Department of Digital Business
Journal of Artificial Intelligence and Digital Business (RIGGS)
Homepage: https://journal.ilmudata.co.id/index.php/RIGGS
Vol. 5 No. 1 (2026) pp: 2423-2436
P-ISSN: 2963-9298, e-ISSN: 2963-914X

Redesign of a 60,000 DWT Dry Bulk Jetty from Precast to Cast-In-Situ System in a Seismic Port Area: Structural Design and Cost Implication

Gandhi Kesuma[1], Kohar Yudoprasetyo[2], Raden Buyung Anugraha Affandhie[3]
[1]Student Department of Civil Infrastructure Engineering, Faculty of Vocation, Institut Teknologi Sepuluh Nopember
[2,3]Department of Civil Infrastructure Engineering, Faculty of Vocation, Institut Teknologi Sepuluh Nopember
[1]gandhik.wrk@gmail.com, [2]kohar.yudo@its.ac.id, [3]ba_affandhie@its.ac.id

Abstract

This study presents the redesign and structural verification of a 60,000 DWT dry bulk jetty in the coastal area of Gresik, East Java, where the project site is characterized by soft ground (Site Class SE) and requires earthquake-resistant performance. The original structural concept is reconfigured from a precast system to a cast-in-situ reinforced concrete system to improve continuity, connection robustness, and constructability in a seismic port environment. A three-dimensional numerical model was developed in SAP2000 to represent the interaction between deck slab, beams, pile caps, and steel pipe piles (SPP Ø1016 mm, t = 15.9 mm). Design actions include dead and operational live loads (3.0 t/m²), crane wheel loads under critical transverse positions, vessel berthing and mooring loads (fender reaction and bollard demand), and response-spectrum earthquake loading based on Indonesian provisions. The redesigned superstructure adopts a 350 mm deck slab with D16–150 reinforcement, and beam detailing is provided for longitudinal, transverse, and crane beams. Foundation checks confirm that axial reactions remain below the pile bearing capacity, pile utilization ratios are below unity, maximum vertical settlement is 0.63 cm (within the allowable limit), and maximum lateral deflection is 85.14 mm (below the serviceability threshold). The lump-sum construction cost implication for the redesigned system is USD 11.062 million.

Keywords: Dry Bulk Jetty; Cast-In-Situ Redesign; Seismic Jetty; Marine Structure; Soft Soil; SAP2000; Steel Pipe Pile; Berthing and Mooring Loads; Cost Implication

1. Introduction

Ports are strategic infrastructures that sustain industrial supply chains by enabling the continuous movement of raw materials and products. In dry bulk logistics, jetty/wharf facilities directly govern berth availability, loading–unloading efficiency and overall port throughput [1], [2]. In highly industrialized coastal corridors, such as Gresik City, East Java, the reliability of dry bulk terminals is becoming increasingly critical because the port system supports large-volume commodity distribution and time-sensitive operations.

Unlike typical onshore structures, a dry bulk jetty is exposed to multi-hazard actions that may occur concurrently: operational loads from cargo handling systems, berthing and mooring forces, and environmental actions such as waves, currents, wind, and tidal effects. In addition, many Indonesian coastal regions are categorized as seismically active, which elevates the design demand and shifts structural performance targets beyond strength requiring strict control of serviceability (deflection, cracking, and joint behavior) and robust load-transfer mechanisms under cyclic actions [3].

Previous engineering studies on Indonesian jetty structures have commonly adopted a deck-on-pile configuration and verified the structural response using 3D numerical modeling (SAP2000) under combined vertical and lateral actions. For example, a cargo jetty designed for a 50,000 GT vessel was analyzed by explicitly checking pile capacity and joint displacement, where embedded piles were represented using nonlinear spring supports to capture soil stiffness effects, and structural serviceability was evaluated under both operational and seismic conditions. The study showed that displacement demands and member utilization can be kept within allowable limits when environmental loads (waves/currents), berthing reactions, mooring actions, and response-spectrum earthquake loads are consistently incorporated in the model [4].

Similarly, a structural planning study for the Tanjung Gudang Belinyu jetty (planned to serve 30,000 DWT) employed SAP2000-based 3D structural analysis, integrating site data (tides, wind, bathymetry, and N-SPT soil information) into the design process. The study also demonstrated how berthing energy can be translated into fender reaction and then applied as a structural load case, alongside mooring loads (bollard) and environmental actions. Importantly, the design utilized inclined piles as a lateral-load resisting strategy, highlighting that jetty performance is often governed by the lateral demand envelope rather than gravity loads alone [5].

However, despite the practical value of these studies, published engineering works rarely address an integrated question that is highly relevant for seismic port infrastructure: how a structural system transition (from precast to cast-in-situ) changes the global behavior, detailing demand, and overall cost implications. Most existing studies evaluate a single structural alternative (typically deck-on-pile) without explicitly positioning the study as a system redesign problem, where continuity, connection vulnerability (for precast), constructability constraints, and marine execution risks become central considerations. [6], [7] This gap is particularly important in seismic port areas, where connection performance and global integrity under cyclic actions can control safety and serviceability.

Therefore, this study focuses on the redesign of a 60,000 DWT dry bulk jetty from a precast system to a cast-in-situ reinforced concrete system in a seismic port area, followed by structural analysis/design verification under combined operational–environmental–seismic actions and an assessment of the cost implication associated with the system change. The results are expected to support decision-making for resilient and economical dry bulk terminal development, particularly for projects requiring system modification due to constructability, performance, or budget constraints.

2. Methods

Location and Soil Condition

Figure 1 Jetty Location

As shown in Figure 1, the planned jetty is located along the coastal area of Gresik City (East Java), Indonesia, positioned on the shoreline zone facing the Madura Strait. The regional geological setting, referring to the Indonesian Geological Map [8] presented in the same figure, indicates that the coastal belt of Gresik is dominated by Quaternary alluvial deposits (Qs), which are typically associated with young sedimentary deposits in coastal environments. This geological setting is consistent with the soft ground condition reported in previous studies in the Gresik coastal reclamation area, where the seabed/subsoil is described as being predominantly soft clay and characterized by a very soft to soft clay layer of significant thickness based on offshore soil investigations (SPT-based stratigraphy) [9]. With the presence of soft soil deposits, the seismic site condition for the jetty is classified

DOI: https://doi.org/10.31004/riggs.v5i1.6551

as Site Class SE, which represents soft soil behavior in the seismic design framework and was adopted for the earthquake loading definition in this study.

Jetty Specification

The preliminary design stage was conducted to establish the key geometric and material specifications of the jetty as the basis for subsequent structural analysis and redesign evaluation. As summarized in Figure 1, the alternative jetty layout consists of a pile-supported wharf with a total length of 528 m and a platform width of 24 m, arranged to accommodate the planned berthing line, mooring dolphin configuration, and vessel approach alignment. The operational level of the deck (apron elevation) was set at +5.0 m relative to LWS, while the required navigation depth was ensured by adopting a berth (dredging) level of −14.0 m LWS, which governs the minimum water depth for the design vessel. The structural materials adopted for the analysis are presented in Table 1, where the concrete compressive strength is taken as $f_c' = 35$ MPa, the reinforcement yield strength is $f_y = 420$ MPa, and the pile steel yield strength is $f_{ypl} = 250$ MPa in accordance with the stated standards. The substructure is supported by steel pipe piles (SPP) with an outer diameter of 1016 mm and a wall thickness of 15.9 mm, which were used consistently in the modelling and foundation design to represent the primary load-carrying elements transferring vertical and lateral actions from the deck system to the underlying soil.

Table 1 Material Specifications of Jetty

Symbol	Description	Value	Standard
f_c	Concrete compressive strength (MPa)	35	ACI / SNI 2847 [10], [11]
f_y	Yield strength of the Rebar (MPa)	420	BjTS 420B [10]
f_{ypl}	Yield strength of the pile foundation (MPa)	250	ASTM A252 Gr. 2 [12]

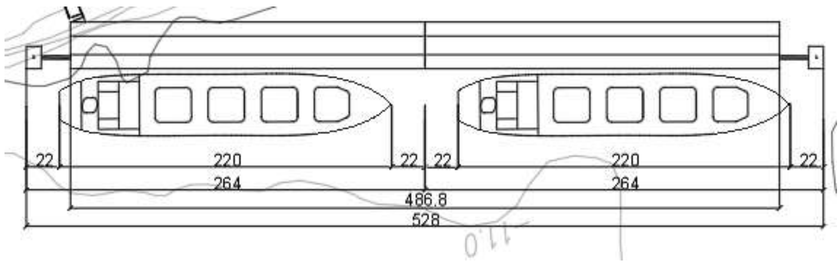


Figure 2 Layout of Alternative Jetty Design

Based on Figure 3 presents the typical cross-section of the jetty and the design berthing envelope for the design vessel. Figure 3a illustrates the Fully Loaded Condition, representing the maximum vessel draft during berthing, while Figure 3b shows the Empty Load Condition, where the vessel draft is smaller and the under-keel clearance is less critical. The jetty superstructure is supported by steel pipe piles (SPP) and referenced to tidal levels (HWS and LWS) to ensure consistent evaluation of navigation clearance and structural demand. To enable safe berthing and maneuvering of the maximum design vessel capacity of 60,000 DWT, dredging is required to provide sufficient water depth at the berthing line. As indicated in the figure, the dredging level is set deeper than the existing seabed, forming a dredged pocket/channel that ensures adequate under-keel clearance for the 60,000 DWT vessel, particularly under the Fully Loaded Condition which governs the minimum required depth.

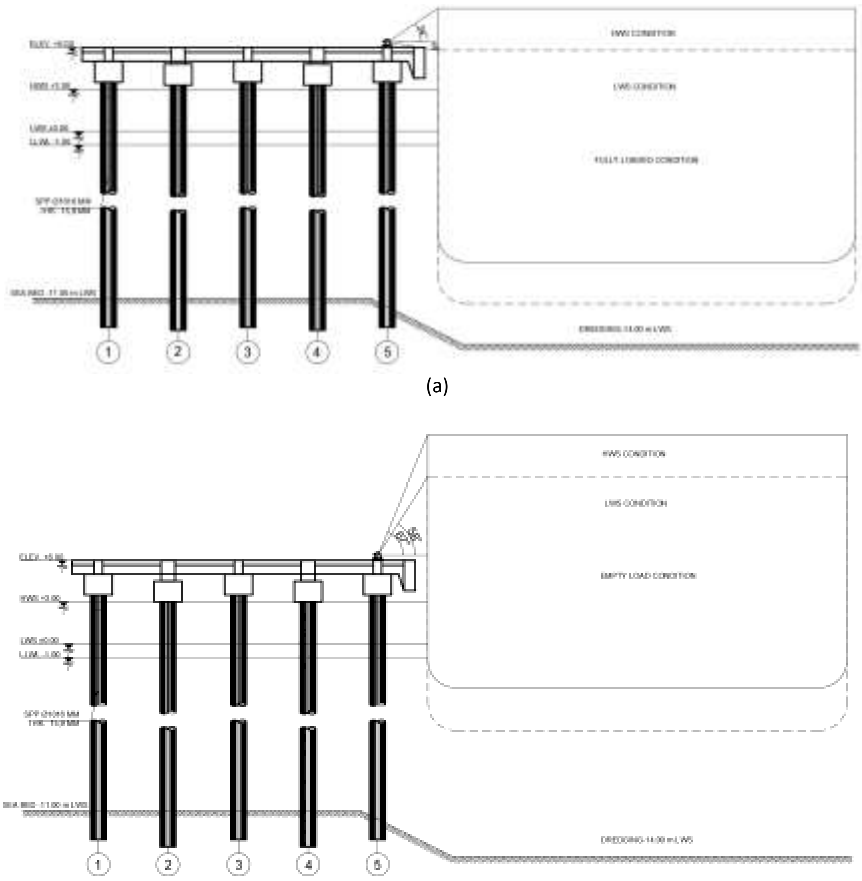
Structural Modelling

The jetty structure was modelled in SAP2000 [13] to evaluate the internal forces and global responses of the structural system under the specified load combinations [14]. The numerical model represents the interaction between the deck slab, longitudinal and transverse beams, pile caps, and steel pipe piles as an integrated pile-supported wharf system. As illustrated in Figure 4, the superstructure framing is idealized using frame elements for beams and pile components, while the deck slab is modelled using shell elements to capture two-way slab

DOI: https://doi.org/10.31004/riggs.v5i1.6551

action and distribute loads to the supporting beams and pile caps. All structural members were assumed to behave within the linear-elastic range for internal force extraction, which was subsequently used for member design checks.

For the pile foundation modelling, the fixity point concept was adopted to represent the effective lateral boundary condition of the embedded steel pipe piles. In this approach, the pile is treated as a beam supported by the surrounding soil (i.e., beam on elastic foundation), where the fixity length denotes the depth at which the pile can be idealized as effectively fixed for lateral bending behavior. This idealization provides a practical estimate of the effective embedment length required to achieve adequate stiffness and capacity under combined axial and lateral demands, particularly for pile-supported marine structures. The fixity length is governed by soil stiffness/strength, pile flexural rigidity, and loading conditions, and it is commonly supported by empirical formulations and complementary numerical interpretation to account for soil–pile interaction effects [15]. The resulting pile boundary representation was implemented in the SAP2000 model shown in Figure 4, enabling consistent evaluation of deck–pile load transfer and pile force demands under operational, environmental, berthing/mooring, and seismic actions.



(a)

(b)

Figure 3 Jetty Cross Section, (a) Fully Load Condition; (b) Empty Load Condition

Loading on the Jetty

Structural loads applied to the jetty structural elements were defined in accordance with relevant port engineering and structural design standards. The load cases considered in this study include dead loads, operational live loads, crane loads, berthing and mooring loads, environmental loads (e.g., wind, wave, current, and tidal effects where applicable), and seismic loads. Load magnitudes as well as load combinations were established by referring to the Technical Standards and Commentaries for Port and Harbour Facilities in Japan [16], Port of Long Beach Wharf Design Criteria [17], SNI 2847:2019 [10], SNI 1725:2016 [18], and SNI 2833:2016 [19].

DOI: https://doi.org/10.31004/riggs.v5i1.6551

The operational live load on the jetty platform was determined following the Standard Design Criteria for Ports in Indonesia (1984), Section V.2. This live load represents the maximum service condition on the jetty apron due to cargo handling activities, temporary stockpiling, vehicle movements, and general operational demands. In the numerical model, the live load was idealized as a uniformly distributed area load (UDL) acting over the platform surface.

In this study, the operational live load was modelled as a uniform area load of 3.0 t/m² applied to the jetty deck. This value was adopted to represent the critical operational loading scenario on the jetty surface. For structural analysis purposes, the live load was assigned to the relevant platform zones to reflect realistic loading patterns along the jetty length, consistent with the planned operational arrangement. The implementation of the uniform area load in the structural model is illustrated in Figure 5.

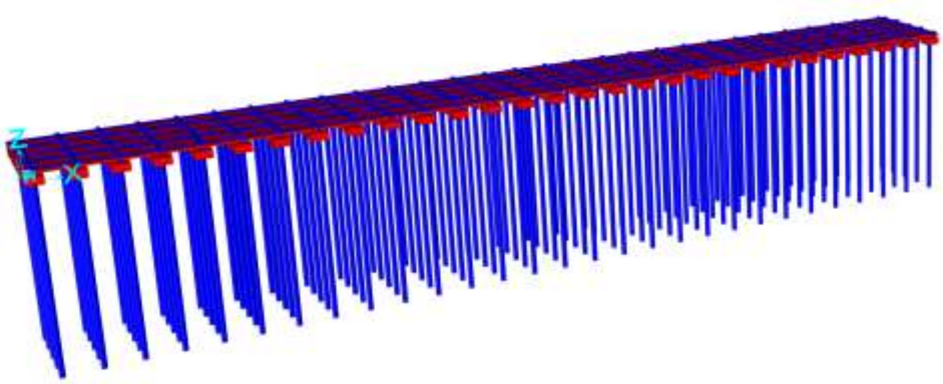


Figure 4 Jetty structure modeling

Crane loads were defined based on the technical specifications provided in the equipment brochure for a grab-type ship unloader. The brochure classifies the unloader into small, medium, and large categories according to the target vessel size. Since the design vessel in this study is 60,000 DWT, the crane specification was selected from the "small" category (applicable for vessels up to approximately 80,000 DWT). The corresponding operational characteristics include a lifting capacity of 20–35 tons, unloading capacity of 1000–1500 tph, and typical geometric ranges such as outreach 30–33 m, rail span 11–20 m (Figure 6), and backreach 0–20 m. The brochure also provides indicative typical wheel loads, reported in the range of 35–45 tons per wheel (and line load equivalent 30–40 tons/m), which were used as the basis for structural load input. In this work, a conservative approach was adopted by taking the upper-bound wheel load values from the brochure and converting them into equivalent concentrated wheel loads acting on the crane rail beams, consistent with the structural analysis unit system [20].

Figure 5 Uniformly Distributed Live Load on Jetty

In the structural model, the crane action was applied as discrete wheel loads along the rail lines to represent critical operating positions that may govern both local and global responses of the jetty deck system. The crane load input was evaluated under three loading conditions corresponding to the most unfavorable transverse positioning across

DOI: https://doi.org/10.31004/riggs.v5i1.6551

the jetty width (Figure 7), namely: (i) edge position (waterside), where the crane is placed near the outer edge of the jetty causing maximum torsional demand and higher local bending on the edge beams/rail supports (Figure 7a); (ii) middle position, where the crane is located at the mid-width of the jetty representing a more balanced configuration and typically governing the global flexural response of the deck system (Figure 7b).

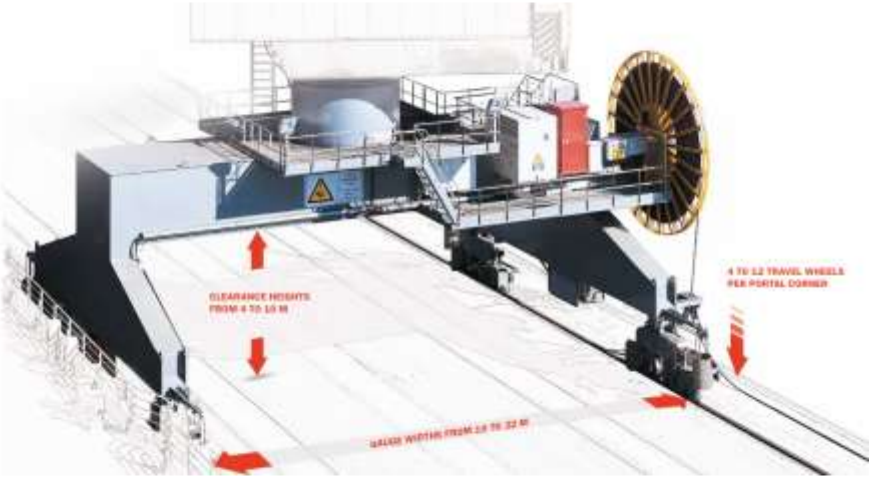


Figure 6 Crane Wheel Load Arrangement [20]

These three crane positions were introduced to ensure that both local effects (rail beam and transverse beam demands) and global effects (deck system response and pile group force redistribution) were captured under the critical crane operating envelope.

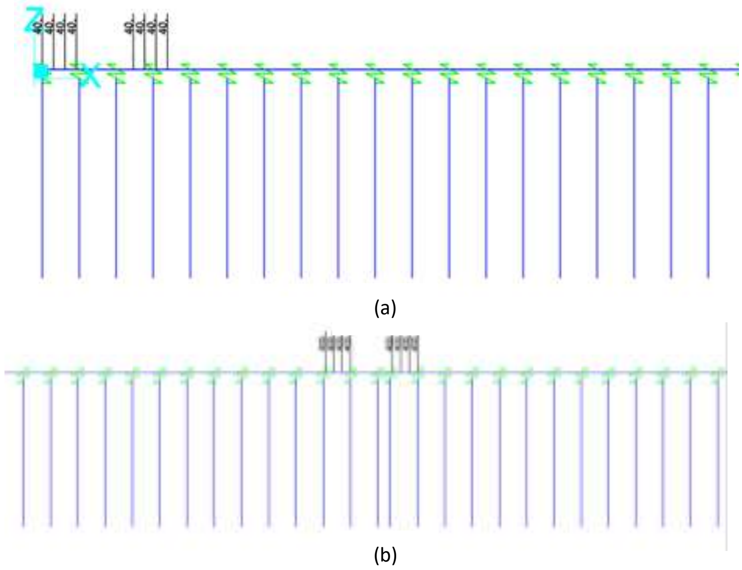


Figure 7 Crane Load Input, (a) Location on edge of jetty, (b) Location on middle of jetty

Vessel-induced actions on the jetty were considered through berthing and mooring loads. The effective berthing energy was evaluated using the standard port engineering approach recommended by PIANC [21], where the ship's kinetic energy at contact is calculated from the design vessel displacement (including added-mass effects) and the assumed berthing velocity, and then corrected by coefficients representing the berthing configuration (e.g., eccentricity and angle of approach) and the energy-sharing characteristics between the fender system and the jetty structure. The resulting berthing energy demand was subsequently used to select/verify the fender performance (energy absorption and reaction), and the corresponding fender reaction was applied to the structural model at the fender elevation as the governing horizontal action during berthing. While the exact formula is not given explicitly in the retrieved context, in the context of port and dock engineering, berthing energy is generally expressed as:

DOI: https://doi.org/10.31004/riggs.v5i1.6551

$$E = \frac{1}{2} \times M \times V^2 \times C \tag{1}$$

where E is the effective berthing energy, m is the ship's mass, v is the berthing velocity, and C is a coefficient accounting for energy absorption and berthing conditions.

This equation quantifies the energy transferred during the berthing process, which must be resisted by the jetty structure and fender system to avoid damage. The dynamic responses of the jetty under such impact loading, including internal forces and potential pile damage, depend strongly on this effective berthing energy [22]. Moreover, PIANC guidelines [21] emphasize considering dynamic amplification factors as high as 1.5 to account for dynamic loading effects during berthing incidents.

Thus, the PIANC effective berthing energy calculation provides a critical parameter for the structural design and safety assessment of wharves, ensuring adequate resistance to vessel collision loads and preventing structural damage and fatigue [22].

The determination of bollard capacity for mooring systems is a critical aspect of ensuring the safe mooring of vessels subjected to environmental forces, including wind and current, consistent with PIANC guidelines for mooring analysis. PIANC provides methodologies to evaluate the environmental loads acting on a moored vessel, emphasizing the forces induced by wind and current which directly influence the design and sizing of mooring components such as bollards.

The berthing load for the jetty was defined based on the design berthing energy generated during vessel approach and contact with the berth. The fender system was selected by comparing the calculated design berthing energy demand with the energy absorption capacity of the selected fender, ensuring that the fender can dissipate the berthing energy while limiting the reaction transferred to the jetty structure. In this study, a CSS 1700 fender manufactured by the Shibata Fender Team was adopted. The selected fender has a height of 2.1 m and a weight of 2,724 kg, with a performance capacity characterized by a rated energy absorption of 940 kNm and a corresponding fender reaction of 1,255 kN at the selected working condition. The design berthing energy demand (referred to as mooring/berthing energy in the calculation) was 1,204.19 kNm, and therefore the effective design check was performed using the manufacturer's performance curve to ensure adequate energy absorption under the prescribed compression. As shown in Figure 8, the fender working point was defined at a deflection of 50% (Ci = 50%), where the energy ratio reaches approximately 70% and the reaction ratio is approximately 95% of the rated value, and these ratios were used to determine the applied horizontal berthing action in the structural model. The resulting fender reaction at the design deflection was then applied as the governing berthing load at the fender elevation to represent the lateral impact transferred from the vessel to the jetty during berthing.

Environmental forces, primarily those resulting from wind and current, generate significant loads that must be accounted for in mooring line design and bollard capacity determination. These forces produce tension in the mooring lines through the vessel's response, impacting stability and safety. The bollard capacity must therefore be sufficient to withstand maximum expected forces without failure, including dynamic effects caused by vessel motions and environmental variability. For floating offshore structures such as floating wind turbines, similar principles apply, where mooring lines undergo tension due to combined environmental factors of wind, current, and waves [23], [24].

The analysis involves calculating environmental loads based on meteorological and oceanographic data at the mooring site and simulating their effects on the vessel's mooring system. Wind loads typically depend on wind speed, direction, and the exposure area of the vessel above water, whereas current loads depend on water velocity and submerged hull geometry. The combined wind and current forces induce mooring line tension which translates directly into the pulling forces on the bollard [25], [26].

Moreover, mooring system design criteria dictate that bollard capacity must accommodate the highest expected tension from combinations of environmental loads with appropriate safety factors. This includes adherence to design return periods such as 100-year extreme events. As mooring systems are critical to vessel safety, bollard capacities are validated against anchor uplift forces and maximum breaking strengths of mooring components under these environmental forces [23], [27].

DOI: https://doi.org/10.31004/riggs.v5i1.6551

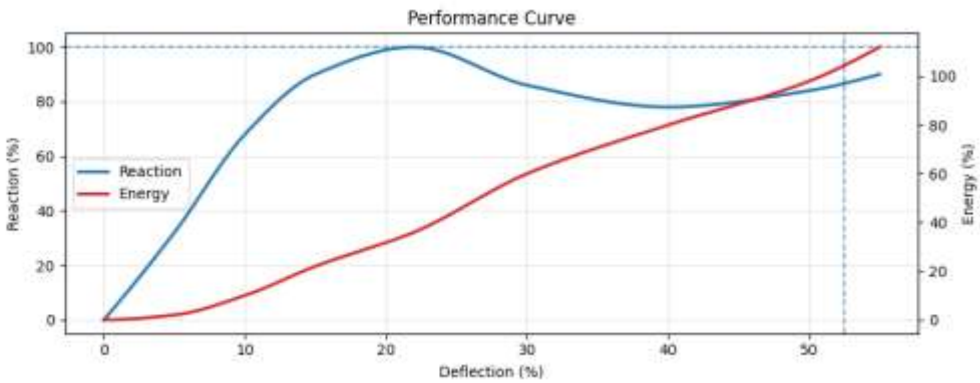


Figure 8 Fender Performance Curve

In summary, PIANC's mooring analysis approach for determining bollard capacity entails detailed evaluation of environmental wind and current forces acting on the vessel, coupling these loads with vessel motion responses to estimate mooring line tensions, and sizing bollards to safely sustain those peak forces under worst-case combined environmental scenarios. This holistic method ensures mooring system integrity and vessel safety during docking or offshore mooring operations.

The bollard system is designed to resist the mooring forces acting on the vessel due to wind and current loads. The wind- and current-induced forces acting on the ship are summarized in.

Table 2 Load on bollard

Ship condition	Load by ship due to			
	Wind (ton)		Current (ton)	
	0°	90°	0°	90°
Empty Vessel Condition	0,25	1,36	1,14	75,5
Full Vessel Condition	0,1	0,55	0,38	226,51

The maximum mooring demand was assumed to occur under current loading when the vessel is in the fully loaded condition, where the combined effects of wind and current must be resisted by the jetty bollards. In this design, the total mooring force was distributed equally to four bollards, such that the design line tension per bollard was calculated resulting in 76.46 tons. Based on this demand, a 100-ton rated bollard was selected, and the bollards were arranged with a spacing of 18 m between adjacent units to ensure adequate mooring capacity and practical operational layout.

Seismic loading for the jetty structure was defined in accordance with SNI 2833:2016 as the primary reference for earthquake-resistant design of port and harbour facilities in Indonesia. The response spectrum method was adopted to represent the dynamic behavior of the pile-supported jetty while accounting for local site effects. The site condition was established based on the soil characterization reported by Irmawan et al., which describes the project area as soft ground dominated by a compressible layer with an approximate thickness of 8 m; therefore, the site was classified as soft soil (Site Class SE) for seismic design purposes [9].

Seismic load analysis in this study was carried out using the response spectrum method in accordance with SNI 2833:2016, which serves as the primary guideline for earthquake-resistant infrastructure design in Indonesia [4]. The response spectrum approach was selected to represent the dynamic behavior of the jetty structure under seismic excitation while considering site-specific soil conditions.

Seismic hazard parameters were taken from the Indonesian seismic hazard database (Lini/seismic hazard map) for the Gresik region, including PGA at bedrock = 0.232 g, $S_s = 0.471g$, and $S_1 = 0.178g$. To obtain the ground-

DOI: https://doi.org/10.31004/riggs.v5i1.6551

surface design spectrum, these parameters were modified using the site amplification factors FPGA, F_a, and F_v, resulting:

$$A_s = FPGA \times PGA \tag{2}$$

$$S_{DS} = F_A \times S_S \tag{3}$$

$$S_{D1} = F_V \times S_1 \tag{4}$$

where A_s is the peak ground surface acceleration coefficient (in units of g), obtained by amplifying the bedrock peak ground acceleration PGA using the zero-period amplification factor $FPGA$ to account for local site effects. The term S_{DS} denotes the design spectral acceleration at short periods, calculated by modifying the mapped short-period spectral acceleration S_s with the site coefficient F_a, which represents the amplification of high-frequency ground motion due to the local soil class. Similarly, S_{D1} is the design spectral acceleration at a period of one second, derived by adjusting the mapped one-second spectral acceleration S_1 with the site coefficient F_v, reflecting the amplification of longer-period ground motion associated with the soil profile. Therefore, Equations (5)–(7) provide the site-adjusted seismic parameters required to construct the ground-surface design response spectrum in accordance with SNI 2833:2016, which is subsequently applied in the response spectrum analysis of the jetty structural model.

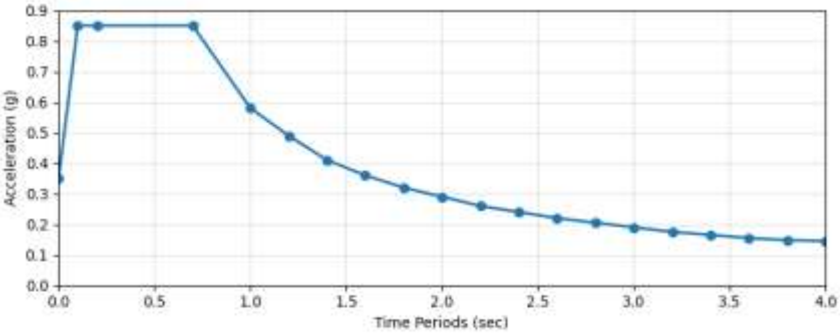


Figure 9 Response Spectrum Curve for Gresik City

Load combinations were arranged following the Port of Long Beach Wharf Design Criteria to ensure adequate structural strength, stability, and safety under critical loading conditions [17].

3. Results and Discussions

The structural design results are presented in the form of reinforcement detailing for the deck slab and primary beams of the jetty superstructure. For the deck slab, the adopted thickness is 350 mm, and the reinforcement arrangement for both the support (negative moment region) and field (positive moment region) is specified as D16–150, indicating a uniform reinforcement pattern to satisfy strength and serviceability requirements under the governing load combinations. The corresponding slab reinforcement detailing and its integration with the supporting beams are illustrated in Figure 10 Concrete Slab Reinforcement Detail, which shows the placement of the slab bars across the beam zones and the continuity of reinforcement over the supports.

The reinforcement design of the jetty beams is summarized in Table 3, while the section-by-section detailing is provided in next Figures. Three beam types are considered: B1 (Longitudinal Beam), B2 (Transverse Beam), and B3 (Crane Beam), each designed to resist distinct load paths within the deck system. Beam B1 and B2 are designed with a cross-section of 600 × 1000 mm, whereas the crane beam B3 adopts a larger section of 800 × 1200 mm to accommodate the higher concentrated wheel loads from crane operations. As listed in Table 2, the required longitudinal reinforcement differs between the support region and the field region to reflect the moment demand distribution, while the shear reinforcement is provided using D13 stirrups with spacing varying by beam type and location. Additional torsional reinforcement is also specified to ensure adequate capacity at critical regions where torsional effects are significant, particularly near beam intersections and edge zones. The finalized reinforcement layouts for each beam type—including top and bottom longitudinal bars and stirrup arrangements—are shown in

DOI: https://doi.org/10.31004/riggs.v5i1.6551

Figure 17 for B1, Figure 18 for B2, and Figure 19 for B3, confirming consistency between the calculated design requirements and the detailing adopted for construction drawings.

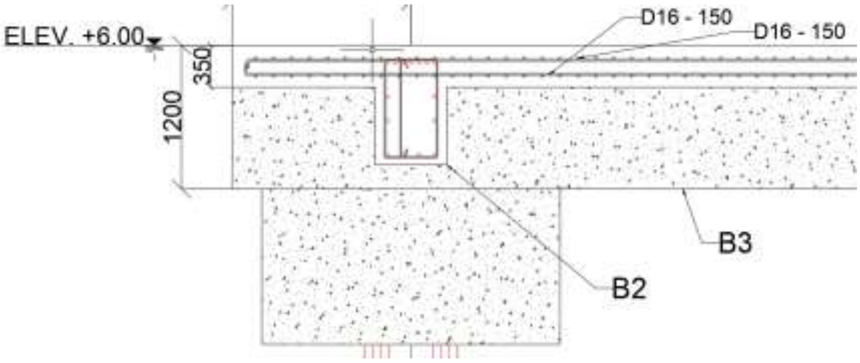


Figure 10 Concrete Slab Reinforcement Detail

Table 3 Recapitulation of beam reinforcement

Beam Type	Dimension		Support reinforcement			Field reinforcement			Torsion Reinforcement
	B	H	Top	Bottom	Stirrups	Top	Bottom	Stirrups	
B1 (long)	600	1000	11D25	10D25	3D13 - 150	6D25	4D25	3D13 - 150	4D25
B2 (short)	600	1000	12D25	11D25	3D13 – 150	7D25	4D25	3D13 – 150	4D25
B3 (crane beam)	800	1200	12D25	10D25	4D13 - 150	6D25	12D25	4D13 - 150	4D25

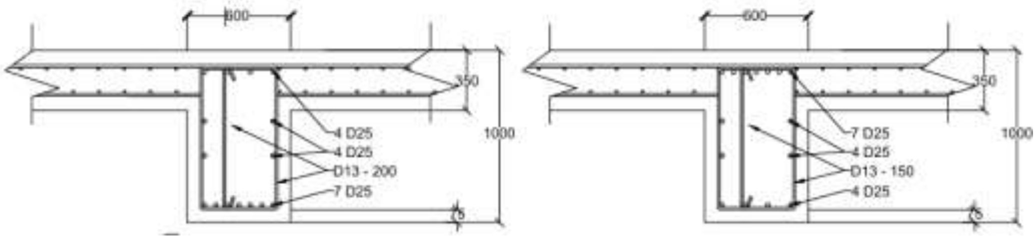


Figure 11 Reinforcement of beam B1

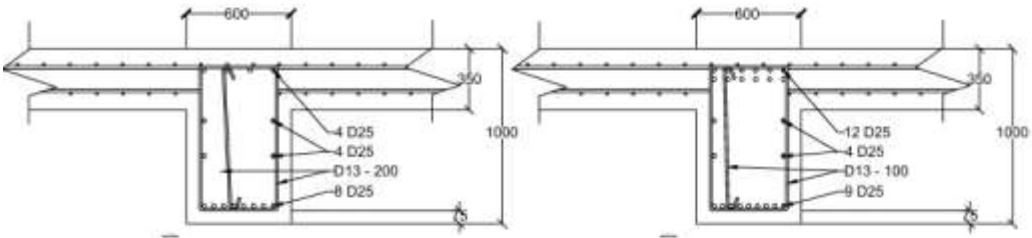


Figure 12 Reinforcement of beam B2

DOI: https://doi.org/10.31004/riggs.v5i1.6551

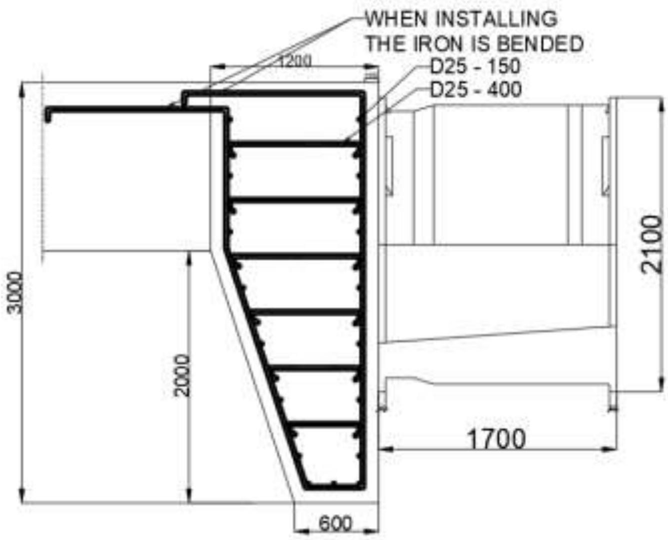


Figure 13 Precast fender plank reinforcement details

The structural detailing of the fender plank was designed to ensure adequate resistance against the horizontal fender reaction transferred during vessel berthing while maintaining durability under repetitive impact in a marine environment. The fender plank planning resulted in a plank thickness of 800 mm with main reinforcement D25–100, which was selected to provide sufficient flexural capacity and crack control under the governing berthing load demand. The finalized reinforcement arrangement and geometric configuration of the precast fender plank are presented in Figure 13, showing the reinforcement layout, anchorage detailing, and the installation-related bending of reinforcement bars to secure proper load transfer between the fender unit and the supporting structural components. This detailing ensures that the fender plank acts as a robust interface element, distributing the fender reaction to the jetty substructure and preventing local damage concentration at the contact region during berthing events.

Table 4 Pile bearing capacity

Pile Diameter (mm)	Combination	Reaction (ton)	Bearing Capacity (ton)	Check
1016	Service	265.96	321.53	Safe

The jetty foundation system employs a single pile type, namely steel pipe piles (SPP) with a diameter of 1016 mm, which were verified through both geotechnical bearing capacity checks and structural capacity checks. The geotechnical evaluation compares the calculated pile bearing capacity against the governing axial reaction obtained from the global structural analysis. As summarized in Table 4, the available bearing capacity of the 1016 mm piles is 321.53 tons, while the maximum axial reaction transmitted to the pile is 265.96 tons, indicating that the foundation satisfies the geotechnical requirement with an adequate margin (i.e., capacity exceeds demand).

In addition to the soil-bearing verification, the structural strength of the steel pipe piles was assessed using the steel design module in SAP2000, where combined axial force and bending effects under the critical load combinations were checked through capacity (stress) ratios. The results show that the governing utilization ratios for all piles remain below 1.0, confirming that the piles meet the structural strength criteria for the applied actions. The spatial distribution of the pile stress ratios is presented in Figure 14, which demonstrates that no pile member exceeds the allowable demand-to-capacity threshold, thereby validating the adequacy of the selected SPP section for the jetty substructure.

DOI: https://doi.org/10.31004/riggs.v5i1.6551

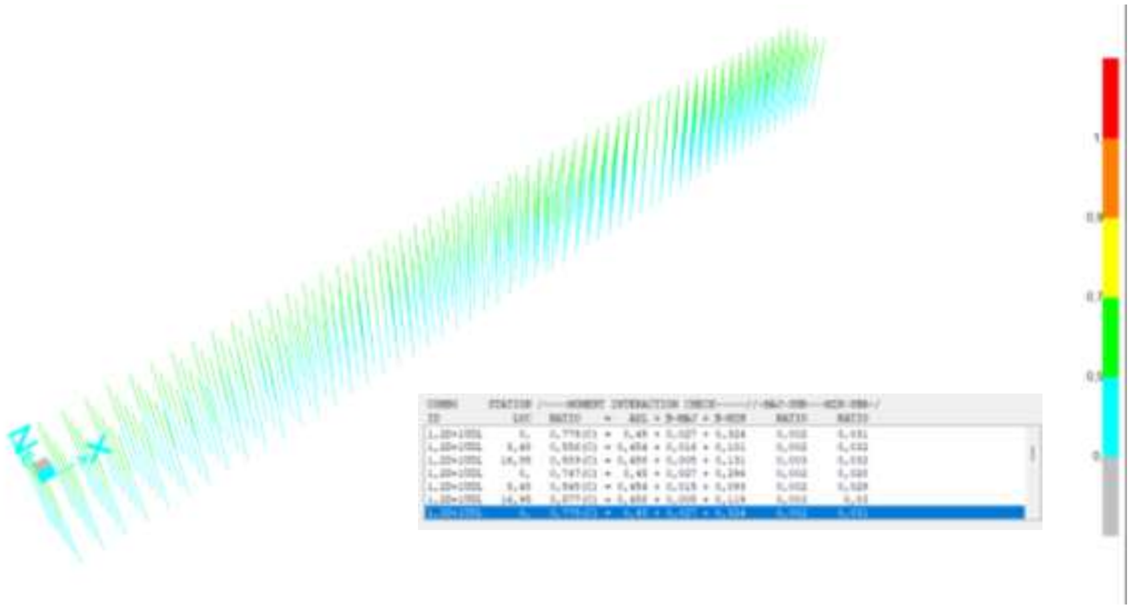


Figure 14 Stress Ratio of Jetty Steel Pipe Piles

The serviceability performance of the pile foundation was verified through a vertical settlement analysis using the AllPile program. The analysis evaluates the load–settlement response of the steel pipe piles by considering the contribution of shaft resistance (side) and end bearing (tip), as illustrated by the load–settlement curve in Figure 15. The results indicate that the maximum settlement of the jetty piles is 0.63 cm, which is significantly lower than the allowable settlement limit of 2.5 cm specified in SNI 8460:2017 [28] for geotechnical serviceability requirements. A similar evaluation was performed for the mooring dolphin piles, yielding a maximum settlement of 0.11 cm, which also satisfies the same allowable criterion. These findings confirm that the selected pile system meets the serviceability limit state with respect to vertical settlement, ensuring adequate stiffness and operational performance of the jetty under the governing vertical load demand.

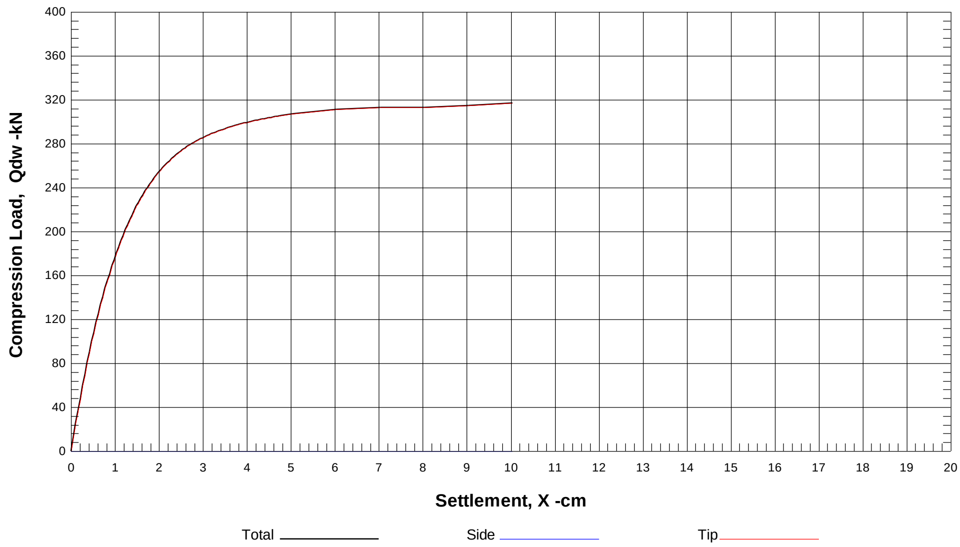


Figure 15 Relationship Vertical Load (kN) and Settlement (cm)

DOI: https://doi.org/10.31004/riggs.v5i1.6551

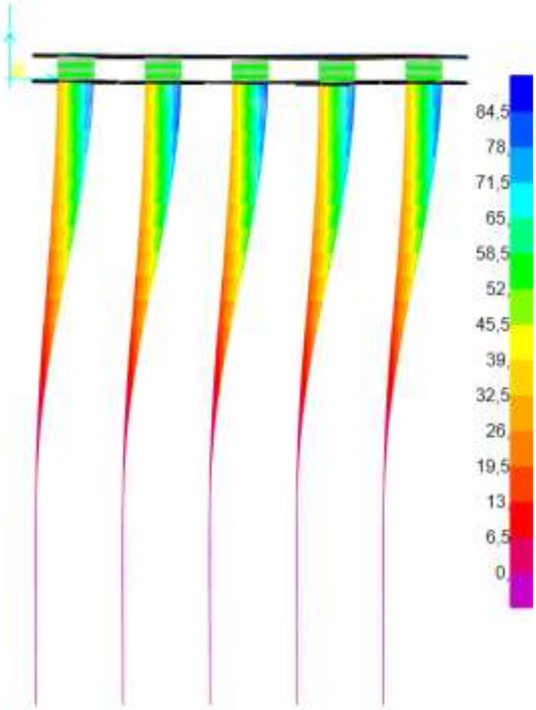


Figure 16 Maximum Deflection in the Jetty

The lateral deflection of the steel pipe pile system was evaluated using the displacement outputs from the SAP2000 global structural model under the governing lateral load combinations. The computed pile-head and overall lateral deflection demands were assessed against the serviceability limits recommended in BS 6349-2:2010 (Code of Practice for the Design of Quay Walls, Jetties and Dolphins) [29], which specifies a maximum allowable lateral deflection of 100 mm for important marine structures. The analysis indicates that the maximum lateral deflection of the jetty piles is 85.14 mm, which is below the prescribed limit. As shown in Figure 16, the displacement contours confirm that the pile group deformation remains within acceptable bounds, demonstrating that the foundation system provides adequate lateral stiffness to resist berthing- and mooring-induced actions while satisfying the serviceability performance requirements.

The cost estimation in this study was prepared at a lump-sum level to reflect the overall budget requirement for the redesigned dry bulk jetty, covering the main construction scope including steel pipe pile installation, pile cap construction, reinforced-concrete beam works, deck slab works, plank fender installation, and the associated berthing and mooring facilities. Based on this consolidated estimate, the total construction cost of the dry bulk jetty is USD 11,062,000. This value represents the aggregated cost of the principal structural and marine facilities required to deliver the jetty system under the proposed redesign scenario.

4. Conclusion

The redesign of the 60,000 DWT dry bulk jetty in Gresik was successfully developed using a cast-in-situ reinforced concrete system to enhance structural continuity and robustness in a seismic port area founded on soft soil (Site Class SE). The jetty was evaluated using a three-dimensional SAP2000 model that captured the interaction of the deck slab, beams, pile caps, and steel pipe piles (SPP Ø1016 mm, t = 15.9 mm) under the governing load combinations, including operational live load, crane loading at critical positions, berthing and mooring actions, and response-spectrum seismic loading. The superstructure detailing—comprising a 350 mm deck slab reinforced with D16–150 and the designed reinforcement for longitudinal, transverse, and crane beams—was shown to satisfy the strength demands derived from the analysis. The foundation system also met both geotechnical and structural requirements: the maximum axial pile reaction remained below the computed bearing capacity, pile utilization ratios were below unity, and serviceability criteria were satisfied with a maximum vertical settlement of 0.63 cm and a maximum lateral deflection of 85.14 mm, both within the applicable allowable limits. Overall, the proposed redesign provides a structurally adequate and serviceable jetty solution, with an estimated lump-sum construction cost of USD 11.062 million, representing the aggregated budget requirement for the main structural and berthing–mooring facilities under the cast-in-situ configuration.

References

[1] J. Al-Hammadi and A. Diabat, “An integrated berth allocation and yard assignment problem for bulk ports: Formulation and case study,” RAIRO-Oper. Res., vol. 51, no. 1, pp. 267–284, 2017.

[2] A. Pabón-Noguera, M. G. Carrasco-García, J. J. Ruíz-Aguilar, M. I. Rodríguez-García, M. Cerbán-Jimenez, and I. J. Domínguez, “Multicriteria Decision Model for Port Evaluation and Ranking: An Analysis of Container Terminals in Latin America and the Caribbean Using PCA-TOPSIS Methodologies,” Applied Sciences, vol. 14, no. 14. p. 6174, 2024.

[3] R. Alsultani, A. Rwayyih Hasan, and A. Al-Yasir, “Joint hazard fragility analysis of pile foundation bridge piers in coastal soft soil areas subjected to water waves and earthquake actions,” ISH J. Hydraul. Eng., pp. 1–27, Sep. 2025.

[4] E. Hafudiansyah and G. R. Prima, “Analisa Struktur Dermaga Cargo Dengan Kapasitas Kapal 50.000 GT (Studi Kasus: Pelabuhan Kolaka Sulawesi Tenggara),” Akselerasi J. Ilm. Tek. Sipil, vol. 2, no. 1, pp. 1–12, 2020.

[5] D. Yanti, I. Gunawan, and E. S. Hisyam, “Perencanaan struktur dermaga pelabuhan tanjung gudang belinyu kabupaten bangka,” J. Fropil, vol. 6, pp. 1–13, 2018.

[6] H. Huang, M. Li, Y. Yuan, and H. Bai, “Experimental Research on the Seismic Performance of Precast Concrete Frame with Replaceable Artificial Controllable Plastic Hinges,” J. Struct. Eng., vol. 149, Jan. 2023.

[7] Z. Wang, G. Xing, J. Huang, Z. Chang, and D. Luo, “Seismic Performance of Precast Concrete Exterior Beam-Column Joint with Disc Spring Devices,” J. Earthq. Eng., vol. 28, pp. 1–28, Jul. 2024.

[8] J. B. Supandjono, K. Hasan, H. Panggabean, D. Satria, and Sukardi, “Peta Geologi Lembar Surabaya & Sapulu, Jawa,” Geological Map of the Surabaya & Sapulu Quadrangle, Jawa. Pusat Penelitian dan Pengembangan Geologi (Geological Research and Development Centre), Bandung, Indonesia, 1992.

[9] M. Irmawan, K. Yudoprasetyo, A. N. Refani, K. Indrasurya, and D. N. Parwita, “The Evaluation of Pipeline Protection Influenced by Causeway Embankment Using the Finite Element Method (FEM),” Applied Sciences, vol. 14, no. 11. 22-Jun-2024.

[10] Badan Standarisasi Nasional, SNI 2847:2019 Persyaratan beton struktural untuk bangunan gedung dan penjelasan. Jakarta: Badan Standarisasi Nasional, 2019.

[11] American Concrete Institute, Building Code Requirements for Structural Concrete and Commentary (ACI 318M-19). Farmington Hills, MI: Farmington Hills, MI : American Concrete Institute, [2019] ©2019, 2019.

[12] ASTM International, ASTM A252 - Standard Specification for Welded and Seamless Steel Pipe Piles. 2019.

[13] I. Computers & Structures, CSI Analysis Reference Manual For SAP2000, ETABS, SAFE, and CSiBridge. California, 2025.

[14] K. Yudoprasetyo et al., “Upaya Penguatan Keselamatan Prasarana Sekolah melalui Evaluasi Teknis Struktur Kolom pada Bangunan Pendidikan di Bojonegoro,” Inov. J. Pengabdi. Masy., vol. 3, no. 3 SE-Artikel, pp. 255–266, Jan. 2026.

[15] D. He, Y. Cheng, H. Liu, and H. Lin, “Pile–Soil Interaction and Group Pile Effect in Composite Foundation Under Different Pile Length Conditions,” Buildings, vol. 15, no. 8. p. 1248, 2025.

[16] The Overseas Coastal Area Development Institute of Japan, Technical Standards and Commentaries of Port and Harbour Facilities in Japan. Tokyo: The Overseas Coastal Area Development Institute of Japan, 2009.

[17] Port of Long Beach (POLB), Wharf Design Criteria, Version 5.0. California: Port of Long Beach: Long Beach, 2021.

[18] Badan Standarisasi Nasional, SNI 1725:2016 Pembebanan untuk jembatan. Jakarta: Badan Standarisasi Nasional, 2016.

[19] Badan Standardisasi Nasional Indonesia, “SNI 2833-2016. Perencanaan Jembatan Terhadap Beban Gempa.” BSN, Jakarta, 2016.

[20] Konecranes, “Konecranes Gottwald Portal Harbor Cranes,” Hyvinkää, 2015.

[21] PIANC, Pianc fender guidelines 2024. Brussels: PIANC, 2024.

[22] X. Xu, X. Di, Y. Zheng, A. Liu, C. Hou, and X. Lan, “Dynamic Response Characteristics and Pile Damage Identification of High-Piled Wharves under Dynamic Loading,” Applied Sciences, vol. 14, no. 20. p. 9250, 2024.

[23] M. Chen et al., “Study on Mooring Design of 15 MW Floating Wind Turbines in South China Sea,” Journal of Marine Science and Engineering, vol. 12, no. 1. p. 33, 2024.

[24] J. Guo, M. Liu, Z. Fang, L. Xiao, W. Chen, and X. Pan, “Motion and Mooring Load Responses of a Novel 12-MW Semi-Submersible Floating Wind Turbine: An Experimental Study,” J. Offshore Mech. Arct. Eng., vol. 146, pp. 1–26, May 2024.

[25] G. Benassai, A. Campanile, V. Piscopo, and A. Scamardella, “Optimization of Mooring Systems for Floating Offshore Wind Turbines,” Coast. Eng. J., vol. 57, Nov. 2015.

[26] S. S. Sáenz, G. Diaz-Hernandez, L. Schweter, and P. Nordbeck, “Analysis of the Mooring Effects of Future Ultra-Large Container Vessels (ULCV) on Port Infrastructures,” Journal of Marine Science and Engineering, vol. 11, no. 4. p. 856, 2023.

[27] J. Huang et al., “Analysis of mooring performance and layout parameters of multi-segment mooring system for a 15 MW floating wind turbine,” Front. Energy Res., vol. Volume 12-2024, 2024.

[28] Badan Standarisasi Nasional, SNI 8460:2017 Persyaratan perancangan geoteknik. Jakarta, 2017.

[29] British Standards Institution, BS 6349-2:2010 Code of Practice for the Design of Quay Walls, Jetties, and Dolphins,. London: British Standards Institution, 2010.

DOI: https://doi.org/10.31004/riggs.v5i1.6551