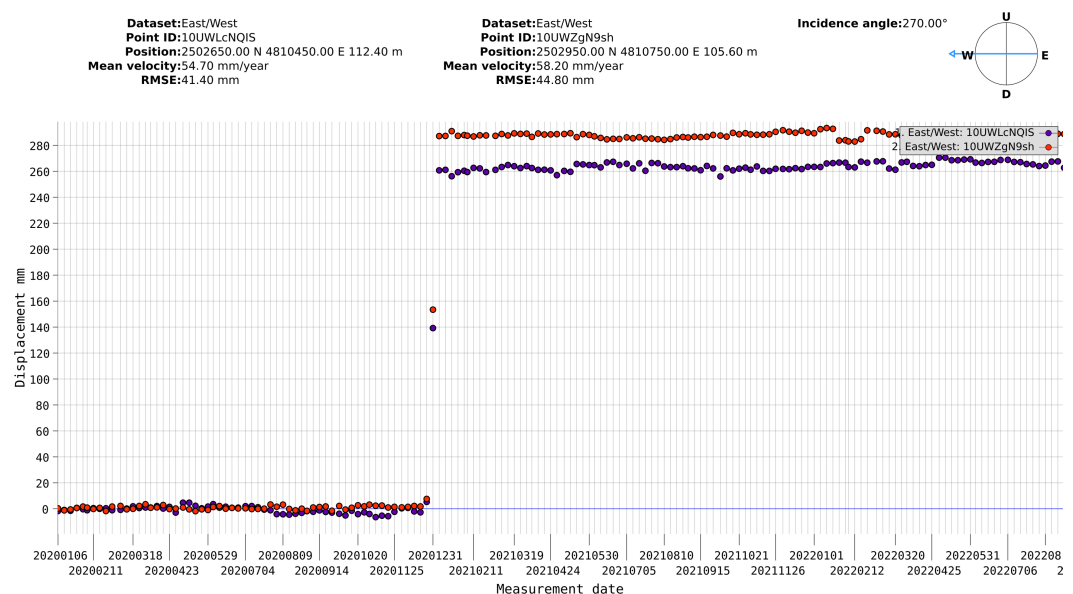


Figure 15. East–west movement components for Petrinja earthquake obtained from EGMS service ORTHO (Level 3) data set.

6. Conclusions

The 2020 Petrinja earthquake triggered widespread liquefaction and lateral spreading phenomena in the alluvial deposits along the Kupa River floodplain, causing significant ground deformations and damage to infrastructure and residential areas. Numerous manifestations of lateral spreading, including ground cracking, settlements, and horizontal displacements toward the river, were observed in the near-source region following the earthquake. The study employed fully coupled dynamic numerical analyses to simulate the generation and dissipation of excess pore water pressures and the resulting lateral spreading displacements within a gently sloping sandy deposit. The final horizontal displacements, as well as the displacement gradients with distance along the profile, were evaluated at the stage corresponding to 90% excess pore pressure dissipation. The results showed that excess pore pressure dissipation strongly influences the predicted magnitude of lateral spreading, potentially due to slight numerical imbalances in the velocity at the end of shaking, which propagate in the dissipation phases, with dissipation times ranging from approximately ~ 80 – 120 s for the SF1000 simulations and ~ 340 – 440 s for the SF100 simulations. The numerical analyses produced horizontal displacement profiles characterized by an approximately linear trend within the central part of the model domain, with an

average displacement gradient of approximately 3×10^{-4} m/m. The predicted horizontal displacements varied for different SF simulations, demonstrating a strong sensitivity of the results to the adopted scaling factor and associated dissipation characteristics. However, the displacement gradients remained consistent across the different SF realizations. Therefore, they can be reliably used to predict displacements along the analysed profile, provided that the displacement is known at at least one location. In our case, the comparison is made through a geodetic benchmark point (GB-1) and two EGMS points (EGMS-1, EGMS-2), showing an expected shift in total displacement direction towards the river for locations closer to it. In the analysed case the imbalance is counteracted by the scaling factor of 1000, as the displacement predicted with the GB-1 and displacement gradient method shows a good agreement with its results. However, this cannot be used as a general rule, as this effect is condition-specific. These results highlight the importance of excess pore pressure dissipation and scaling assumptions in the prediction of post-earthquake ground deformations in liquefiable alluvial environments. On the other hand, the observed displacement trends, which remain invariant with respect to hydraulic conductivity, provide a practical means of predicting displacements along the analysed profile, provided that a displacement measurement is available at some reference location.

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