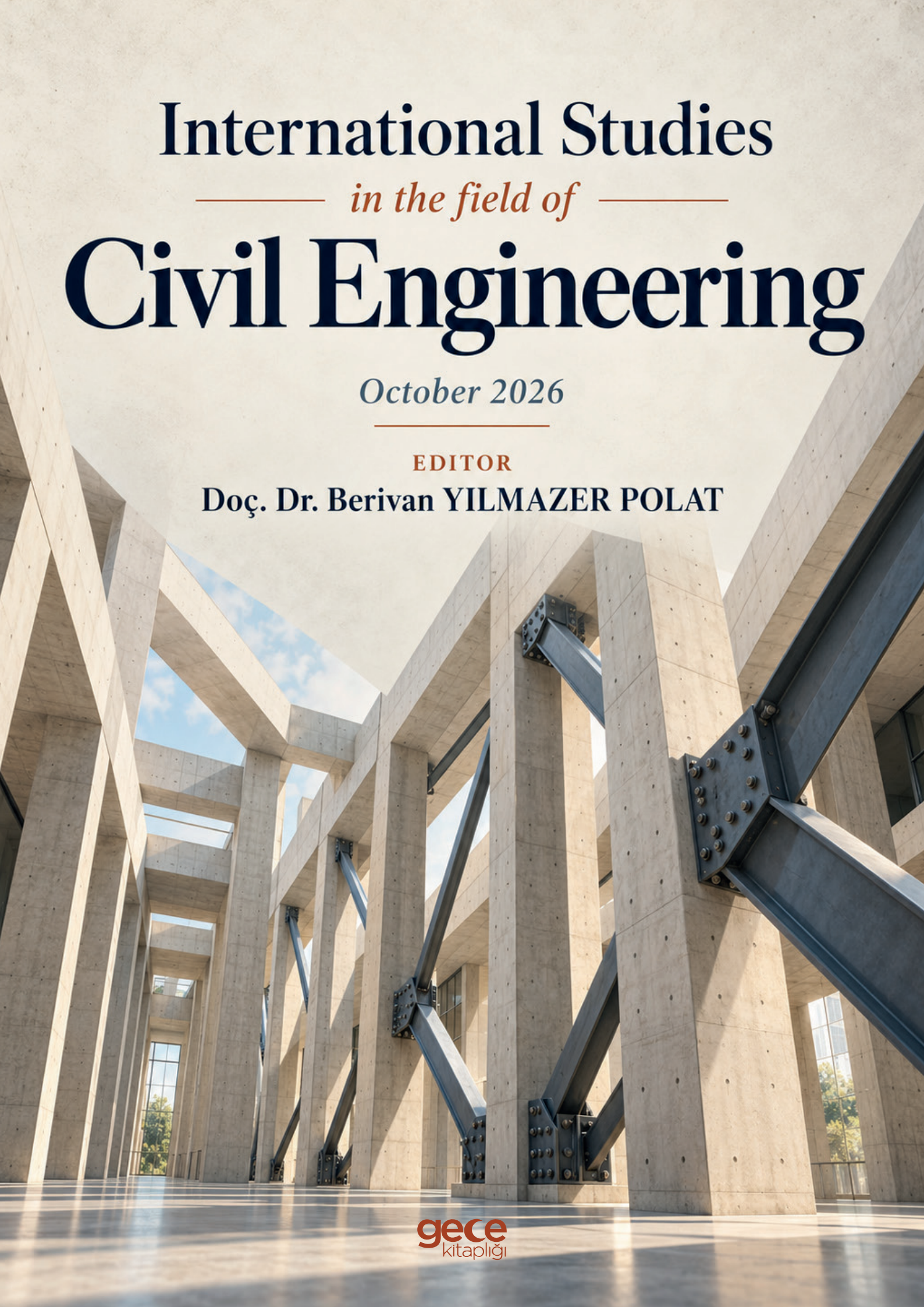




International Studies *in the field of* Civil Engineering

October 2026

EDITOR

Doç. Dr. Berivan YILMAZER POLAT

İmtiyaz Sahibi / Yaşar Hız
Yayına Hazırlayan / Gece Kitaplığı

Birinci Basım / Ekim 2026 - Ankara
ISBN / 978-625-321-321-3

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CHAPTER 1

BEYOND LINEAR SOIL SPRINGS: FORCE-DEPENDENT VARIATIONS IN THE NATURAL FREQUENCY OF MONOPILE-SUPPORTED OFFSHORE WIND TURBINES

Abdullah Kürşat DEMİR¹

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1. Introduction

Offshore wind energy has expanded rapidly in both installed capacity and turbine scale. This development has been accompanied by a shift toward larger rotor diameters, taller towers, and increasingly large monopile foundations. As support structures become more flexible and foundation diameters increase, the dynamic interaction between the tower, monopile, and surrounding soil becomes a central design issue. The first natural frequency is particularly important because it must remain separated from dominant environmental and operational excitation bands throughout the service life of the turbine. For a monopile-supported offshore wind turbine (OWT), this requirement cannot be evaluated reliably without an appropriate representation of soil–structure interaction (SSI).

The traditional engineering representation of lateral pile response is based on Winkler-type springs and nonlinear p – y curves. These methods remain attractive because of their simplicity and low computational cost, and they are embedded in widely used offshore design procedures (API, 2014; DNV, 2014). Their original calibration, however, was largely based on comparatively slender piles. Modern offshore monopiles can have diameters of several metres and relatively low length-to-diameter ratios, so their deformation mechanism increasingly resembles that of a rigid or semi-rigid pile rather than a conventional slender pile. This change in geometry makes base shear, base moment, distributed moment resistance, and shaft shear more relevant to the global response.

The PISA research programme was developed to address these limitations through large-scale field testing, three-dimensional finite element (3D FE) modelling, and a one-dimensional design framework containing four soil reaction components (Byrne et al., 2015, 2019; Zdravković et al., 2015). The resulting framework is physically richer than a conventional p – y model and has been implemented in the Monopile Designer Tool (MoDeTo) environment (Panagoulas et al., 2018; Brinkgreve et al., 2020). These developments are important not only for deformation and capacity calculations but also for dynamic assessment, because foundation stiffness directly contributes to the effective lateral stiffness that controls the natural frequency of the complete OWT system.

Many simplified frequency formulations treat foundation stiffness as constant. Under this assumption, changing the magnitude of the operational thrust changes structural deformation but does not change the natural frequency. Such an assumption is consistent with linear coupled-spring models and with any linearized p – y representation evaluated at a fixed tangent or secant stiffness. Nonlinear soil behaviour leads to a different outcome. As lateral

load increases, the force–displacement and moment–rotation curves may progressively soften; the effective foundation stiffness then decreases and the first natural frequency becomes load-dependent. This distinction is especially important when the design frequency is close to the boundary of a resonance-avoidance region.

This chapter examines the force-dependent dynamic response of a monopile-supported OWT by comparing four SSI representations: conventional p – y curves, coupled lateral–rotational springs, a PISA-based one-dimensional model, and a validated 3D FE model. The discussion focuses on sandy soil conditions and the DTU 10 MW reference turbine. Particular attention is given to mudline displacement and rotation because these quantities control the equivalent flexibility introduced by the foundation. The numerical framework is first verified against Dunkirk PISA field-test data and is subsequently used to interpret how increasing operational thrust modifies the first natural frequency and the proximity of the system to resonance-sensitive frequency ranges.

2. Natural Frequency Framework for Monopile-Supported OWTs

The dynamic design of an OWT requires separation between the structural natural frequencies and the principal excitation frequencies generated by wind, waves, rotor imbalance, and blade passage. Bhattacharya (2014) summarized this concept in terms of soft–soft, soft–stiff, and stiff–stiff design regions. For modern three-bladed turbines, the 1P excitation corresponds to rotor rotational frequency, whereas the 3P band is associated with blade-passing effects. Because variable-speed turbines operate over a range of rotational speeds, both 1P and 3P are frequency bands rather than single values.

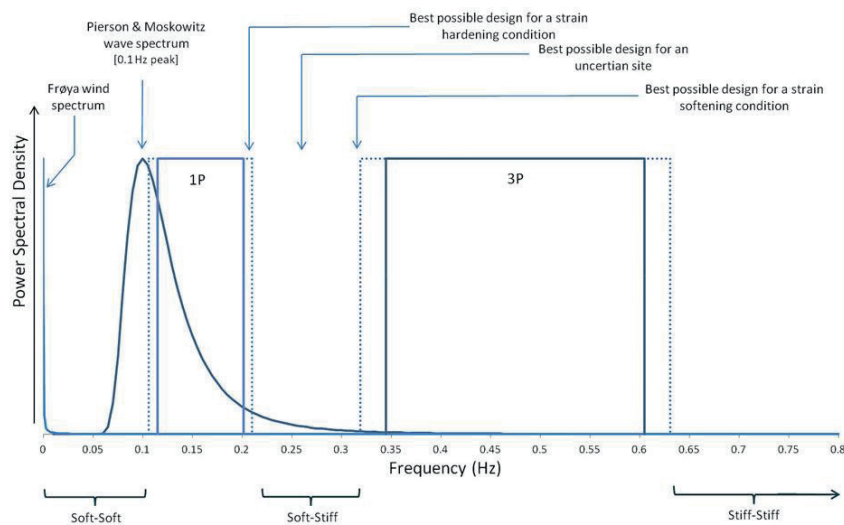


Figure 1. Frequency spectrum of dynamic loads and typical design regions (Bhattacharya, 2014).

2.1. Excitation bands and resonance-avoidance regions

Rotor-induced excitation is commonly described through the 1P and blade-passing frequency bands. The 1P component is associated with rotor rotation and may arise from mass or aerodynamic imbalance, whereas the 3P component of a three-bladed turbine is related to the repeated passage of blades and the corresponding periodic aerodynamic disturbance. Because modern offshore wind turbines operate over a range of rotational speeds, these excitations occupy frequency bands rather than single values. Wave loading introduces another frequency range that may overlap the low-frequency structural response. The purpose of preliminary dynamic design is therefore not merely to maximize stiffness, but to place the first structural natural frequency in a range that avoids persistent coincidence with the principal excitation bands (Bhattacharya, 2014).

Three broad design regions are generally distinguished. In a stiff-stiff configuration, the structural frequency lies above both the dominant wave and rotor excitation bands; although dynamically robust, this solution may require a comparatively heavy support structure. In a soft-soft configuration, the first natural frequency lies below the relevant excitation bands, creating a highly flexible system and potentially increasing sensitivity to low-frequency environmental loading. Between these extremes, the soft-stiff region places the structural frequency above the dominant wave range but below the 3P band and is frequently targeted for monopile-supported OWTs because it provides a practical balance between structural weight and resonance avoidance. The exact boundaries remain turbine- and site-specific, which makes accurate prediction of foundation stiffness essential whenever the calculated frequency approaches a design boundary.

At preliminary design level, the first mode of a monopile-supported OWT can be interpreted through an equivalent single-degree-of-freedom representation. If m_{eff} is the effective modal mass and k_{eff} is the equivalent lateral stiffness, the undamped first natural frequency can be written as

$$f_1 = \frac{1}{2\pi} \sqrt{\frac{k_{eff}}{m_{eff}}} \quad (1)$$

This compact expression shows why the foundation model is dynamically important: any SSI-induced reduction in effective stiffness shifts the natural frequency downward. For a fixed-base idealization, the tower and exposed monopile can be treated as a stepped cantilever and their flexibility obtained through strain-energy methods such as Castigliano's theorem (Arany et al., 2016). The tower mass is commonly condensed through an effective-mass

coefficient, while the rotor–nacelle assembly is represented as a concentrated mass near hub level. The corresponding fixed-base system provides a useful reference but does not capture mudline translation or rotation.

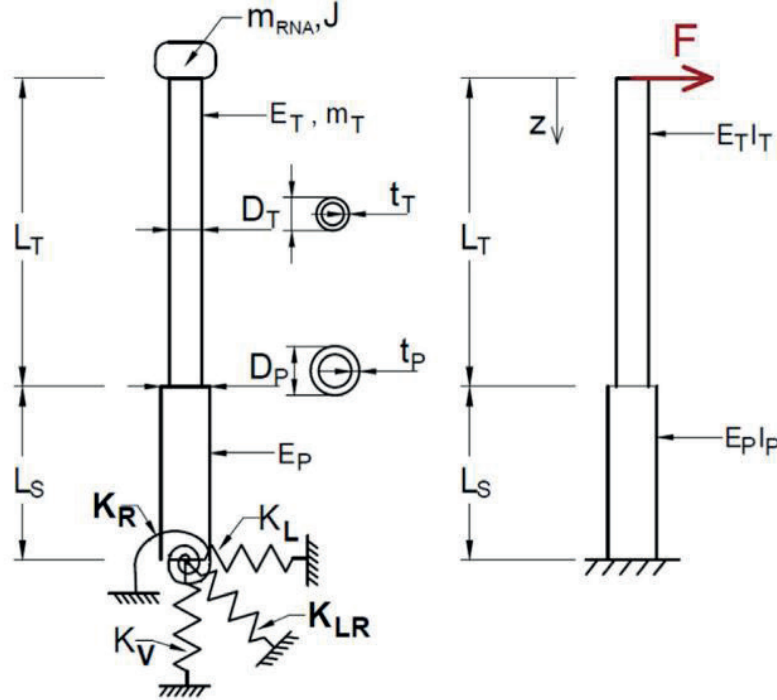


Figure 2. Idealized variable-section cantilever representation of the OWT support structure (after Arany et al., 2016).

When SSI is included, the top displacement contains three components: structural bending, mudline translation, and the additional displacement caused by mudline rotation. In a simplified form, the top displacement y_t under a lateral hub load F may be expressed as

$$y_t = y_0 + L_t \sin(\Psi_0) + \frac{\partial V_c}{\partial F_b} \quad (2)$$

where y_0 denotes the deformation at the mudline, Ψ_0 represents the rotation at the mudline, and $\partial V_c / \partial F_b$ corresponds to the deformation at the tower top induced by a force applied at the same location. Jiang et al. (2023) used an equivalent-flexural-rigidity concept in which the SSI-supported system and an equivalent fixed-base system are required to have the same top displacement under the same load. The equivalent stiffness can therefore be interpreted as the inverse of the total flexibility contributed by structural bending and foundation deformation.

The consequence for natural-frequency estimation is direct. If y_0/F and ψ_0/F remain constant, foundation flexibility is constant and the predicted frequency is independent of load magnitude. If either ratio increases with load because the soil response becomes nonlinear, the effective stiffness decreases and Eq. (1) predicts a lower frequency. In large-diameter

monopiles, rotation can be especially important because the lever arm between mudline and hub magnifies its contribution to top displacement. A relatively small change in mudline rotation may therefore produce a substantial change in the equivalent structural flexibility.

3. Soil–Structure Interaction Modelling

SSI models used for monopile-supported OWTs can be grouped into fixed-base, apparent-fixity, coupled-spring, and distributed-spring approaches (Bush & Manuel, 2009; Alkhoury et al., 2020). The present comparison excludes apparent fixity and focuses on methods that explicitly represent the soil response at or below the mudline. The four approaches discussed here differ principally in the number of resistance mechanisms represented and in whether their stiffness is constant or load-dependent.

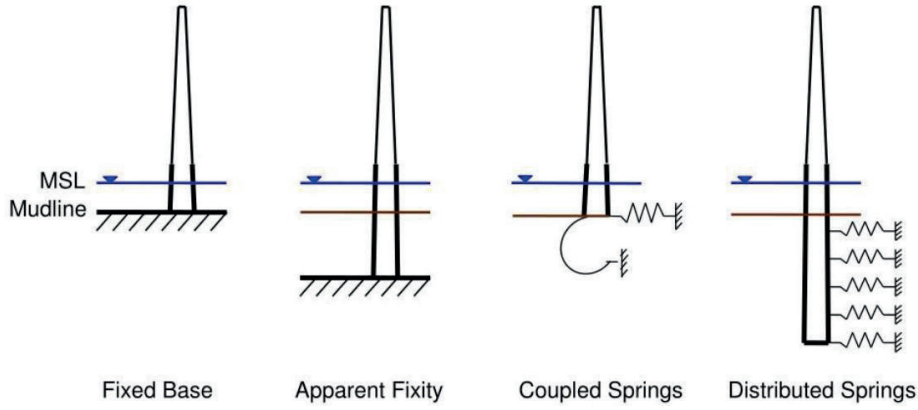


Figure 3. Representative SSI modelling approaches for monopile-supported OWTs (after Bush & Manuel, 2009).

3.1. Coupled lateral–rotational springs

In the coupled-spring approach, the soil is condensed at the mudline into lateral, rotational, and lateral–rotational coupling components. Vertical stiffness may be neglected when vertical deformation has a negligible influence on the first fore–aft mode. The mudline displacement and rotation can be written in compliance form as

$$\begin{Bmatrix} y_0 \\ \Psi_0 \end{Bmatrix} = \begin{bmatrix} \delta_L & \delta_{LR} \\ \delta_{LR} & \delta_R \end{bmatrix} \begin{Bmatrix} F_0 \\ M_0 \end{Bmatrix} \quad (3)$$

where δ_L is the lateral compliance, δ_R is rotational compliance, and δ_{LR} is the lateral–rotational coupling compliance. Classical formulations by Poulos and Davis (1980) provide these terms as functions of pile geometry and soil stiffness. Because the coefficients are constant for a selected soil profile, the method produces a linear force–deformation relationship. The effective foundation stiffness, and hence the natural frequency, therefore remain essentially constant with increasing lateral thrust.

3.2. Conventional p - y curves

The conventional p - y method represents the soil by independent nonlinear lateral springs distributed along the pile. It is computationally efficient and remains widely used in design practice. For large-diameter monopiles, however, it does not explicitly include all three-dimensional resistance mechanisms that develop around short or intermediate piles. In particular, distributed moment resistance, base shear, and base moment can materially affect the response. Consequently, a p - y model can provide a different mudline stiffness from a PISA-based or 3D FE representation even when the same global soil profile is considered.

The limitation becomes more pronounced as the monopile becomes relatively short. Short piles rotate more nearly as rigid bodies and mobilize a meaningful fraction of their resistance near the pile tip. Byrne et al. (2015) showed that conventional API/DNV p - y curves do not reproduce the complete response of large-diameter monopiles in sand. For the configurations examined in the PISA programme, the traditional formulation tended to produce a stiffer response than the more complete 1D and 3D representations over important ranges of lateral deformation.

3.2.1. Influence of monopile slenderness on lateral response

The distinction between long- and short-pile behaviour is important for interpreting why a conventional p - y model may depart from PISA-based or continuum solutions. For a relatively long pile, lateral deformation is concentrated in the upper part of the embedded shaft and the influence of the pile tip becomes progressively smaller. By contrast, a short pile can rotate more nearly as a rigid body about a point within the soil mass, so that toe shear, toe moment, and shaft-related moment reactions contribute more strongly to the overall resistance. As turbine capacities have increased, monopile diameters have grown faster than embedment lengths, and modern foundations have therefore moved toward lower L/D ratios where these mechanisms become increasingly relevant (Byrne et al., 2015).

This mechanical transition explains the additional reaction components introduced in the PISA framework. A model based only on distributed lateral resistance can reproduce one important part of the soil reaction, but it does not explicitly represent distributed moment resistance generated by vertical shear along the pile wall, nor the base shear and base moment mobilized at the pile tip. In sandy soil, the omission is particularly important for large-diameter piles. The PISA field programme showed that conventional API/DNV-type p - y curves may predict a substantially different force-deformation response from the 1D PISA and 3D FE solutions. Consequently, the apparent stiffness ranking of alternative SSI models is not a

universal material property; it depends on pile geometry, soil state, deformation level, and the set of resistance mechanisms included in the formulation.

3.3. PISA-based one-dimensional model

The PISA one-dimensional model extends the distributed-spring concept by introducing four separate reaction mechanisms: distributed lateral load, distributed moment, base shear, and base moment (Byrne et al., 2015). The distributed lateral reaction is conceptually similar to a p-y spring, whereas the distributed moment accounts for vertical shear stresses around the pile wall. Base shear and base moment represent additional resistance mobilized at the pile tip. This decomposition allows short-pile behaviour to be represented without the computational cost of a full 3D continuum model.

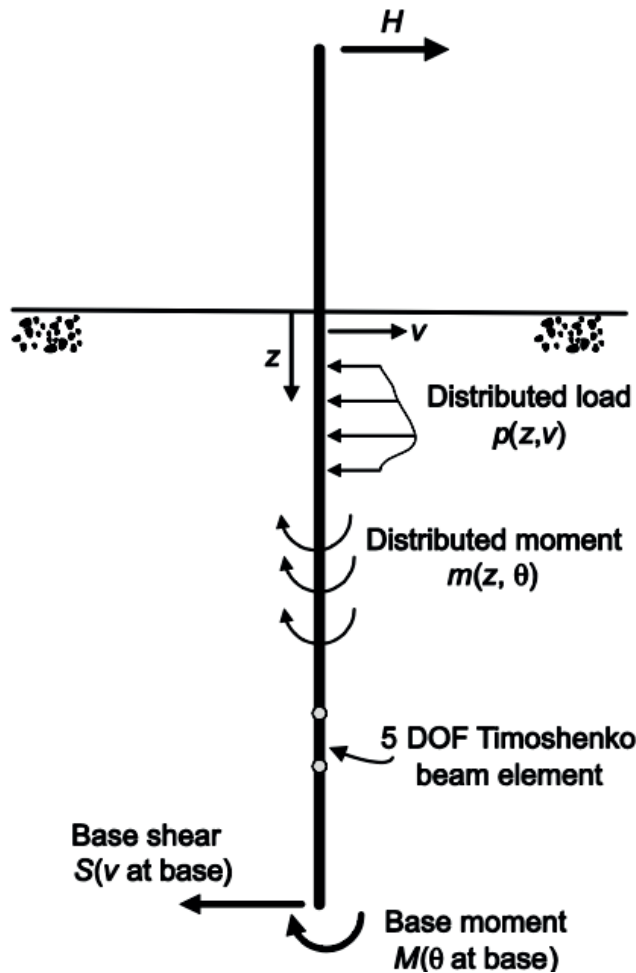


Figure 4. Four-reaction framework of the PISA one-dimensional monopile model (after Byrne et al., 2015).

The PISA curves are nonlinear. Their tangent and secant stiffnesses therefore evolve as load and deformation increase. This feature is critical for dynamic interpretation: the natural frequency calculated from an equivalent stiffness is no longer a single immutable value for the structural configuration, but may change over the operational load range.

3.4. Three-dimensional finite element model

A 3D FE model provides the most complete representation considered in this chapter because it resolves continuum soil deformation and pile–soil contact directly. The analyses were performed using PLAXIS 3D 2023.2 with the MoDeTo workflow. The pile was represented using the software’s monopile modelling environment, with local mesh refinement around the shaft and pile tip. The soil response was described with the Hardening Soil model with small-strain stiffness (HSsmall), allowing stiffness degradation from the small-strain range to larger operational deformations. This nonlinear stiffness evolution is the mechanism through which increasing thrust affects the calculated natural frequency.

4. Numerical Verification and Application

4.1. Verification against Dunkirk PISA field tests

Before applying the 3D FE model to the turbine-scale problem, the numerical procedure was checked against the Dunkirk field tests used in the PISA programme. Two reference piles were considered: DM4 with diameter 0.762 m and embedded length 3.98 m, and DL1 with diameter 2.0 m and embedded length 10.61 m (Byrne et al., 2015). The groundwater level at the Dunkirk site was taken as 5.4 m below ground surface. Layer-dependent HSsmall parameters followed the calibration reported by Minga and Burd (2019), including relative density, unit weight, friction angle, small-strain shear stiffness, reference secant stiffness, unloading/reloading stiffness, and small-strain threshold parameters.

The MoDeTo-generated 3D mesh used 10-node tetrahedral elements. Because of axisymmetry, a half-pile representation was adopted for the verification simulations. The circumference was discretized into sectors, and the mesh was refined close to the pile where large stress and strain gradients occur. An interface formulation was used between pile and soil in accordance with the MoDeTo recommendations and the Dunkirk calibration adopted in the source study.

The verification model retained the layered nature of the Dunkirk profile used by Minga and Burd (2019). The groundwater table was located 5.4 m below ground level. In the upper 0–3 m layer, the reported relative density was 100%, with $K_0 = 0.4$, saturated unit weight 17.1 kN/m^3 , reference cohesion 5 kPa, and effective friction angle 44° . From 3 to 5.4 m, the relative density was 75%, $K_0 = 0.4$, saturated unit weight 17.1 kN/m^3 , reference cohesion 10 kPa, and effective friction angle 40° . Below the groundwater level, the 5.4–10 m and 10–20 m layers were represented with relative density 75%, $K_0 = 0.4$, saturated unit weight 19.9 kN/m^3 , very small reference cohesion (0.1 kPa), and an effective friction angle of 40° . These values were

implemented using the HSsmall constitutive model so that both stress-dependent stiffness and the elevated small-strain shear stiffness of dense sand were represented.

The pile-soil interface and mesh treatment were also kept consistent with the MoDeTo verification procedure. A half-model was used because of geometric symmetry, and the embedded pile circumference was divided into nine 20° sectors. Mesh density was increased around the pile shaft and below the tip, where the strongest stress gradients and deformation localization were expected. The interface parameters followed the MoDeTo recommendations adopted in the original verification work, including an effective interface friction angle of approximately 29° and zero dilation. Agreement with the field response was therefore used not as a calibration to the DTU turbine case, but as an independent check that the continuum modelling procedure could reproduce the measured lateral behaviour of piles in Dunkirk sand before being transferred to the larger-scale application.

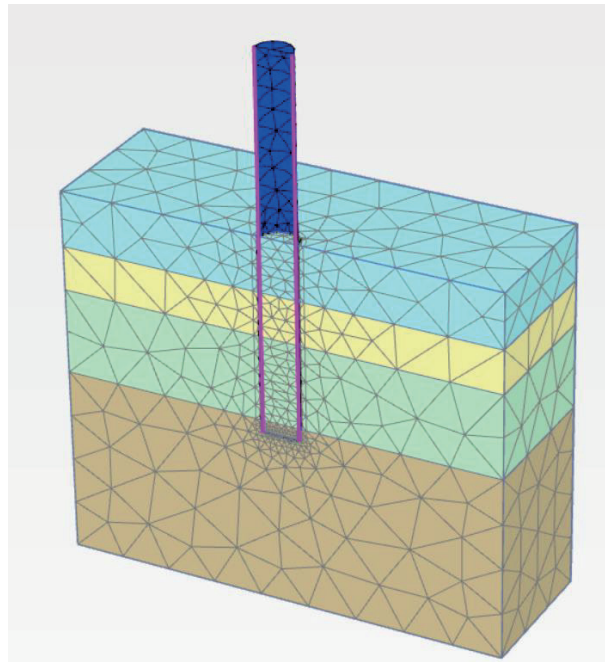


Figure 5. PLAXIS 3D representation of the Dunkirk field-test configuration and local mesh refinement.

The calculated load–displacement response was then compared with the PISA field measurements. The numerical curves reproduced the measured mudline response with good agreement over the loading range used for verification. This comparison supports using the same modelling strategy for the subsequent DTU 10 MW analyses, where the objective is not

to reproduce the field-test geometry but to quantify how nonlinear foundation stiffness influences the system frequency.

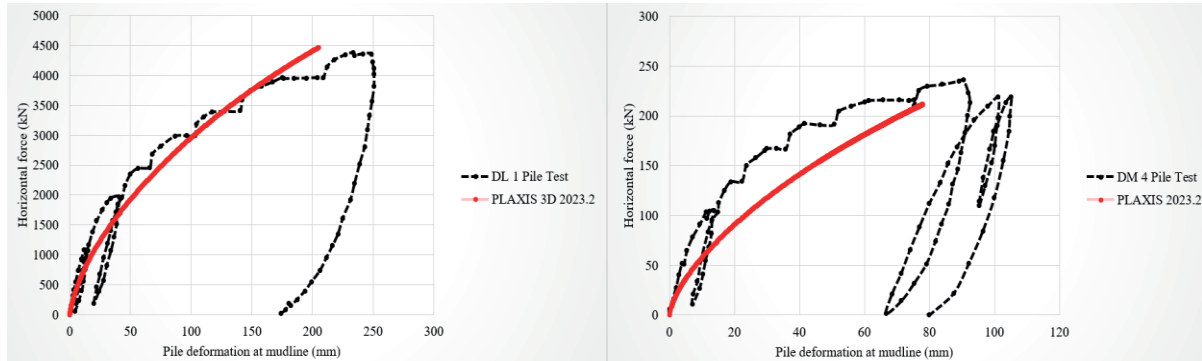


Figure 6. Comparison between PISA field-test response and the PLAXIS 3D verification model

4.2. DTU 10 MW reference turbine and loading

The application case is based on the DTU 10 MW reference wind turbine as adopted by Alkhoury et al. (2020). The turbine has a rated power of 10 MW, a rotor diameter of 178.332 m, and a hub height of 119 m. The cut-in, rated, and cut-out wind speeds are 4, 11.4, and 25 m/s, respectively, and the rotor operates from approximately 6 rpm to 9.6 rpm over the principal operating range. The tower and monopile geometry were represented consistently across the SSI alternatives so that differences in the calculated frequency arise from the foundation modelling rather than from changes in the superstructure.

Table 1. Selected properties of the DTU 10 MW reference turbine used in the application.

Property	Value
Rated power	10 MW
Rotor diameter	178.332 m
Hub height	119 m
Cut-in / rated / cut-out wind speed	4 / 11.4 / 25 m/s
Cut-in / rated rotor speed	6 / 9.6 rpm

Loose sand was selected as the reference condition for comparison with the study of Alkhoury et al. (2020). Their sand profiles were defined using relative densities of 30%, 50%, and 80% for loose, medium-dense, and very-dense conditions. In the present application, the operational thrust history of the DTU 10 MW turbine was used to define the lateral loading

range. The maximum operational thrust was taken as 1,650 kN. With the adopted structural height, this corresponds to a maximum mudline moment of approximately 254,100 kN·m.

For each load level, the mudline lateral displacement and rotation were obtained from the selected SSI representation. These values were introduced into the equivalent-flexibility framework to calculate a load-specific effective stiffness and first natural frequency. In the coupled-spring and conventional p - y solutions, the response over the examined operating range was effectively linear. In the PISA-based 1D and PLAXIS 3D solutions, the incremental stiffness decreased with increasing load, allowing the frequency to vary continuously with turbine thrust.

5. Results

5.1. Mudline deformation response

The comparison of SSI models reveals a fundamental distinction between linear and nonlinear representations. The coupled-spring model gives proportional increases in displacement and rotation as the lateral force increases. The conventional p - y solution used in the comparison also behaves almost linearly over the operational load range. Because the slopes of these response curves remain nearly constant, the effective stiffness entering the frequency calculation remains essentially unchanged.

The PISA-based 1D and 3D FE curves show a different pattern. As the applied load increases, displacement and rotation grow at an increasing rate. The reduction in force–displacement and moment–rotation slopes indicates progressive foundation softening. Since both mudline translation and mudline rotation contribute to the equivalent top displacement, this softening reduces the effective lateral stiffness of the overall turbine–foundation system.

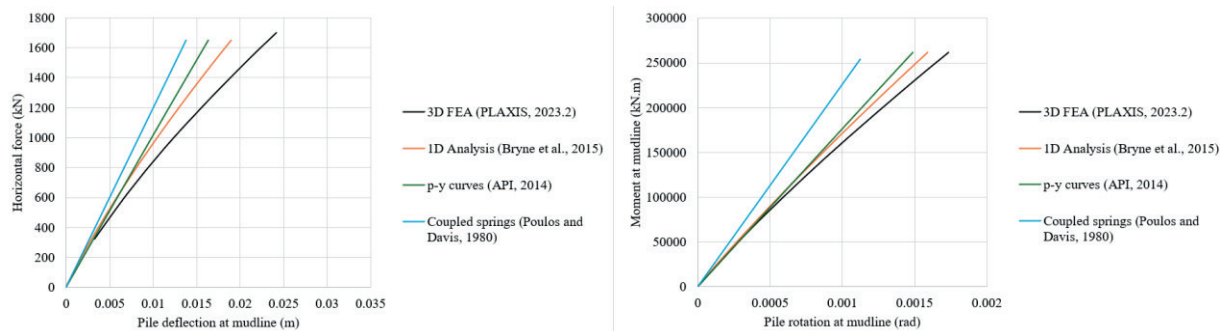


Figure 7. Mudline displacement and rotation obtained using different SSI models over the operational loading range.

The role of rotation is particularly important. Even when lateral displacement remains within a conventional serviceability limit, the rotational contribution is amplified by the distance from mudline to hub. Consequently, a model that reproduces lateral displacement reasonably well but underestimates rotation can still overestimate the effective dynamic stiffness and the first natural frequency.

5.2. Load-dependent natural frequency

The resulting natural-frequency curves confirm the direct relationship between nonlinear SSI and load-dependent dynamics. The coupled-spring solution remains approximately constant at 0.183 Hz, while the conventional p - y solution remains approximately constant at 0.181 Hz. In contrast, the PISA-based 1D model varies between about 0.182 and 0.180 Hz, and the 3D FE model varies from approximately 0.181 to 0.178 Hz as the operational load increases. The magnitude of the change is modest in absolute terms, but it is structurally significant because the initial frequency lies close to a lower resonance-avoidance boundary.

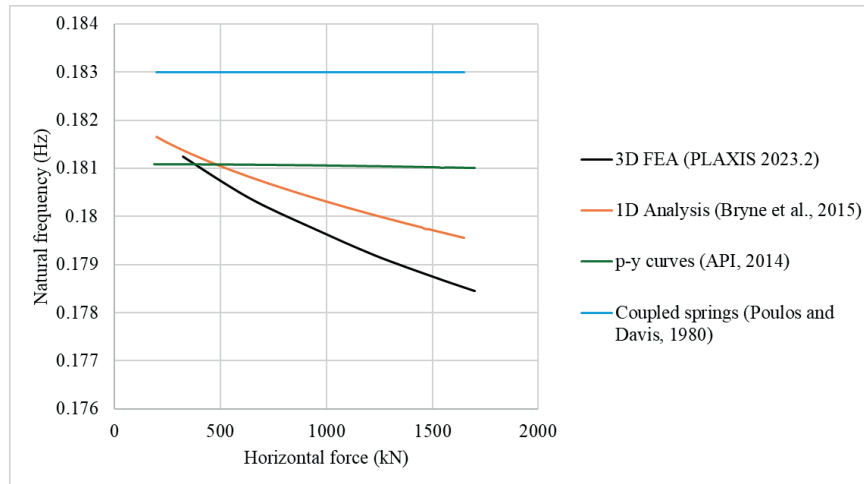


Figure 8. First natural frequency as a function of operational load for the different SSI representations.

Table 2. First natural frequency obtained with different SSI methods.

SSI method	Present application	Alkhoury et al. (2020)	Interpretation
Coupled springs	0.183 Hz	0.181 Hz	Essentially load-independent
p–y curves	0.181 Hz	0.188 Hz	Essentially load-independent
PISA-based 1D	0.180–0.182 Hz	Not available	Load-dependent
3D FE (PLAXIS 2023.2)	0.178–0.181 Hz	0.201 Hz	Load-dependent

The relative ordering of the methods is also important. For the sandy-soil, large-diameter configuration examined here, the conventional p–y approach is stiffer than the PISA-based 1D and 3D FE solutions over much of the response range, leading to a higher frequency estimate. This trend is consistent with the PISA interpretation of short monopiles in sand and with the comparison reported by Demir et al. (2024), but it differs from the numerical ordering reported by Alkhoury et al. (2020). The discrepancy indicates that model assumptions, soil constitutive behaviour, stiffness calibration, and the displacement range over which stiffness is evaluated can materially affect the frequency comparison.

A small-strain stiffness estimate should therefore not be interpreted as representative of the full operational load range. At very small displacement, the tangent stiffness of the 3D model is relatively high. As deformation increases, the HSsmall model captures stiffness degradation and the force–deformation curve becomes flatter. If only the near-origin slope were used, the inferred natural frequency would be higher than the value corresponding to the larger operational deformation. This explains why load-independent stiffness models can conceal the approach toward a resonance-sensitive region.

5.2.1. Initial stiffness versus operational stiffness

The distinction between initial and operational stiffness is central to the interpretation of natural frequency. At very small displacement, the force-deflection curve is steep and a tangent stiffness evaluated close to the origin can be considerably higher than the secant stiffness mobilized at an operational load level. If the initial tangent is used throughout the loading range, the foundation is effectively treated as if the soil retained its small-strain rigidity even after appreciable pile translation and rotation have developed. The resulting frequency can

therefore be biased upward. This issue is especially relevant when a coupled-spring representation is back-calculated from small-amplitude measurements, because the spring constants then describe only the near-origin response and do not automatically contain the stiffness degradation observed at larger deformation (Amel et al., 2022).

The comparison in this chapter also clarifies the apparent disagreement with Alkhoury et al. (2020). Their reported 3D FE natural frequency for the DTU 10 MW configuration was about 0.201 Hz, whereas the present 3D FE-based range is approximately 0.181-0.178 Hz as load increases. By comparison, the present coupled-spring and p-y solutions remain close to 0.183 Hz and 0.181 Hz, respectively. Rather than treating this difference as a simple numerical discrepancy, it should be interpreted as evidence that dynamic conclusions are strongly influenced by constitutive stiffness, soil profile idealization, load level, and the manner in which foundation flexibility is transferred into the frequency formulation. A frequency value without an accompanying definition of the relevant deformation range can therefore be misleading when different SSI approaches are compared.

5.3. Interaction with SLS and ULS criteria

The dynamic implications become clearer when deformation criteria are considered simultaneously. Published limit-state criteria for laterally loaded monopiles include rotational limits and displacement-based limits. For example, Doherty and Gavin (2012) and Arany et al. (2017) use rotation-based serviceability limits, while other studies report larger rotations or displacement fractions as ultimate criteria. In the present application, the more conservative criterion adopted from Arany et al. (2017) was used to interpret the operational response.

Table 3. Representative lateral-response criteria reported in the literature.

Reference	SLS criterion	ULS criterion
Luo et al. (2018)	—	$\theta = 2^\circ$
Klinkvort & Hedeal (2013)	—	$\theta = 4^\circ$
Zdravković et al. (2015)	—	$u = 0.1D$
Doherty & Gavin (2012)	$\theta = 0.5^\circ$	—
Arany et al. (2017)	$\theta = 0.5^\circ$ or $u = 0.2$ m	—

For the model examined, rotation is the governing response quantity. When the mudline rotation exceeds approximately 0.25° , the calculated first natural frequency from the nonlinear 3D FE-based framework falls below about 0.176 Hz, which is the lower frequency boundary used in the source analysis. At the same load level, the lateral displacement remains below its

adopted allowable threshold. The result demonstrates that a purely displacement-based serviceability check can miss a dynamically important reduction in stiffness.

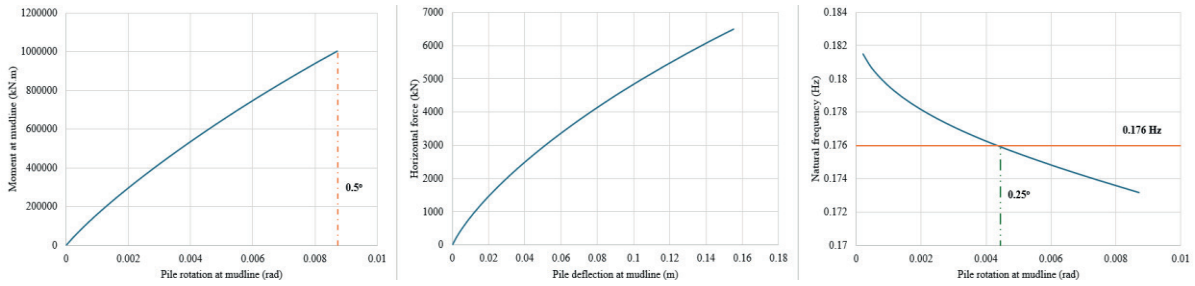


Figure 9. Mudline displacement, rotation, and corresponding natural-frequency evolution under the adopted SLS/ULS interpretation.

If the same operating condition were evaluated only with the force-independent coupled-spring or p-y solutions, the predicted frequencies of approximately 0.183 and 0.181 Hz would remain above this lower boundary. The nonlinear model, however, indicates that progressive soil softening and mudline rotation can move the system into a resonance-prone frequency range. The comparison therefore shows that a design may satisfy a static deformation criterion while simultaneously becoming less favorable from a dynamic perspective.

6. Discussion

The central finding of the comparison is not that a single SSI method should universally replace all others, but that the stiffness measure used in dynamic analysis must correspond to the deformation range being assessed. Linear spring models are valuable for preliminary calculations, sensitivity studies, and configurations that remain close to the small-strain range. Their limitation arises when a constant spring stiffness is carried into load levels at which the soil response is no longer approximately linear. In that situation, the resulting natural frequency is a property of the chosen linearization rather than of the actual load state.

For large-diameter monopiles in sand, this issue is amplified by the shift toward short- or intermediate-pile behaviour. The contribution of base reactions and distributed moment resistance becomes more significant, and mudline rotation can no longer be regarded as a secondary response. PISA-based models provide a practical intermediate level of complexity because they capture the relevant reaction mechanisms while avoiding the computational cost of repeated 3D FE analyses. Nevertheless, the PISA reaction curves themselves should be calibrated consistently with the soil conditions and pile geometry of interest.

The difference between the present 3D FE results and those reported by Alkhoury et al. (2020) also illustrates the sensitivity of dynamic conclusions to constitutive assumptions and stiffness definitions. A model that emphasizes small-strain stiffness can yield a higher apparent foundation stiffness than a model evaluated at operational deformation levels. Conversely, a nonlinear constitutive model that permits stiffness degradation may predict a lower frequency as thrust increases. Comparisons between studies should therefore report not only the resulting frequency but also the soil model, strain range, pile geometry, and method used to obtain lateral and rotational stiffness.

From a design perspective, a useful workflow is to treat the first natural frequency as a response that may vary over an operational envelope rather than as a single deterministic constant. A fixed-base or constant-spring model can be used to establish an initial estimate. Where the resulting frequency lies comfortably inside a resonance-avoidance region, this may be sufficient for screening. Where the estimate is close to a 1P, 3P, or wave-related boundary, a load-dependent SSI model should be used to check whether operational thrust, soil nonlinearity, and rotation reduce the frequency enough to compromise the intended separation margin.

A practical implication is that frequency assessment should be linked to the same loading states used for geotechnical and structural checks. A preliminary model may still employ a fixed-base or constant-spring representation to identify the approximate dynamic range, but a design located close to a resonance-avoidance boundary should be re-evaluated using a nonlinear SSI model at representative operational load levels. The resulting frequency envelope can then be compared with the 1P, 3P, and wave excitation bands rather than relying on a single unloaded or small-strain frequency. In this framework, mudline rotation deserves explicit attention because its contribution to hub-level displacement is multiplied by the large tower lever arm. Consequently, a modest increase in rotation may alter the equivalent flexibility more strongly than an apparently acceptable change in mudline translation.

The present results should nevertheless be interpreted within their modelling scope. The frequency calculation is based on an equivalent first-mode representation and does not constitute a fully coupled aero-hydro-servo-elastic simulation. Structural, aerodynamic, hydrodynamic, and soil damping are not resolved as interacting mechanisms, and the principal comparison concerns the evolution of equivalent stiffness with lateral operational loading. The results therefore support a design-screening and model-selection argument: they show when a constant foundation stiffness can conceal a load-induced shift in frequency. They do not imply

that the numerical values obtained for the selected DTU 10 MW and sand profile can be transferred unchanged to other turbines, water depths, soil profiles, or pile geometries.

The present framework remains a simplified dynamic interpretation because aerodynamic, hydrodynamic, and structural damping are not explicitly coupled into the frequency calculation, and the soil response is examined primarily through lateral quasi-static stiffness. Nevertheless, damping has a relatively small effect on the undamped natural frequency compared with stiffness changes, and the load-dependent trend observed here provides a clear mechanism for identifying cases in which a constant-stiffness assumption may be unconservative. Future work may extend the framework to cyclic degradation, layered soils, frequency-dependent soil impedance, and fully coupled aero-hydro-servo-elastic simulations.

7. Conclusions

This chapter examined how the first natural frequency of a monopile-supported offshore wind turbine changes when soil–structure interaction is represented with models of increasing mechanical fidelity. The comparison shows that the assumption of a load-independent natural frequency is a consequence of using a constant effective foundation stiffness rather than an inherent property of the turbine–foundation system.

- Coupled lateral–rotational springs and the conventional p–y representation used in the comparison produce nearly linear mudline response over the operational load range; their predicted first natural frequencies remain approximately constant.
- The PISA-based 1D model and the 3D FE model exhibit stiffness degradation with increasing load. The corresponding first natural frequency therefore decreases as turbine thrust increases.
- For the DTU 10 MW application in loose sand, the 3D FE-based frequency decreases from approximately 0.181 Hz to 0.178 Hz, while the coupled-spring and p–y estimates remain near 0.183 Hz and 0.181 Hz, respectively.
- Mudline rotation can govern the dynamic response even when lateral displacement remains within an adopted serviceability threshold. Because rotation is magnified by the hub-height lever arm, it should be included explicitly in preliminary frequency checks.
- When the calculated frequency lies close to a resonance-avoidance boundary, nonlinear and load-sensitive SSI modelling provides a more defensible basis for design than a stiffness value derived only from the small-strain range.

As offshore wind turbines continue to increase in size, the distinction between initial stiffness and operational stiffness will become increasingly important. Incorporating load-dependent SSI into dynamic assessment can improve resonance-avoidance checks and provide

a clearer connection between geotechnical deformation criteria and the global structural response of monopile-supported OWTs.

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CHAPTER 2

OPEN BIM AND GIS STANDARDS FOR DISASTER RESILIENT DECISION MAKING IN CONSTRUCTION PROJECTS

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1. Introduction

The Sendai Framework provides a foundation for understanding disaster risk, strengthening resilience and reducing disaster risk (UNDRR, 2015). In construction management, disaster resilience raises the question of which practices are critical before, during and after a disaster. Marzouk and Al Daour (2018) link site evacuation performance to construction sequencing, while Hosseini and Maghrebi (2021) integrate time-dependent building information with fire risk assessment and evacuation simulation. Deng et al. (2022) propose a framework for determining evacuation and response routes on construction sites. These studies highlight the importance of information technologies for disaster management and response in construction.

BIM and GIS offer substantial capabilities for interoperability. BIM provides detailed representations of project objects and their relationships, while GIS connects these objects to terrain, transport networks and other spatial conditions. Reviews by Liu et al. (2017), Song et al. (2017), and Sarigul and Gunaydin (2026) outline the scope of integration between the two technologies. For construction-stage decisions, the key relationship is between changing project conditions and the external systems on which the project depends. An internal escape route provides insufficient information for decision-making if it ends at an inaccessible gate. Similarly, a site that is accessible from outside may still contain unavailable stairs or obstructed circulation spaces.

However, BIM–GIS integration requires more than displaying models together. IFC–CityGML studies identify differences in semantics, geometry and levels of detail, while GeoBIM benchmark studies reveal inconsistencies in how software applications process standardized data (Deng et al., 2016; Donkers et al., 2016; Noardo et al., 2020, 2021a, 2021b). A visually convincing scene can therefore conceal missing attributes, incorrect positioning or relationships that do not support the intended analysis. Open BIM and GIS standards are important because they define information structures that enable data exchange and allow implementations to be examined; they do not, by themselves, make a visualization reliable.

This chapter investigates how open BIM and GIS standards support disaster-resilience decision-making during construction. It addresses this question by examining the relationship between a standard's information-exchange capability, the decision-support function enabled by that capability, and the potential contribution to resilience of the action taken in response to the decision. Figure 1 represents the workflow of BIM-GIS integration for disaster-resilience decision support during construction.

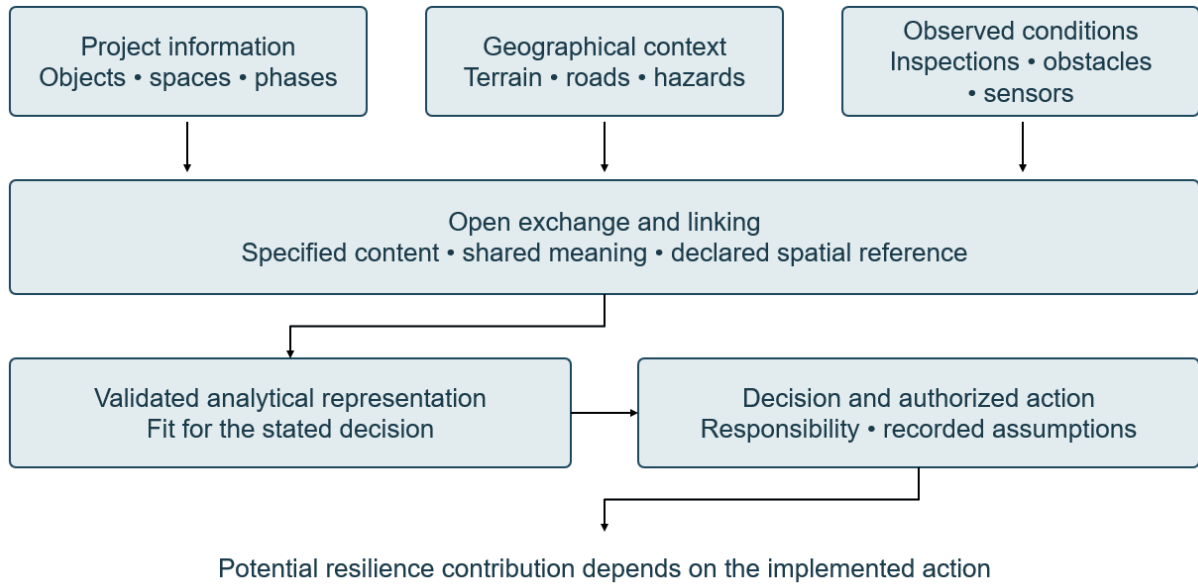


Figure 1. Workflow of BIM-GIS integration for disaster-resilience decision support during construction

Note. Original conceptual synthesis informed by Noardo et al. (2020), Patacas et al. (2020), and Mallela and Bhargava (2025).

2. BIM and GIS Applications on Construction and Disaster Management Context

Marzouk and Al Daour (2018) combine BIM, scheduling and agent-based simulation to compare construction-method alternatives. Their contribution is the inclusion of evacuation performance within a management choice that also considers cost and duration. The study supports phase-specific scenario comparison; it does not isolate the effect of an open exchange standard. Similarly, Hosseini and Maghrebi (2021) combine 4D BIM, social-force modelling and quantitative fire risk assessment, considering both emergency occurrence and evacuation performance. This provides a richer basis for comparing scenarios than an evacuation-time estimate considered in isolation, but remains dependent on the study's hazard and behavioral assumptions.

Deng et al. (2022) integrate BIM, computer vision and semantic-web reasoning to account for obstacle accessibility. Their construction site test is particularly relevant to the distinction between model availability and physical traversability. The authors identify further testing in complex environments and refinement of obstacle classifications and accessibility coefficients as research needs. Yang et al. (2022) combine monitoring, visualization, staged alarms and escape-route optimization in a construction site fire management platform. Zavari et al. (2022) integrate BIM and GIS for dynamic temporary facility placement, using travel

distance and safety-related proximity preferences as objectives. This provides a construction management example of spatially informed alternative selection. A proximity preference, however, is not a measured disaster-loss outcome. For this chapter, the study supports the feasibility of phase-dependent spatial optimization, while the addition of flood exposure, earthquake-related access loss or emergency-service constraints would require a separate model and validation.

2.1. Applications with an exchange mechanism

Mirahadi et al. (2019) provide a more direct connection between an open information structure and an analytical function. Their EvacuSafe framework uses IFC-based integration with fire and agent-based simulations to evaluate routes and compartments under fire scenarios. The case concerns an office building rather than an active construction site. It demonstrates how IFC-derived information can participate in a performance-based analytical workflow; transferring the approach to construction requires temporary and incomplete conditions to be represented explicitly.

Teo and Cho (2016) derive a geometric network from IFC and connect interior paths to exterior streets through entrance connections. The resulting network supports indoor–outdoor route planning. This is important for construction because emergency movement crosses administrative and modelling boundaries: work zone, building, site gate and public road. However, an IFC input does not imply that every downstream component is an open-standard implementation. Their use of a GIS geodatabase also illustrates why the exchange chain should be documented interface by interface rather than assigned a single open or closed label.

Sebastian et al. (2013) investigate IFC–CityGML exchange for low-disturbance bridge construction, including an application domain extension for relevant properties. The work connects model transformation with planning consequences, while identifying coordinate, conversion and property-consistency issues. Cengiz et al. (2016) situate BIM–GIS integration within construction logistics. These studies are important because they address construction activities within a spatial context

2.2. Adjacent evidence and its appropriate use

Cao et al. (2023) synthesize BIM-GIS applications in the fields of disaster prevention, response, and recovery. Their review helps identify analytical functions and integration challenges while also encompassing environments and hazards beyond the construction itself.

Demirkesen et al. (2023) address BIM-GIS disaster management within smart cities. Hong et al. (2024) developed an open disaster information platform for high-rise and complex facilities that integrates BIM, GIS, and monitoring. Their discussion distinguishes the need to model dynamic interactions between hazards from existing, predefined hazard workflows. This distinction is significant: the aggregation of multi-hazard datasets and their utilization during the construction phase provide a framework that supports disaster resilience.

Fathnejat et al. (2026) linked BIM-derived material data with GIS routing to assess embodied carbon during the supply-related transportation phase. The study demonstrates the importance of actual supplier geography and network-derived distances for decision-making. Patacas et al. (2020) demonstrate requirements definition, information validation and a common data environment using open standards in facilities management. Narindri et al. (2022) develop a web-based BIM–GIS building-management platform within an ArcGIS environment. Comparing them highlights the need to separate accessibility to users from portability across systems. A convenient web interface may improve access to information while leaving significant dependence on particular software services.

Ramonell Cazador et al. (2026) use knowledge graphs and standards-based information delivery for bridge-network condition monitoring. Their demonstrative system connects IFC, geospatial and inspection data, but the authors identify industrial-scale performance and usability evaluation as future needs; some condition-index calculations are heuristic approximations. Alongside Weise et al. (2023) and the broader digital-construction discussion of Sepasgozar et al. (2023), this supports an emerging direction for linked information rather than evidence of a fully validated construction-disaster digital twin.

2.3. Evidence synthesis

The literature supports three distinct propositions. Construction studies demonstrate the significance of time, constraints, and site configurations regarding selected safety and planning analyses. Interoperability studies reveal that while enabling the reuse of data exchange and transformation, these processes can also lead to identifiable losses. Standards specify representations, interfaces or management principles that can make such exchanges more explicit. Table 1 summarizes evidential roles of selected studies.

Table 1. *Evidential roles of selected studies*

Evidence and setting	Supported function	Boundary of inference
Marzouk & Al Daour (2018); Hosseini & Maghrebi (2021) Construction evacuation	Phase-sensitive scenario comparison	Simulation and case-specific assumptions; no isolated open- standard effect
Deng et al. (2022); Yang et al. (2022) Construction-site response	Obstacle-aware routing; monitoring and alarms	Updating and analytical accuracy require further context-specific validation
Mirahadi et al. (2019); Teo & Cho (2016) Building and network applications	IFC-linked analysis; indoor–outdoor routing	Transfer to unfinished sites requires temporary-state modelling
Zavari et al. (2022); Sebastian et al. (2013) - Construction planning	Dynamic layout; spatial impact comparison	Planning objectives are not demonstrated disaster outcomes
Patacas et al. (2020); Ramonell Cazador et al. (2026) - Asset management	Requirements validation and linked records	Recovery use is a proposed transfer, not a measured construction benefit

3. Open BIM and GIS Standards for Interoperability

openBIM describes a collaborative approach supported by open standards, rather than a single file type (Jiang et al., 2019). Open GIS is used here to denote the use of openly specified geospatial representations and interfaces. Open standards, open-source software and openly accessible data are distinct: software can implement an open standard without being open source, and a standards-compliant dataset can remain access-controlled. Standards also operate at different levels. IFC and CityGML define information structures; CityJSON provides a particular encoding approach; OGC service specifications address access interfaces; and ISO 19650 addresses information-management processes. Treating them as interchangeable integration solutions obscures what each can and cannot do. Table 2 summarizes the roles considered in this chapter.

Table 2. *Standards and specifications by role in the proposed workflow*

Standard or mechanism	Information role	Decision relevance and limitation
IFC / ISO 16739-1:2024	Building and infrastructure objects and relationships	Project information exchange; actual delivered content and implementation must be checked
IDS 1.0; BCF 3.0	Supported information checks; model-linked issues	Content acceptance and coordination; neither validates overall safety
CityGML 3.0; CityJSON 2.0	Semantic geographical context; JSON encoding of a subset	Contextual representation; version and information losses matter
LandInfra; InfraGML	Land and infrastructure concepts and GML encoding	Infrastructure coordination; hazard and operating conditions need additional data
IndoorGML 2.0 Part 1	Conceptual navigation-network model	Movement representation; derivation and current traversability need validation
OGC API – Features; GeoPackage; SensorThings API	Feature access; portable spatial data; observations	Delivery of context and updates; accuracy and freshness are not guaranteed
ISO 19650-2, -4 and -6	Delivery, exchange, and health and safety information management	Responsibilities and controlled information; no geometry or hazard solver
ISO/TS 19166:2025	Requirements-led conceptual BIM-to-GIS mapping	Mapping perspective, elements and detail; excludes coordinate and physical-schema mapping
ISO/TR 23262:2021; ISO/DIS 23143-1 and -3	Interoperability report; developing exchange principles	Cross-domain guidance; DIS documents remain drafts at the reference date

3.1. IFC and supporting openBIM mechanisms

ISO 16739-1:2024 defines IFC for the exchange of building and infrastructure information and extends infrastructure coverage to facilities including roads and bridges (ISO, 2024). Its significance is the ability to communicate structured objects and relationships across applications. For a construction decision, however, relevant temporary objects, classifications and status information must actually be present in the delivered model. The existence of a schema does not populate missing content or establish that a value is correct.

Information Delivery Specification (IDS) 1.0 enables computer-interpretable specification and checking of supported information requirements (buildingSMART International, 2024). In the workflow proposed here, IDS can be used for suitable classification or property requirements in IFC deliveries. BIM Collaboration Format (BCF) supports issue exchange with references to model elements and viewpoints (buildingSMART International, 2021). It can carry the coordination record around a problematic access condition, but communicating an issue does not correct the model or remove the physical obstruction. The relevant management mechanism is the linkage between a reported condition, a responsible party, an agreed action and subsequent verification. Terminology services such as the buildingSMART Data Dictionary can assist consistent definitions, but project-specific concepts still need explicit governance (Jiang et al., 2019; Weise et al., 2023).

3.2. CityGML and CityJSON

CityGML 3.0 provides a conceptual model for semantic three-dimensional city information (OGC, 2021). It supplies a geographical context in which project objects can be related to surrounding features. Tan et al. (2023) discuss the implications of its space concepts, revised levels of detail, Versioning and Dynamizer modules for BIM–GIS integration. Earlier conversion studies must be read in their version context. Donkers et al. (2016) address geometrically valid and semantically meaningful CityGML LOD3 building conversion, while Deng et al. (2016) use schema mediation and an extension to preserve additional information. Their results establish the complexity of selecting and transforming information. They should not be interpreted as a universal, lossless mapping from every IFC representation into every CityGML edition. Tan et al. (2023) likewise identify continuing semantic and solid-to-surface conversion difficulties.

CityJSON 2.0 is an OGC Community Standard that uses JSON to encode a subset of the CityGML 3.0 data model (OGC, 2026). Its role is distinct from both a complete CityGML implementation and a BIM schema. For project decision support, a CityJSON delivery can offer a manageable contextual representation, provided the selected features and attributes are sufficient. A conversion should document unsupported content and the use of extensions; changing serialization cannot by itself resolve differences in object meaning. These distinctions have practical consequences. A generalized building envelope may be adequate for contextual visualization but insufficient for deriving indoor movement. A network may be adequate for a connectivity question but insufficient for a flood analysis requiring reliable elevations and terrain relationships. Thus, the choice between retained IFC content, a derived

CityGML or CityJSON representation, and an analytical network should follow the decision requirements.

3.3. LandInfra and InfraGML

The OGC Land and Infrastructure Conceptual Model, LandInfra, covers subjects including land features, facilities, projects, alignments, roads, railways, surveys and land division. InfraGML implements relevant concepts through GML encodings (OGC, 2016, 2017). For infrastructure coordination, these standards provide a vocabulary for exchanging spatial and engineering context. However, their potential contribution to construction-stage resilience is indirect: an agreed representation of road or alignment information can be used alongside site models when examining access and construction interfaces. Hazard exposure, current closure states and operational restrictions still require appropriate data and application rules.

3.4. Indoor navigation and geospatial access services

IndoorGML 2.0 Part 1 defines a conceptual schema for exchanging indoor navigation-network models (OGC, 2025). This is relevant where the question concerns movement and connectivity rather than the complete physical description of a building. It does not independently derive a trustworthy route from an IFC file or determine whether a temporary connection is usable. Model derivation, connection rules, obstacle treatment and routing remain implementation tasks. Teo and Cho (2016) and Deng et al. (2022) illustrate why those tasks need dedicated methods and validation. For construction management, the proposed implication is to define both normal exchange and degraded-operation procedures. A field package should carry its issue time and scope; a service should identify the dataset and observation time relevant to a decision.

3.5. Information management and cross-domain mapping

ISO 19650-2 addresses information management during asset delivery, and ISO 19650-4 addresses information exchange (ISO, 2018, 2022). ISO 19650-6:2025 specifically addresses collaborative classification, sharing and delivery of structured health and safety information (ISO, 2025a). These standards provide a management context for assigning information responsibilities and organizing exchanges. They do not perform geometric transformation or prescribe a hazard simulation. Their relevance is strongest when technical delivery is connected with review, acceptance and authorized use.

ISO/TS 19166:2025 defines a conceptual framework for transferring BIM information to GIS based on user requirements, including mappings for perspectives, elements, and levels of detail (LOD). The scope of the standard explicitly excludes physical schema integration, bidirectional mapping, and coordinate system mapping (ISO, 2025b). Therefore, while the standard supports a requirement-oriented mapping approach, it should not be considered an IFC–CityGML converter or a georeferencing solution.

4. Decision Support through Open BIM and GIS Standards

Open BIM and GIS standards support construction decision-making by enabling the sharing of project-related and geographic information across applications. The practical contribution of these standards depends on preserving the objects, relationships, and status information required for a specific task. The following section discusses the relationship between information exchange capabilities and processes such as construction site planning, emergency access, response coordination, and recovery.

4.1. Spatial assessment and phase-dependent site planning

Combining project objects with terrain, roads and hazard information provides a spatial basis for evaluating site configurations. Reliable georeferencing is essential: errors in position, orientation or elevation can alter the apparent relationship between construction activities and surrounding conditions (Noardo et al., 2020). Standards support the representation and exchange of these inputs; exposure and risk assessments require suitable hazard data and analytical methods (Cao et al., 2023).

Construction studies demonstrate several relevant applications. Zavari et al. (2022) optimize dynamic site layouts, Marzouk and Al Daour (2018) evaluate construction alternatives using duration, cost and evacuation time, and Sebastian et al. (2013) examine bridge construction in relation to surrounding disturbance. Together, these studies show how changing site configurations can be assessed against spatial and operational constraints.

For disaster preparedness, these approaches could be extended to compare temporary-facility locations under a specified flood scenario. The analysis would require the current construction phase, facility functions, terrain elevations, access routes and dated hazard information. This is a proposed application rather than a demonstrated result of the cited layout studies. Its usefulness would depend on whether the exchanged information preserves the constraints needed to assess location and access; a spatial overlap alone would not constitute a complete risk assessment.

4.2. Connecting evacuation routes and external access

Emergency routing requires continuity between movement inside the site and access beyond its boundary. Teo and Cho (2016) connect IFC-derived indoor networks to exterior streets, while Deng et al. (2022) incorporate obstacle accessibility into construction-site evacuation routing. Combining these approaches offers a basis for representing routes across the site boundary, although their integration on an evolving construction site requires further validation.

The required attributes depend on the route user. Pedestrian evacuation and emergency vehicle access involve different dimensions, restrictions and movement assumptions. Network connections must therefore represent current availability and permitted use as well as geometric continuity. Fire and evacuation studies also demonstrate why distance alone is insufficient for evaluating routes (Hosseini & Maghrebi, 2021; Mirahadi et al., 2019).

Figure 3 illustrates this dependency. A blockage at Gate A changes the usable network and requires reassessment of access through Gate B. The example shows why route analysis must use an updated site state; it does not represent a simulated evacuation outcome.

4.3. From observations to coordinated action

Monitoring and visualization become useful for management when they inform a response. Yang et al. (2022) and Hong et al. (2024) connect monitored information with visualization and response workflows. Open standards can support this connection through complementary functions: BCF communicates model-related issues, IDS specifies and checks supported IFC information requirements, and ISO 19650 provides procedures for managing and accepting information exchanges (buildingSMART International, 2021, 2024; ISO, 2018, 2022).

A proposed construction workflow would link a field observation to the affected object or location, communicate the issue, check the required information and assign an action. Application-specific checks would then assess the relevant spatial or operational condition. For example, reporting an obstructed corridor could lead to its clearance or the approval of an alternative route. The decision record should identify the information used, the responsible person and the criteria for confirming completion. Closing a digital issue should follow verification of the relevant site condition. This connects information exchange to an accountable management action.

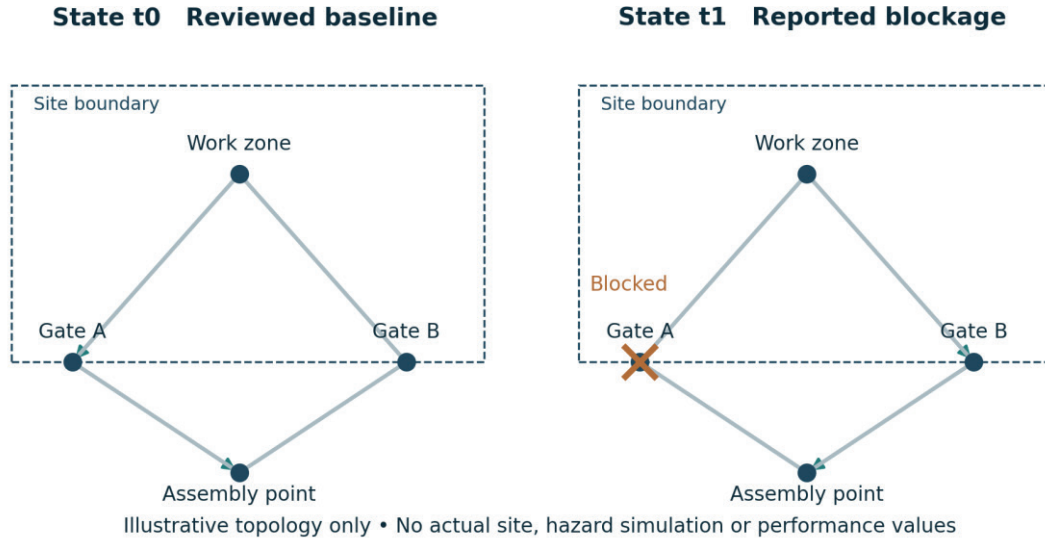


Figure 2. Route reassessment after an access blockage

Note. Schematic illustration informed by Teo and Cho (2016) and Deng et al. (2022).

4.4. Information continuity for inspection and recovery

Patacas et al. (2020) integrate structured and document-based information within a common data environment based on open standards. Meanwhile, Ramonell Cazador et al. (2026) link bridge components, geographic information, and inspection records via a knowledge graph. These studies offer mechanisms for maintaining traceable records of assets throughout their changing conditions.

In a construction project affected by an interruption, these mechanisms can link assessment findings to identifiable components and restricted-access zones while preserving pre-disaster information. This enables managers to distinguish between planned conditions, observed damage, and approved actions. Information continuity can be achieved by establishing selective connections between source systems, rather than converting all data into a single model. Therefore, persistent identifiers, up-to-date relationships, and clearly defined responsibilities for updates are the key elements of the proposed approach.

4.5. Exchange validation and decision acceptance

In these applications, the workflow consists of defining the decision, identifying the necessary information, sharing or linking records, validating the representation to be used in the analysis, and evaluating and implementing the action. Table 3 combines the information exchange issues identified by Noardo et al. (2020) with the acceptance checks proposed based on the requirement definition and validation approach presented by Patacas et al. (2020).

These checks demonstrate the fitness for purpose of the information regarding a specific decision. The contribution to disaster resilience, meanwhile, should be evaluated based on the implemented action and the outcomes of that action—whether observed or verified through appropriate methods.

Table 3. *Proposed acceptance checks and their management consequences*

Dimension	Example check	Reason for the decision
Syntactic	Selected schema, version and encoding can be read	Prevents failed or partial ingestion
Semantic	Required status, units and object meanings survive exchange	Prevents interpretation of planned availability as current permission
Geometric	Relevant openings and spaces remain valid	Prevents unsuitable analytical geometry
Spatial	CRS, orientation, units and elevation reference are checked	Prevents incorrect relations with terrain, roads or hazards
Temporal	Model, observation and restriction times are compatible	Prevents decisions based on inconsistent site states
Organizational workflow	and Source, reviewer, decision owner and update trigger are recorded	Makes acceptance, action and reassessment accountable

5. Limitations and Research Directions

A fundamental technical constraint is the loss or misinterpretation of information critical to the decision-making process. Objects with similar geometries may possess different functions or access rights; furthermore, the model conversion process can alter openings, levels, or relationships. Research on IFC–CityGML demonstrates why semantic and geometric checks must be conducted separately (Deng et al., 2016; Donkers et al., 2016; Tan et al., 2023). The significance of these checks depends on the specific decision at hand, as overlooking a decorative element versus an exit point can yield vastly different analytical outcomes.

Spatial and temporal consistency are equally important. Coordinate reference systems, units, and elevation references must be aligned, while observations and model states must reflect the relevant time period. A model that is internally consistent but outdated may suggest an unusable route. The information management process must also define who is authorized to publish, update, and accept the data. The roadmap proposed by Mallela and Bhargava

(2025) emphasizes the need to address implementation and organizational responsibilities alongside technical specifications.

The evidence base is limited. While construction-related efforts focus on specific site layouts and issues regarding fire and evacuation, various open-standard practices address completed assets. The results of these efforts cannot be automatically generalized to scenarios involving earthquake damage, flooding, or post-construction recovery. Furthermore, the selective narrative approach of this section does not support combined impact estimates or claims of comprehensive coverage. The proposed workflow and recovery practices require testing on construction sites.

Future research should measure whether information exchange alters decision quality. At the exchange level, tests should examine the preservation of necessary attributes, identifiers, geometry, and spatial references. At the analytical level, connectivity, sensitivity to missing information, and the influence of past observations should be evaluated. At the management level, relevant outcomes include decision accuracy and timing, completed actions, and the effort required for data maintenance. Comparisons should keep the analytical task constant while varying exchange profiles or input quality. Intermittent connectivity, blocked links, and mixed observation intervals would provide useful test conditions. This will help identify both the benefits and the failure conditions of standards-based decision support systems.

6. Conclusions

Open BIM and GIS standards can support construction-stage disaster resilience decisions by making project and geographical information exchangeable, interpretable and traceable across systems. Their contribution operates through specific mechanisms: relating the site to its hazard and infrastructure context, connecting internal and external movement, representing phase-dependent configurations, coordinating information around action, and maintaining records for inspection and recovery. The reviewed applications establish components of these mechanisms, with particularly relevant evidence in evacuation, obstacle-aware routing, dynamic layout and information validation.

The contribution remains conditional. IFC does not ensure that temporary conditions are modelled; CityGML or CityJSON does not ensure that analytical detail is sufficient; navigation and service standards do not establish current traversability or data accuracy; and information-management standards do not replace hazard assessment or professional responsibility. Semantic, geometric, spatial, syntactic, organizational and workflow

interoperability must be considered together. A successful exchange is therefore an intermediate result, not evidence in itself of improved disaster performance.

The answer to the research question is consequently decision-specific. Open standards support resilience-oriented construction management when they preserve the information required for a defined decision, enable validated analysis, and connect the result to an accountable action. The proposed workflow and evaluation dimensions make that relationship explicit without presenting a new validated resilience index or claiming effects beyond the available evidence. Future work should test this relationship on evolving construction sites, documenting both successful decisions and the information failures that limit them.

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CHAPTER 3

EVALUATION OF RECLAIMED ASPHALT PAVEMENT AS A PARTIAL REPLACEMENT FOR FINE AGGREGATE IN HOT MIX ASPHALT: MARSHALL PROPERTIES AND VOLUMETRIC PERFORMANCE

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1. GİRİŞ

The development of road transport networks and the ongoing maintenance and repair of existing roads consume significant quantities of natural aggregates and bituminous binders. In the production of hot-mix asphalt, for instance, mineral aggregates form the skeletal structure of the mixture, while the bitumen acts as a binder, connecting the aggregate particles and facilitating load transfer. The growing demand for these materials has made the conservation of natural resources and the evaluation of alternative materials as a key areas of research in road construction. In this context, reusing materials generated during the renewal or rehabilitation of existing road pavements offers a significant opportunity to reduce natural resource consumption and repurpose waste (Naser et al., 2022; Qasrawi and Asi, 2016; Ramadan et al., 2025; Tarsi et al., 2020; Tuncan et al., 2003).

Reclaimed Asphalt Pavement (RAP) material, obtained through the milling or removal of existing asphalt pavements, contains both aggregate particles and aged bituminous binder (Han et al., 2019; Kumar et al., 2022). Due to this characteristic, RAP is considered a valuable secondary material that can replace a portion of the natural aggregate—and, under certain conditions, a portion of the new bitumen—in new asphalt mixtures. Reusing RAP in new asphalt mixtures enables the use of existing aggregate and aged bitumen resources, while reducing the need for primary raw materials (Tarsi et al., 2020). However, the incorporation of RAP into new mixtures should not be viewed merely as a means of increasing the amount of recycled material; the mechanical properties, volumetric parameters, and long-term performance of the mixture must also be considered.

The behavior of asphalt mixtures containing RAP depends on many factors, including the proportion and properties of the reclaimed material, particle size distribution, the characteristics of the aged bitumen, the amount of virgin binder, and the mix design (Ma et al., 2015). The hardening of bitumen during aging can increase the stiffness of mixtures containing RAP; however, the magnitude of this effect may vary with RAP content and production conditions (Sias Daniel et al., 2013). In some cases, while increased rigidity may contribute to resistance against permanent deformation, it can lead to adverse outcomes regarding fatigue behavior and low-temperature performance (Elkashaf et al., 2018). Therefore, it is not accurate to evaluate the impact of using RAP on the performance of asphalt mixtures based on a single trend applicable to all mixtures. The material properties and the replacement ratio must be considered together.

In hot-mix asphalt, mineral aggregates generally consist of coarse aggregate, fine aggregate, and mineral filler fractions. Each of these fractions plays a distinct role in the formation of the mixture's granular skeleton and void structure (Chen et al., 2019). Fine aggregate contributes to filling the voids between coarse aggregates and ensuring the continuity of the aggregate skeleton, while mineral filler, together with bitumen, plays a role in the formation of the mastic structure (Chen et al., 2019; Yan et al., 2013). For this reason, identifying which aggregate fraction is replaced by RAP is important for evaluating the properties of the resulting mixture. Beyond the overall proportion of RAP within the total mixture, replacing natural aggregate with RAP within a specific particle-size group allows for a more controlled examination of the impact of the reclaimed material (Han et al., 2019; Stroup-Gardiner and Wagner, 1999).

The use of RAP in the fine aggregate fraction not only reduces the consumption of natural aggregates but also allows for the investigation of how the particle size distribution of the reclaimed material—as well as the aged bitumen it contains—affects the behavior of the mixture (Han et al., 2019). However, the effect of RAP in the fine fraction on mixture performance cannot be explained solely by the presence of aged bitumen. The distribution of aggregate particles, the interaction between the existing bitumen and the new binder, and the volumetric properties of the mixture must all be considered together (Hettiarachchi et al., 2020). Furthermore, since the optimum bitumen requirement may change as the RAP replacement ratio varies, it is essential to examine each mix group under its own specific design conditions (Sabouri, 2020).

The Marshall method, used in the design of asphalt mixtures, enables the determination of mixture stability and flow characteristics and the evaluation of volumetric parameters related to bitumen content. Marshall stability (MS) refers to the maximum load a specimen can withstand under standard test conditions, while the flow value indicates the deformation that occurs at or near the maximum load. Evaluating these two parameters together is essential for interpreting the mixture's load-bearing capacity and deformation behavior. However, it is not appropriate to draw a definitive conclusion about the overall performance of a mixture based solely on stability or flow values; volumetric properties—such as air voids, voids in the mineral aggregate, and voids filled with bitumen—must also be taken into account (Coree, 1999).

In this study, the effect of using RAP material as a replacement for natural fine aggregate in hot-mix asphalt on Marshall parameters was experimentally investigated. For this purpose, three different mixture groups were prepared: a reference group (Ref), a group with 25% RAP replacement (RAP25), and a group with 50% RAP replacement (RAP50). For each group,

specimens were produced with bitumen contents of 3.5%, 4.0%, 4.5%, 5.0%, 5.5%, 6.0%, and 6.5%. Subsequently, the optimum bitumen content (OBC) for each mixture group was determined based on the relationship between air voids and bitumen content, corresponding to an air void level of 5%.

The primary objective of this study is to investigate the effects of incorporating 25% and 50% RAP into the fine-aggregate fraction on the OBC and Marshall parameters, compared with a reference mixture. In doing so, the study aims to evaluate the impact of using fine RAP on mix design and present experimental findings on the feasibility of using this material as an alternative to natural fine aggregates in hot-mix asphalt.

2. MATERIALS AND METHODS

2.1. Materials

2.1.1. Aggregate

In this study, crushed limestone aggregate and RAP material were used as aggregates in the preparation of hot-mix asphalt. The limestone-based aggregates served as the natural aggregate component in both the reference mixture and the mixtures containing RAP. The RAP material was evaluated as a replacement for 25% and 50% of the natural fine aggregate by weight, while no RAP was included in the reference mixture. The RAP aggregate was obtained by processing asphalt pavement material—reclaimed from end-of-life roads by the Tokat Municipality—using jaw crushers in a laboratory setting.

The particle size distribution of the aggregates used in the mixtures was determined from sieve analysis results and adjusted to achieve the target gradation. The aggregate gradation was established by considering the gradation limits specified for the binder course. The physical properties of the aggregates used are presented in Table 1, while the aggregate gradation obtained from the sieve analysis is shown in Figure 1.

Table 1. Physical properties of aggregates

Experiment Name	Natural Aggregate			RAP	Specification Limit	
	Coarse	Fine	Fillers	Fine	Coarse	Fine
Specific Gravity (g/cm ³)	2.713	2.731	2.722	2.578	-	-
Water Absorption (%)	0.34	0.81	-	0.67	≤ 2.5	≤ 2.5
Los Angeles Abrasion (%)	20.8				≤ 30	
Flakiness Index (%)	17.6				≤ 30	

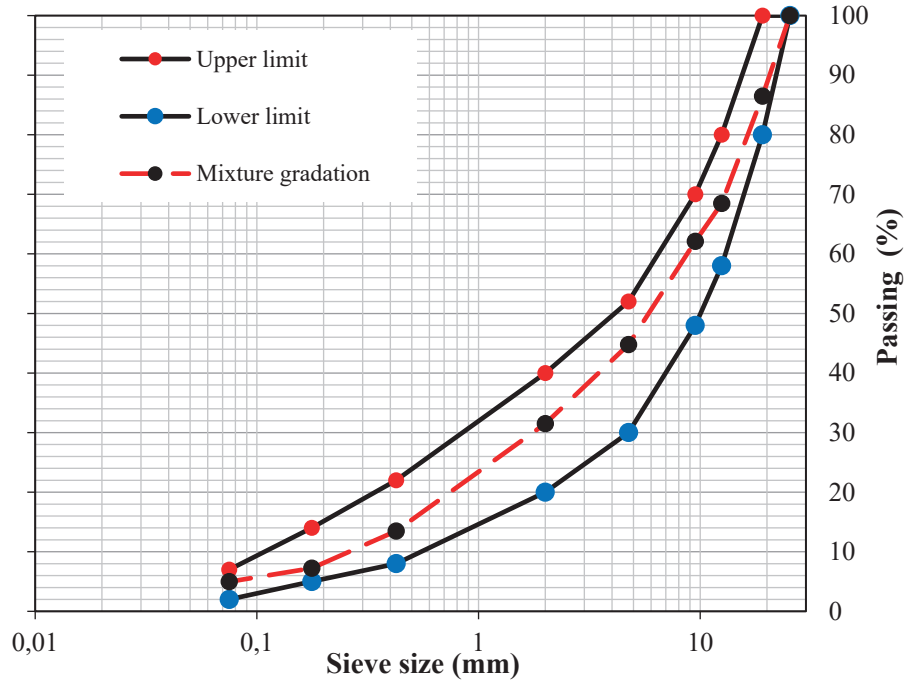


Figure 1. Aggregate gradation curve

2.1.2. Bitumen

In preparing the Marshall mixtures, bitumen with a penetration grade of 50/70, produced at the Kırıkkale Refinery of Turkish Petroleum Refineries Corporation (TÜPRAŞ), was used. The penetration value of the bitumen was determined in accordance with the TS EN 1426 standard, the softening point according to TS EN 1427, and the specific gravity according to TS EN 15326. The physical properties of the bitumen are presented in Table 2.

Table 2. Physical properties of bitumen

Experiment Name	Test Result	Required Limits
Penetration (25°C, 1/10 mm)	57	50-70
Softening point (°C)	51	46-54
Specific gravity (g/cm ³)	1.029	-

2.2. Method

In accordance with the aggregate gradation, 1150 g of aggregate mixture was prepared for each Marshall specimen. In this study, three groups of mixtures were created to examine the effect of replacing natural fine aggregate with RAP on the performance of hot mix asphalt. A reference mixture (Ref) was prepared using natural aggregates, while in the other mixtures, 25% (RAP25) and 50% (RAP50) of the natural fine aggregate—by weight—were replaced with RAP material.

The objective was to compare the effects of the RAP replacement ratio on the Marshall parameters and volumetric properties of the mixtures.

For each of the three mixture groups, Marshall specimens were prepared with bitumen contents of 3.5%, 4.0%, 4.5%, 5.0%, 5.5%, 6.0%, and 6.5% by weight. One day prior to specimen preparation, the aggregate blends and bitumen were heated in an oven at 155 °C to ensure they were ready for mixing. The heated aggregate and bitumen were combined according to the specified bitumen percentages and mixed until a homogeneous blend was achieved. To minimize heat loss during mixing, the mixing bowl was heated from below.

The homogenized bituminous mixture was placed into a pre-oiled Marshall mold. Compaction of the specimens was carried out by applying 75 blows to each face of the specimen using a Marshall hammer. Following the compaction process, the specimens were left to cool at room temperature. Once cooled, the specimens were removed from their molds with the aid of a jack and prepared for the testing phase.

Marshall tests were conducted on the prepared specimens to determine their stability and flow values. Furthermore, to evaluate the volumetric properties of the mixtures, the air voids (V_h), voids in the mineral aggregate (VMA), voids filled with asphalt (VFA), and bulk specific gravity (D_p) were calculated. Based on the experimental results, graphs were generated for each mixture group to illustrate the variations in these properties as a function of bitumen content.

The OBC for each mixture group was determined individually based on the relationship between air voids and bitumen content by identifying the bitumen percentage corresponding to 5% air voids. This approach also aimed to evaluate how variations in the RAP replacement ratio affect the OBC. The determined OBCs and the corresponding Marshall parameters were used to compare the Ref, RAP25, and RAP50 mixtures.

3. RESULTS AND DISCUSSION

3.1. Determination of OBC

To determine the OBC for hot-mix asphalt, the air void values obtained at various bitumen contents were evaluated for the Ref, RAP25, and RAP50 mixtures. For each mixture group, the relationship between air voids and bitumen content was expressed using second-order polynomial equations based on experimental data. Figure 2 presents the air void plots for these mixtures.

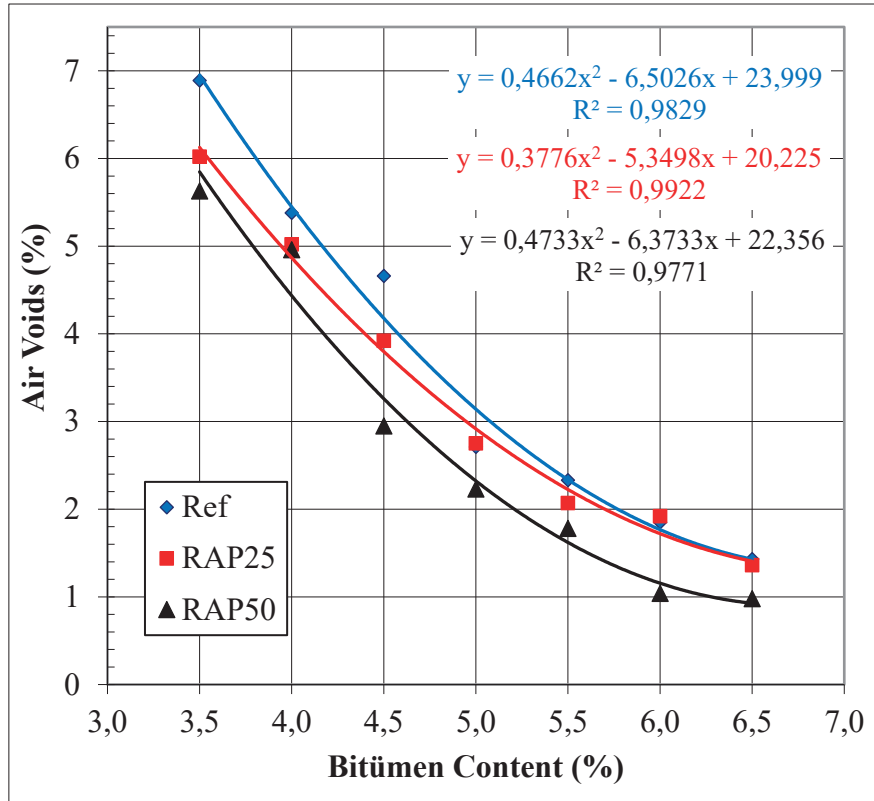


Figure 2. V_h versus bitumen content for Ref, RAP25, and RAP50 mixtures

OBCs were calculated using the equations provided on the graph, assuming an air void ratio of 5%.

For example, the relationship obtained for the mixture labeled "Ref" is expressed by the following equation:

$$V_h = 0.4662x^2 - 6.5026x + 23.999 \quad (1)$$

Here, " V_h " represents the air void ratio, and " x " denotes the bitumen content. To determine the OBC, " $V_h=5$ " was used in the equation. Among the roots obtained from solving the equation, the value falling within the range of bitumen content used in the experiments was selected. Accordingly, the OBC for the reference mixture was determined to be approximately 4.17%.

The OBCs for the RAP25 and RAP50 mixtures were also determined by following the same method, calculating the bitumen proportions that correspond to 5% air voids based on the respective air void-bitumen content curves. Accordingly, the OBC for the RAP25 mixture was approximately 3.94%, while that for the RAP50 mixture was approximately 3.79%.

When comparing OBCs, it is observed that as the RAP replacement ratio increases, the bitumen content required to meet the 5% air void criterion decreases. The OBC, which is 4.17% for the

reference mixture, was determined to be 3.94% for the RAP25 mixture and 3.79% for the RAP50 mixture. A similar trend was reported in a study by Mulatu et al., (2023), in which the OBC decreased from 5.10% to 4.53% as the RAP replacement ratio increased from 0% to 65%. This similarity indicates that the effect of using RAP on OBC can vary depending on the mixture composition and design conditions.

However, it should also be noted that the OBC in RAP-containing mixtures does not always decrease. Han et al., (2019) reported that in mixtures with high RAP content or a high proportion of fine RAP, the OBC generally increases slightly; conversely, the consumption of virgin asphalt decreases due to the presence of the reclaimed binder. This discrepancy demonstrates that the OBC cannot be explained solely by the RAP proportion; rather, factors such as aggregate gradation, the amount of aged bitumen within the RAP, and the effectiveness of this bitumen within the mixture must also be taken into account. In the current study, because the bitumen content of the RAP and the extent to which the aged bitumen blends with the new binder were not directly determined, the reason for the reduction in the OBC cannot be definitively established. Nevertheless, the results indicate that, for the examined mix design and based on a 5% air-void criterion, the OBC decreases as the RAP replacement ratio increases. While this reduction offers a potential advantage by reducing the use of new bitumen, the amount of binder in the RAP and its actual contribution to the mixture must be evaluated separately to determine the true extent of bitumen savings.

3.2. Evaluation of MS and Flow Values

MS and flow value are fundamental parameters used to evaluate the behavior of hot-mix asphalt under load. While MS represents the maximum resistance a mixture can exhibit under loading, the flow value indicates the deformation that occurs during loading. Evaluating these two parameters together provides a more comprehensive understanding of the load-bearing capacity and deformation behavior of the mixtures. In this study, the MS and flow values of the mixtures—designated as Ref, RAP25, and RAP50—were assessed based on the OBC determined for each mixture.

Figure 3 presents the MS values obtained at various bitumen contents. Based on calculations using the regression equations from the graph, the stability values at the OBC were found to be approximately 1389 kgf, 1484 kgf, and 1447 kgf for the Ref, RAP25, and RAP50 mixtures, respectively. Accordingly, the stability of the RAP25 mixture is approximately 6.8% higher than that of the reference mixture, while the stability of the RAP50 mixture is approximately 4.2% higher. The highest stability value, observed for the RAP25 mixture, indicates that this

replacement ratio can positively influence the load-bearing capacity of the mixture under the tested experimental conditions. Although the RAP50 mixture also exhibited higher stability than the reference mixture, the increase in stability was more limited compared to that of the RAP25 mixture.

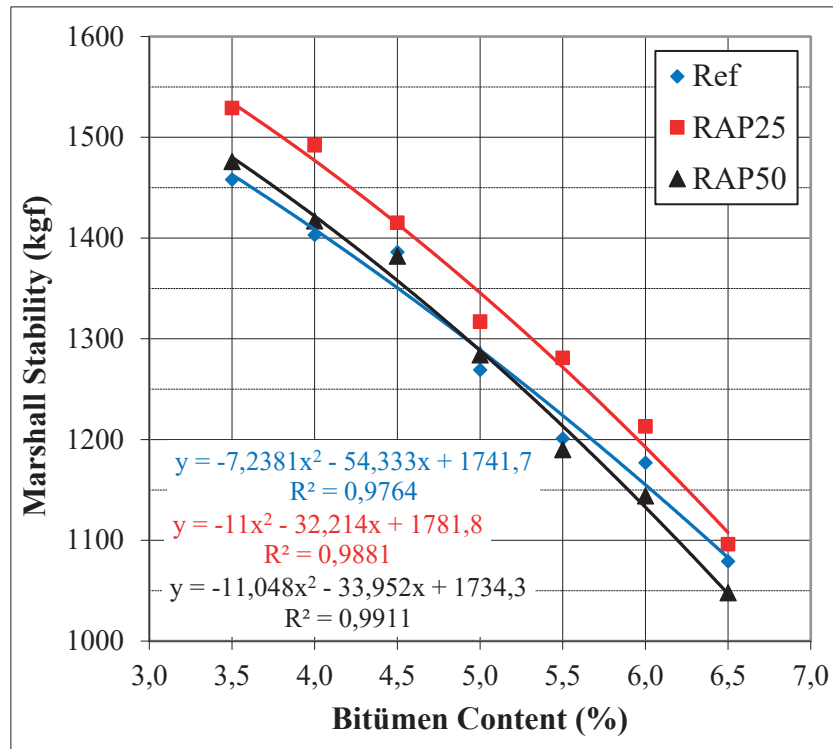


Figure 3. MS versus bitumen content for Ref, RAP25, and RAP50 mixtures

In this study, the higher MS values obtained for the RAP25 and RAP50 mixtures compared to the reference mixture indicate that the use of RAP can enhance load-bearing resistance under specific mix design conditions. Similarly, Ma et al. (2020) reported higher MS in hot-mix asphalt samples containing RAP compared to a reference mixture. Furthermore, Naser et al. (2022) noted that substituting natural aggregates with RAP can have a positive impact on the MS of asphalt mixtures. However, the fact that the highest stability in the current study was observed in the RAP25 mixture—while the increase in the RAP50 mixture was more limited, suggests that increasing the RAP replacement ratio does not necessarily lead to a continuous improvement in stability. This behavior in RAP-containing mixtures should be evaluated by considering the potential stiffening effect of aged bitumen, as well as the aggregate skeleton and volumetric properties.

Figure 4 illustrates the flow values of the mixtures at various bitumen contents. Based on the results derived from the regression equations, the flow values at the OBCs are approximately 2.53 mm, 2.23 mm, and 2.41 mm for the Ref, RAP25, and RAP50 mixtures, respectively. The

lower flow value of the RAP25 mixture compared to the reference mixture indicates that this mixture exhibits more limited deformation under maximum load. The flow value of the RAP50 mixture, meanwhile, falls between those of the Ref and RAP25 mixtures. These results demonstrate that the RAP replacement ratio influences flow behavior and that this effect can vary with the specific replacement ratio.

An examination of the flow values reveals that the RAP25 mixture yielded a lower value than the reference mixture, while the RAP50 mixture showed an intermediate value. This suggests that the RAP replacement ratio influences the mixture's deformation behavior, though the effect does not follow a linear trend. The decrease in flow, accompanied by an increase in stability for the RAP25 mixture, is consistent with higher load resistance and more limited deformation within the scope of the design under study. Nevertheless, definitive conclusions regarding permanent deformation or fatigue performance cannot be drawn solely from MS and flow results. Therefore, these mechanical findings should be interpreted alongside the volumetric properties evaluated below.

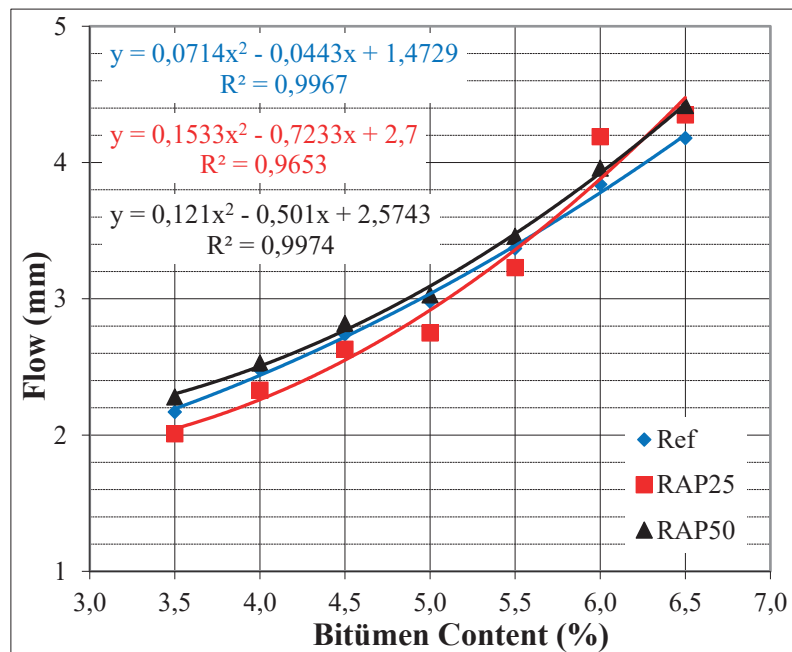


Figure 4. Flow value versus bitumen content for Ref, RAP25, and RAP50 mixtures

According to the General Directorate of Highways Technical Specifications, the binder course must have a MS of at least 750 kgf and a flow value of 2–4 mm. When the calculated results are compared with these limits, all three mixture groups meet the relevant specification criteria for both MS and flow. All stability values exceed the minimum threshold, and all flow values fall within the specified range.

3.3. Evaluation of Volumetric Properties and Compliance with Specifications

In the performance evaluation of hot-mix asphalt, volumetric properties provide significant insight into the structure of the aggregate skeleton and the distribution of bitumen within the mixture. In this context, the D_p , VMA, and VFA for the mixtures designated as Ref, RAP25, and RAP50 were examined, based on the OBC determined for each mixture. The results obtained were evaluated against the limit values specified for the binder course in the General Directorate of Highways Technical Specifications.

Figure 5 illustrates the variations in the D_p of the mixtures as a function of bitumen content. The D_p values at the OBCs—calculated using regression equations—were determined to be approximately 2.426, 2.417, and 2.410 g/cm³ for the Ref, RAP25, and RAP50 mixtures, respectively. An increase in the RAP replacement ratio decreased D_p . Similarly, in a study by Mulatu et al. (2023), the D_p of the mixtures declined from 2.404 to 2.379 g/cm³ as the RAP replacement ratio rose from 0% to 65%. This similar trend observed in both studies indicates that increasing the proportion of RAP used can reduce the mixture's D_p . One potential reason for this reduction is that the specific gravity of the RAP aggregate differs from that of the natural aggregate employed. In fact, the study also attributed the decrease in D_p to that of the RAP aggregate (Mulatu et al., 2023).

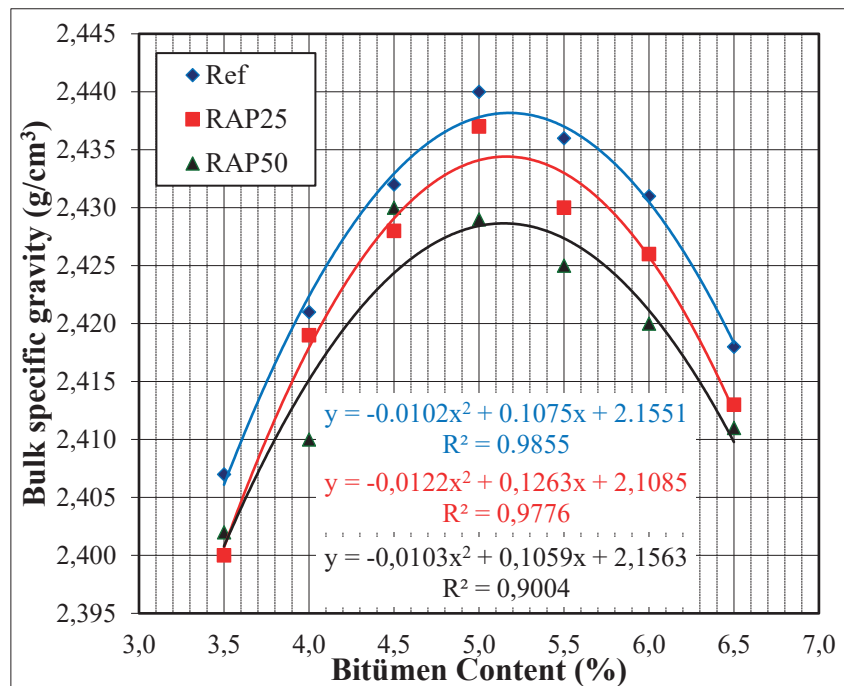


Figure 5. D_p of Ref, RAP25, and RAP50 mixtures

An examination of the VFA results presented in Figure 6 reveals that, at OBCs, the values obtained for the Ref, RAP25, and RAP50 mixtures are approximately 65.88%, 63.96%, and

63.47%, respectively. Given the 60–75% range specified for the binder course in the General Directorate of Highways Technical Specifications, it is evident that all three mixtures meet the VFA requirements. Furthermore, VFA values decreased as the RAP replacement ratio increased. This indicates that the RAP replacement ratio influences the volumetric properties of the mixtures.

Han et al., (2019) reported VFA values in the range of 69.03%–70.55% for mixtures containing RAP and fine RAP. Although the values obtained in the present study—ranging from 63.47% to 65.88%—are lower, the VFA values in both studies remain within the limits specified by the relevant standards. This discrepancy may be attributed to design variables such as the aggregate gradation, RAP content, and OBC of the mixtures.

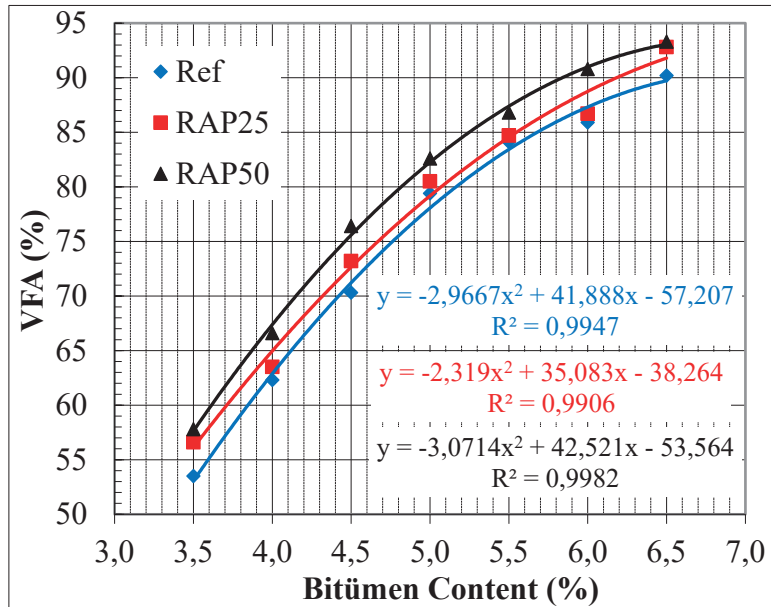


Figure 6. VFA of Ref, RAP25, and RAP50 mixtures

When the VMA values presented in Figure 7 are evaluated, the values calculated for the Ref, RAP25, and RAP50 mixtures are approximately 13.54%, 13.12%, and 12.42%, respectively. According to the criteria for binder courses in the General Directorate of Highways Technical Specifications, the VMA value must fall within the range of 13%–15%. Accordingly, while the Ref and RAP25 mixtures remain within the specification limits, the VMA value of the RAP50 mixture falls below the minimum threshold. The decrease in VMA as the RAP replacement ratio is increased to 50% indicates that the volume of voids in the aggregate skeleton of this mixture has fallen below the minimum stipulated in the specifications.

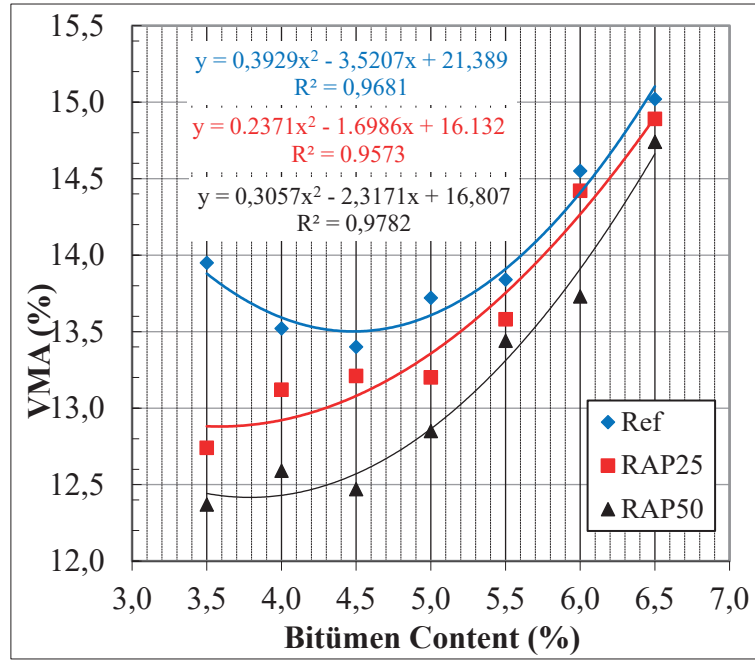


Figure 7. VMA of Ref, RAP25, and RAP50 mixtures

Similarly, another study by Al-Mosawe and Al-Mosawe, (2023) reported a decrease in VMA values as the RAP usage rate increased. This finding aligns with the current study, in which increasing the RAP replacement rate from 25% to 50% reduced the VMA value from 13.12% to 12.42%. Furthermore, the VMA value for the RAP50 mixture in this study falls below the 13% minimum threshold specified in the standards, indicating that any increase in RAP content must be carefully evaluated for volumetric compliance.

When evaluated as a whole, all three mixtures met the specification requirements for VFA. Regarding VMA, the Ref and RAP25 mixtures satisfied the specification criteria, whereas the RAP50 mixture failed to meet the minimum VMA requirement. These findings demonstrate that, when assessing the RAP replacement ratio, it is necessary to consider not only MS and flow values but also the volumetric properties of the mixtures. Consequently, the impact of the RAP replacement ratio on mixture performance should be evaluated by considering both mechanical parameters and volumetric properties.

4. Conclusion

In this study, the effects of replacing natural fine aggregate with RAP material—at proportions of 25% and 50% by weight—in hot-mix asphalt on the Marshall parameters and volumetric properties of the mixtures were investigated. Reference, RAP25, and RAP50 mixtures were prepared with varying bitumen contents. OBCs were determined at the point where the air void

content reached 5%. Based on the experimental findings and comparisons with specifications, the following conclusions were reached:

- The OBCs, determined based on the relationship between air voids and bitumen content, were calculated to be approximately 4.17%, 3.94%, and 3.79% for the Ref, RAP25, and RAP50 mixtures, respectively. It was observed that as the RAP replacement ratio increases, the bitumen content required to meet the 5% air-void condition decreases.
- Based on regression calculations at OBCs, the MS values for the Ref, RAP25, and RAP50 mixtures were determined to be approximately 1389, 1484, and 1447 kgf, respectively. The stability values for the mixtures containing RAP were found to be higher than those of the reference mixture, with the highest value calculated for the RAP25 mixture.
- At OBCs, the flow values for the Ref, RAP25, and RAP50 mixtures were determined to be approximately 2.53, 2.23, and 2.41 mm, respectively. The lowest flow value was obtained for the RAP25 mixture. When MS and flow results are evaluated together, a 25% RAP replacement exhibits favorable mechanical behavior within the scope of the mix design under investigation.
- The D_p values for the Ref, RAP25, and RAP50 mixtures were calculated to be approximately 2.426, 2.417, and 2.410 g/cm³, respectively. A decrease in D_p was observed as the RAP replacement ratio increased.
- The VFA values for all three mixtures remained within the 60–75% range specified in the standard for binder courses. Meanwhile, the VMA values were calculated to be approximately 13.54%, 13.12%, and 12.42% for the Ref, RAP25, and RAP50 mixtures, respectively. While the Ref and RAP25 mixtures met the 13–15% VMA requirement, the RAP50 mixture did not meet the minimum VMA criterion.

Overall, the findings indicate that replacing 25% of the natural fine aggregate with RAP improves MS and flow behavior within the scope of the investigated mix design, while also meeting specification criteria for volumetric properties. Furthermore, the decrease in OBC—from 4.17% to 3.94% and 3.79% as the RAP replacement ratio increases—suggests that the demand for virgin bitumen can be reduced under appropriate mix design conditions. Given the significant share of bitumen in the total cost of an asphalt mixture, this offers a potential economic advantage; simultaneously, it contributes to conserving natural aggregates and reduces the need for landfilling of waste asphalt pavements. However, the VMA value falling

below the minimum limit specified by the standards at a 50% RAP replacement level indicates that high replacement ratios must be evaluated with caution for volumetric compliance. In conclusion, the use of RAP in hot-mix asphalt holds significant potential for waste recovery, the preservation of natural resources, and the reduction of virgin bitumen requirements. Realizing this potential in terms of economic and environmental benefits depends on determining the RAP replacement ratio by carefully integrating mechanical performance, volumetric properties, and regulatory specifications.

In future studies, examining the RAP replacement ratio over a wider range and evaluating various aggregate gradations and bitumen contents could help improve mix design. Furthermore, investigating the properties of the aged bitumen within the RAP material—as well as its interaction with virgin bitumen—could help provide a more detailed explanation for the observed reduction in OBC. In addition to Marshall tests, conducting wheel-tracking, indirect tensile strength, moisture sensitivity, and fatigue tests would allow assessment of how RAP-containing mixtures perform under diverse traffic and environmental conditions. Moreover, monitoring long-term field performance and evaluating life-cycle costs and environmental impacts would provide a more comprehensive understanding of how the use of RAP contributes to economic and environmental sustainability.

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CHAPTER 4

PRELIMINARY SEISMIC RISK ASSESSMENT OF BUILDINGS IN NAZIMIYE DISTRICT (TUNCELI, TÜRKİYE) USING A STREET SCREENING METHOD

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1.Introduction

Türkiye is located in one of the world's seismically active regions and has experienced numerous destructive earthquakes throughout its history. These events have resulted in significant loss of life, extensive structural damage, and substantial socioeconomic consequences. Among the recent earthquakes affecting the region, the Elazığ–Sivrice earthquake (Mw 6.8) is particularly noteworthy because of the damage it caused, as well as its social and psychological impacts (Caglar et al., 2023).

The vulnerability of the existing building stock is a major factor influencing the potential consequences of future earthquakes. Building age, structural system, construction quality, and maintenance conditions may affect the ability of buildings to withstand seismic forces. Consequently, assessing the seismic vulnerability of existing buildings is an important component of earthquake risk reduction. However, evaluating large numbers of buildings through detailed engineering investigations requires considerable time and financial resources. This limitation highlights the need for rapid and cost-effective screening approaches that can identify potentially vulnerable buildings and support the prioritization of further investigations (Dinç, 2015).

The seismic setting of Nazımiye District also highlights the importance of evaluating the vulnerability of its existing building stock. The district is located in the vicinity of the Yedisu segment of the North Anatolian Fault Zone, an area of particular interest in regional seismicity studies. Previous research has examined the paleoseismic history and earthquake potential of the Yedisu seismic gap, emphasizing its significance within the regional tectonic framework (Zabci et al., 2017). The proximity of this active tectonic system provides an important regional context for assessing the potential seismic vulnerability of buildings in Nazımiye. However, the actual performance of individual buildings also depends on their structural characteristics, construction quality, and local ground conditions; therefore, proximity to a fault alone cannot determine the risk classification of a building.

Rapid visual screening methods provide a practical means of assessing building characteristics and identifying structures that may require more detailed evaluation. These methods rely on observable features, including the structural system, building age, number of stories, visible physical condition, and potential structural irregularities. Standardized assessment forms and established evaluation criteria can facilitate the systematic recording and

classification of field observations (Demirbaş et al., 2021). Nevertheless, the results of rapid screening should be interpreted as preliminary indicators of potential vulnerability rather than definitive assessments of structural safety.

Despite the availability of rapid assessment techniques, systematically evaluating the existing building stock in small settlements remains an important challenge. Variations in building characteristics, construction practices, and local topography may influence the distribution of potential seismic vulnerability. Street-level screening offers a practical approach for documenting these characteristics and identifying buildings that may require further investigation.

In this context, the present study evaluates the existing building stock in Nazımiye District, Tunceli, Türkiye, using a street-level screening approach. Approximately 160 buildings were examined through field observations, with particular attention to their observable structural characteristics and surrounding conditions. The main objectives of the study are to classify the surveyed buildings according to their preliminary seismic risk, examine the distribution of the resulting risk categories, and identify building characteristics relevant to potential seismic vulnerability. The findings are intended to provide preliminary information that may support the prioritization of detailed structural investigations and inform local earthquake risk-reduction planning.

2. Methodology

Nazımiye District, located east of Tunceli city centre, is characterized by mountainous terrain and a rugged topography. The local building stock predominantly consists of low-rise reinforced concrete (RC) buildings and masonry dwellings. These characteristics highlight the importance of examining the potential seismic vulnerability of existing buildings in the district.

The building inventory was compiled using information obtained from the local municipality, available maps, and satellite imagery. Field observations were conducted on approximately 160 buildings within the defined study area. The assessment focused on observable building characteristics, including structural system, estimated building age, number of stories, visible physical condition, and surrounding environmental conditions. These characteristics were documented through street-level observations and used to support the preliminary classification of the surveyed buildings according to their seismic risk.

The recorded building characteristics were also considered in relation to the spatial distribution of the inspected buildings within the study area. This approach provided a basis for examining the distribution of preliminary risk categories and interpreting the assessment results in the context of the local built environment.

For a clearer assessment of the scope and representativeness of the survey, additional information should be provided regarding the survey period, the geographical boundaries of the study area, the total number of buildings within those boundaries, and the criteria used to select the inspected buildings. These details are important for determining the extent to which the surveyed sample represents the existing building stock of Nazımiye District.

2.1. Applied Method

Nazımiye District is located in the eastern part of Tunceli Province, in the Eastern Anatolia Region of Türkiye. Covering an area of approximately 553 km², the district lies at an elevation of about 1,550 m above sea level and is characterized by a mountainous and rugged topography. According to the Turkish Statistical Institute's (TurkStat) 2025 Address-Based Population Registration System data, Nazımiye has a population of approximately 3,095. The district and its surrounding areas are located within the seismically active tectonic setting of Eastern Anatolia. The presence of major fault systems in the surrounding region, including the Nazımiye and Pülümür faults and the Yedisu segment of the North Anatolian Fault Zone, makes seismic hazard an important consideration for the district. However, seismic hazard and building-level seismic risk are distinct concepts. While seismic hazard relates to the likelihood and potential severity of earthquake ground motion, building-level risk also depends on structural characteristics, construction quality, and the physical condition of individual buildings. Therefore, field-based assessments of the existing building stock are important for identifying potentially vulnerable structures and supporting local seismic risk reduction efforts.

The street-level screening method was implemented through a series of systematic steps, including preparation, field observation, risk evaluation, and data analysis.

Preparation Phase: Building inventory data and street information for Nazımiye District were obtained from the local municipality. The boundaries of the study area were identified with the support of available maps and satellite imagery.

Field Observation Phase: Field investigations focused on building characteristics observable from the exterior. The recorded parameters included the number of stories, structural system type (e.g., reinforced concrete, masonry, or hybrid construction), estimated building age, visible signs of structural damage, building configuration (detached or adjacent), and the presence of potential short-column effects.

Risk Evaluation Phase: Field observations were recorded using the street-level screening form developed by Kılıç (2014). The collected information was used to classify the surveyed buildings according to their preliminary seismic risk. The evaluation framework proposed by JICA (2002) was also referenced in the assessment process.

Data Analysis Phase: The collected field data were organized and processed using Microsoft Excel. The recorded building characteristics and assigned risk categories were then examined to evaluate the distribution of preliminary seismic risk among the surveyed buildings. The results were also presented graphically to facilitate interpretation.

The principal advantage of the street-level screening method is its ability to provide a preliminary assessment of a large number of buildings within a relatively short period and with limited resources. By documenting observable structural characteristics, the method helps identify buildings that may require more detailed investigation. However, because the assessment is based primarily on visual observations, it cannot replace comprehensive structural analyses or establish the definitive seismic safety of individual buildings (Yakut, 2004).

The screening form developed by Kılıç (2014) was selected because its practical implementation was considered suitable for documenting the characteristics of the building stock in Nazımiye District. The form enabled the systematic recording of relevant building information and field observations, providing a consistent basis for the preliminary risk classification.

It should be emphasized that the results obtained through this approach represent a preliminary screening assessment rather than a definitive determination of structural performance. Buildings identified as potentially vulnerable should therefore be prioritized for detailed engineering investigations to establish their actual structural condition and seismic performance.

DATA COLLECTION FORM FOR REINFORCED CONCRETE BUILDINGS						
BUILDING IDENTIFICATION INFORMATION					Date:	
					No.:	
BUILDING ID NO.		BUILDING PHOTOGRAPH (A CLEAR PHOTOGRAPH OF THE FRONT FACADE THAT REPRESENTS THE BUILDING MUST BE PROVIDED)				
PROVINCE						
DISTRICT						
NEIGHBORHOOD						
AVENUE / STREET						
BUILDING NO.						
BUILDING NAME						
MAP SHEET						
BLOCK						
LOT / PARCEL						
UAVT BUILDING CODE						
ESTIMATED BUILDING AGE						
GEOGRAPHIC COORDINATES	LATITUDE:		LONGITUDE:			
BUILDING USE TYPE	☐ RESIDENTIAL	☐ COMMERCIAL	☐ INDUSTRIAL	☐ PUBLIC	☐ ABANDONED	
BUILDING TECHNICAL INFORMATION						
STRUCTURAL SYSTEM TYPE	☐ RC FRAME		☐ RC FRAME AND SHEAR WALL			
NUMBER OF FREE FLOORS (N _{sk})						
BUILDING VISUAL CONDITION	☐ GOOD		☐ FAIR	☐ POOR		
SOFT STORY / WEAK STORY	☐ YES / PRESENT		☐ NO / ABSENT			
VERTICAL IRREGULARITY	☐ YES / PRESENT		☐ NO / ABSENT			
HEAVY OVERHANGS	☐ YES / PRESENT		☐ NO / ABSENT			
PLAN IRREGULARITY	☐ YES / PRESENT		☐ NO / ABSENT			
SHORT-COLUMN EFFECT	☐ YES / PRESENT		☐ NO / ABSENT			
BUILDING CONFIGURATION	☐ DETACHED		☐ ATTACHED	☐ END-OF-ROW / CORNER-ATTACHED		
FLOOR LEVEL RELATIVE TO ADJACENT BUILDINGS	☐ SAME		☐ DIFFERENT			
NATURAL GROUND SLOPE	☐ FLAT		☐ SLOPING (SLOPE > 30°)			
SOIL CLASS	☐ ZA	☐ ZB	☐ ZC	☐ ZD	☐ ZE	
NOTES:						

Figure 1. Data Collection Form for Reinforced Concrete Buildings - RYTEİE, 2019

The Data Collection Form for Reinforced Concrete Buildings was used to systematically document building-specific information relevant to preliminary seismic risk assessment. The first section of the form records building identification and location details, including the building identification number, province, district, neighborhood, street address, building name, map sheet, block and parcel numbers, UAVT building code, and estimated construction age. It also includes fields for the inspection date, record number, and geographical coordinates

(latitude and longitude). A photograph of the building's front facade is required to facilitate identification and document its external condition. Furthermore, the form records the building's intended use, classified as residential, commercial, industrial, public, or abandoned.

The second section documents the building's technical characteristics and observable indicators of potential seismic vulnerability. It identifies the structural system, distinguishing between reinforced concrete frame systems and reinforced concrete frame systems with shear walls, and records the number of free floors. The building's visible condition is classified as good, fair, or poor. Additional indicators include the presence of soft or weak stories, vertical and plan irregularities, heavy overhangs, and short-column effects. The form also records the building configuration (detached, attached, or end-of-row/corner-attached), differences in floor levels relative to adjacent buildings, natural ground slope, and soil classification (ZA–ZE). A dedicated notes section allows inspectors to document additional observations that may be relevant to the building's seismic performance.

Collectively, the information recorded in the form provides a structured basis for documenting building characteristics and identifying observable indicators of potential seismic vulnerability. These records support the preliminary assessment of the surveyed building stock and help identify structures that may require more detailed engineering investigation. However, the information obtained through visual inspection should be interpreted as a preliminary screening outcome rather than a definitive determination of structural safety.

3. Results and Discussions

This section presents the results of the street-level seismic risk assessment conducted in Nazımiye District, Tunceli, Türkiye. The findings are discussed in terms of the distribution of building risk categories, the observable characteristics of the surveyed structures, and the spatial representation of the assessment results.

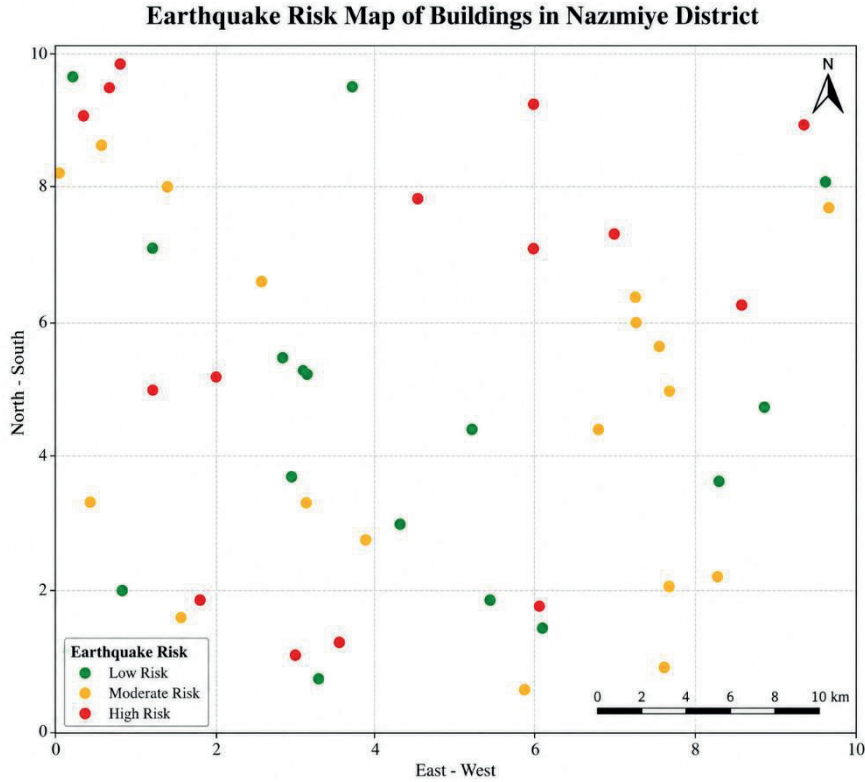


Figure 2. Earthquake Risk Map of Nazımiye District

Figure 2 illustrates the spatial distribution of the preliminary earthquake risk classifications assigned to the surveyed buildings in Nazımiye District. The buildings are categorized as low, moderate, or high risk, with different colours used to distinguish the respective risk classes. This graphical representation provides an overview of the assessment results and facilitates the interpretation of the assigned risk categories in relation to the surveyed building stock.

Within the scope of this study, assessment forms were completed for approximately 160 buildings following field investigations conducted using the street-level screening method. Standardized forms were used to systematically record the observable characteristics of each building. The recorded parameters included the number of stories, estimated construction age,

structural material, visible physical condition, building occupancy type, and overall construction quality. These characteristics constituted the primary data used in the preliminary seismic risk assessment.

The data collected during the field investigations were subsequently digitized and organized using Microsoft Excel. The resulting dataset enabled the systematic organization and classification of the recorded building characteristics and provided a basis for summarizing the assessment results and examining the distribution of risk categories within the study area. This data-processing stage supported the systematic evaluation of the surveyed building stock and the presentation of the study findings.

Table 1. Street screening method assessment summary table

Building ID	Building Age	Visual Quality	Soft Story / Weak Story	Irregularity	Heavy Overhang / Pounding	Natural Ground Slope	Evaluation
1	50	Moderate	None	None	None	Flat	Moderate Risk
2	15	Moderate	None	None	None	Flat	Moderate Risk
3	60	Poor	None	None	None	Sloped	High Risk
4	60	Poor	None	None	None	Sloped	High Risk
5	60	Poor	None	None	None	Sloped	High Risk

The examples presented in Table 1 illustrate how observable building characteristics were considered in the preliminary seismic risk classification. In the examples provided, buildings with poor visual condition and greater age were classified as high risk, whereas those with moderate visual condition were assigned to the moderate-risk category. These examples demonstrate the role of field observations in the screening process; however, they should not be regarded as representative of the entire surveyed building stock.

Building characteristics such as heavy overhangs, visible physical deterioration, and inadequate maintenance may indicate potential vulnerabilities under earthquake loading. Accordingly, these features were considered relevant to the preliminary risk assessment. Older buildings exhibiting visible signs of deterioration, including cracks and deformations, may require particular attention during further investigation. Nevertheless, the presence of

individual visual indicators should be interpreted within the overall assessment framework rather than as definitive evidence of structural inadequacy.

The assessment results indicated that approximately 51.6% of the surveyed buildings were classified as high risk, while the remaining 48.4% were classified as moderate or low risk. These findings indicate that a substantial proportion of the assessed buildings may be vulnerable to earthquake-induced damage and therefore warrant further investigation. The results also highlight the importance of considering building age, visible condition, and other recorded characteristics when prioritizing structures for detailed structural assessment.

Overall, the findings provide preliminary information on the distribution of seismic risk among the surveyed buildings in Nazımiye District. The risk classifications, together with the accompanying tables and spatial representation, provide a basis for identifying buildings that may require further evaluation. However, these results reflect a rapid screening assessment and should not be interpreted as definitive measurements of structural safety. Detailed engineering investigations are necessary to determine the actual seismic performance of individual buildings and to establish appropriate risk-reduction measures.

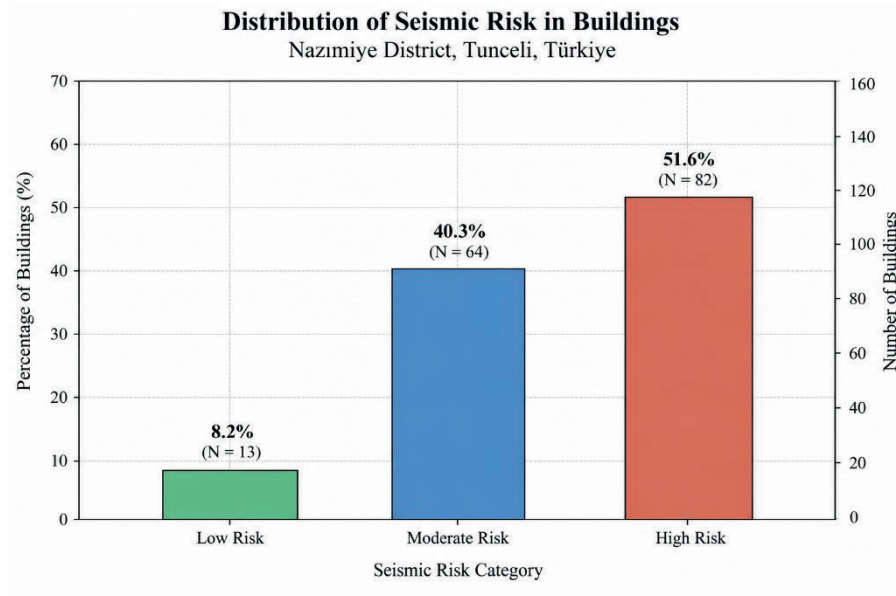


Figure 3. Earthquake risk distribution of buildings in Nazımiye District

The distribution of seismic risk classifications among the surveyed buildings in Nazımiye District is presented in Figure 3. The results indicate that 51.6% of the assessed buildings were classified as high risk, 40.3% as moderate risk, and 8.2% as low risk. High-risk

buildings therefore constituted the largest proportion of the surveyed sample, while low-risk buildings represented the smallest share. These findings provide an overview of the preliminary risk distribution within the assessed building stock.

The substantial proportion of buildings classified as high risk highlights the need to prioritize these structures for further investigation. Buildings assigned to the moderate-risk category also represent a considerable share of the sample and may require additional evaluation to clarify their structural condition. Accordingly, the screening results can support the prioritization of subsequent inspections and help inform local earthquake risk-reduction planning.

The results should, however, be interpreted within the limitations of the street-level screening approach. The reported percentages represent preliminary risk classifications based on the criteria applied during the assessment; they do not constitute definitive measurements of structural safety or direct estimates of earthquake-induced damage. Detailed engineering investigations are therefore necessary to determine the actual seismic performance of individual buildings and to establish whether strengthening, retrofitting, or other interventions are required.

Overall, the findings demonstrate the usefulness of street-level screening as an initial tool for identifying buildings that may require further assessment in Nazımiye District. The risk classifications and their graphical representation provide baseline information on the surveyed building stock and may assist local authorities in prioritizing detailed structural investigations. Further engineering assessments are essential to validate the preliminary classifications and to develop appropriate measures for reducing seismic vulnerability and improving the resilience of the existing building stock.

4. Conclusion

The results of the street-level screening assessment indicate that 51.6% of the surveyed buildings in Nazımiye District were classified as high risk, 40.3% as moderate risk, and 8.2% as low risk. The predominance of the high-risk category highlights the importance of identifying potentially vulnerable buildings and prioritizing them for further investigation. The examples presented in Table 1 also show that buildings with poor visual condition and sloping ground were classified as high risk, whereas buildings with moderate visual condition and flat ground were classified as moderate risk. However, these examples are limited and do not establish a causal relationship between individual building characteristics and risk classification. Overall, the findings provide a preliminary overview of the seismic risk distribution within the surveyed building stock and emphasize the need for detailed structural assessments to verify the actual seismic performance of the buildings.

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