Assessment and Back Analysis of a Swaying-Jetty in Dumai Indonesia

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Abstract. A Jetty structure was constructed as a berthing facility for 50,000 DWT vessels at Dumai, Indonesia. After the construction finished, the main jetty tends to sway frequently, and this was easily noticed by people who stand on the top of the deck. An investigation has been carried out to identify what causing this problem to check the performance of the jetty. A full review of design and construction documentation, direct observation of existing structures included with field vibration measurement tests and new soil investigation campaign have been carried out as part of the investigation. After the new soil investigation, the soil profile at the jetty location was found to be worse than expected, since the borehole used for design is located 90 m distance from the main jetty and hence can not represent the actual condition (Nugroho et al., 2019). This finding leads to a main concern on the global stiffness of the structure. Remodeling was conducted accordingly to evaluate the actual performance of the main jetty. The result shows a good agreement with data from field measurements (e.g., vibration test) and confirms the earlier concern, whereas this jetty has structural stiffness issue in which may result in sway effect. The structural analysis indicates that the existing main jetty may experience an excessive displacement of more than 100 mm allowable displacement under SLS condition, and hence mitigation measures are needed to be taken.

Keywords: Jetty assessment, back analysis, swaying jetty, marine structure vibration test, structural stiffness.

1 Background

Dumai district is one of Indonesia's central industrial area for Oil and Crude Palm Oil (CPO) that is located in Riau province in Sumatra Island. Many ports are built here to support the industry to grow. Dumai district in which located on the Dumai strait is bounded by tectonic faults (Cameron et al., 1982). It is known that these faults have shaped the Dumai strait in combination with erosion during a period of low sea level. During the sea-level rise, this trough has been filled with thick covers of sediment (de Vries, 2017). According to research by Rifardi (2001), Dumai Strait is becoming a deposit area of sediments that were transported from both the Indian Ocean and South China sea, lead to quite some thickness of the soft deposit layer dominating the top layer of the seabed in the area.

In 2016, a jetty structure was constructed for a berthing facility of a maximum 50,000 DWT vessel. After the construction finished, this jetty supposed to directly operated by the project owner, at least in early 2017. However, prior to operation, the jetty tends to sway and seem unstable. The main jetty is found to be swaying not only during high water spring event (HWS) but also during the Low water spring event (LWS). The intensity of the sway is considered as unusual if compared to a normal jetty (before service condition). The sways are noticeably by the people standing on top of the deck.

An investigation was carried out, which consists of new soil investigation on the constructed jetty area along with direct observational data of the jetty to identify the problems. The acquired data is then used as an input for back analysis to assess the condition, and field instrumentation of natural vibration test are used to calibrate the model. Sways problem in jetty is known to be occurred due to the lack of the stiffness of the structural components or due to the bad soil-structure interaction as the global stiffness of the structure itself.

2 Existing Jetty Structure Configuration

2.1 Jetty Configuration

The main jetty is connected to the land via trestle and equipped with four mooring dolphins, four berthing dolphins, and one platform, as presented in **Fig. 1**. The main jetty is an island type of open pile structure with berthing elements. The berthing element is three massive concrete blocks, each supported by a pile group of 8 piles. Beam and floor system are installed in between these blocks, with three rows of the main crossing beam at intermediate of the block are made of in-situ concrete. The longitudinal beam and top slab are precast concrete structures. The finishing of the top slab is in-situ concrete with 15 cm thick topping. The as-built length of the main jetty approximately 100.5 m, and width 17 m.

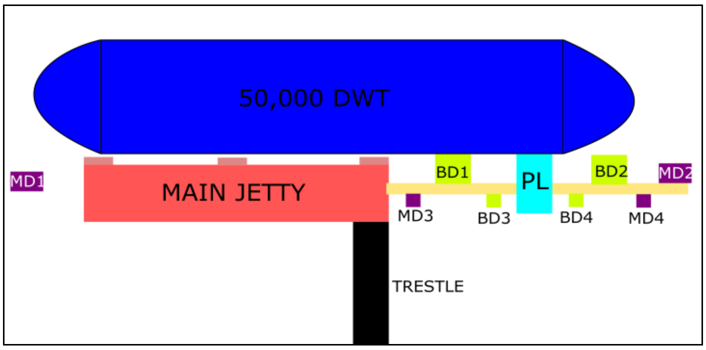

Fig. 1. Jetty structures layout configuration sketch.

2.2 Pile Configuration of The Main Jetty

The main jetty's foundations are configured as nine grids, and each grid is installed with a series of piles. The main jetty has 3 grids for berthing element with eight inclined piles each berthing element pile group. Outside the berthing elements grids, each grid has three piles of each group. In general, the main jetty piles group has a transversal (vertical) center to center distance of 3,500 mm and has 11,500 mm center to center distance in longitudinal (horizontal) direction. The deck design elevation is +5 m+LWS, and the top pile main jetty is +3 m+LWS.

3 Main Jetty Design

3.1 Pile Design of The Main Jetty

The jetty was designed using an initial site investigation consist of 5 boreholes and lab tests data. Those five boreholes are located far from the main jetty. BH-1, the closest borehole of all that has a distance of 90 m, was used to design the main jetty piles. The surface level at the executed boreholes is in the range of +3 to -10 m+LWS, while the jetty location has a seabed level of about -24 to -27 m+LWS.

The 40 m borehole of BH-1 indicates that a dense sand layer with silt mixture is found in depth of 21 m below the seabed level. BH-1 borehole is executed at -10 m+LWS seabed and only has 40 m boring depth; thus, the dense sand information stops at 40 m. Based on the design, this BH-1 soil profile is used for the main jetty design, although the main jetty seabed level is located at -24 to -30 m+LWS, and it was assumed that this dense silty sand would extend to a very deep layer in the design.

As per design calculation based on BH-1, one pile in the main jetty that is located on the -27 m+LWS actual seabed level will have a free length of 30 m from the total length of 62 m. Thus, it is supposed to have 32 m penetration depth, and the piles are supposed to be installed into 21 m stiff layer of fine-grained soils overlying ~11 m dense silty sand layer with SPT more than 40 N-Value. The piles are designed as end-bearing piles and rely significantly on this ~11 m hard layer as lateral and vertical bearing capacity.

Prior to construction, there is no static pile loading conducted to confirm the geotechnical design assumption for the jetty area. Although some PDA tests were conducted during the installation of piles, those PDA tests were executed on the trestle piles and not exactly at the main jetty location.

On the berthing element of the main jetty, the piles are inclined to increase the structural stiffness. The raking piles are designed to have a uniform inclination of 1:5 (H:V) for all structures.

4 Geotechnical Assessment of The Main Jetty

4.1 Actual Soil Condition

Additional soil investigations consist of 6 new additional boreholes (AB1 to AB6) were proposed to confirm the actual soil condition exactly on the jetty location during the assessment process (Nugroho et al., 2019).

Based on the new site investigation, it indicates that the top 1 - 2 m dominated by thick clay deposits with some gravel and sand in certain areas. The next 5 to 7 m consists of a soft to medium clay layer. The clay layer is considered as clay with high plasticity since the PI is 43 (Burmister, 1949). It followed with the medium-dense gravely sand layer with a thickness of about 5 m. Below the gravelly sand layer, medium to hard clays with high plasticity layer is present until 90 m below the seabed layer, with another sand lenses of medium dense relative density, found at depth 20 to 40 m that varies in thickness. The mentioned stiffness consistency of all cohesive soils and the relative density of all granular soils are according to Peck and Terzaghi (1967,1996). The long section of AB1 to AB6 is presented in **Fig. 2**.

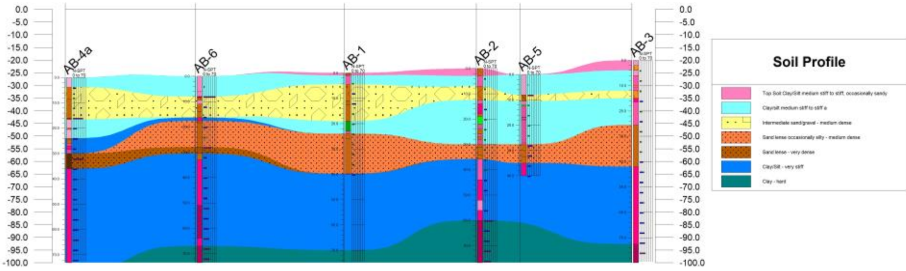

Fig. 2. Long Section of the new boreholes

4.2 Comparison of Actual Soil Condition with Initial Site Investigation with Respect to Pile Bearing Capacity

The new site investigation has brought a completely different view of the geotechnical condition on the main jetty area. The silty sand layer at depth 21 m to 40 m found in BH-1 (initial SI) is later on known as a local lens. The assumption that the dense silty sand layer will extend to a very deep layer is incorrect, confirmed by the new site investigation. BH-1, as used in the design, can not represent the actual condition due to the in inhomogeneity of soil profiles in the waterfront area (Nugroho et al., 2019)

Before having the new site investigation data, the performance of piles can be checked via construction documentation such as the PDA test and CAPWAP analysis. Data from PDA and CAPWAP analysis, as presented in **Table 1**, shows that the highest bearing capacity contribution is from shaft capacity. TRES-37, which is located on the trestle and is considered to be the closest point to the main jetty, shows a total bearing capacity of 3508 kN with the shaft friction capacity of 2878 kN and an end bearing capacity of 630 kN. The contribution of end bearing capacity is only 18 percent out of the total pile capacity. This result is showing a friction pile in which the shaft contribution is quite high, and the pile bearing capacity relies significantly on the friction instead of end-bearing.

Table 1. PDA test result of TRES-37 in the project

Pile Number	Bearing capacity (kN)			
	PDA	CAPWAP		
		Total	Friction	Toe
TRES-37	3508	3508	2878	630

It is given side by side overview comparing new borehole AB-4 and BH-1 in **Fig. 3**. Clearly that until the depth of 40 m below seabed level, the dominant soil layers are medium to very stiff clay, and in the 21 m or deeper barely any dense silty sand layer is found except at depth 30 to 36 m in which a sand layer is found at the new borehole. If the pile to stops at -32 m depth, this means only 2 m of pile toe is covered with sand layer either it is dense or medium dense layer will not give any significant impact to the lateral or vertical bearing capacity. Although the gravelly sand layer is found in the main jetty location, this type of soil will not give a higher friction capacity to the piles, if it is compared to the cohesive soils layer with the same SPT value. The SPT value of this layer is ranging from very low to very high SPT;10 to 50, and definitely, the thickness will vary as well in the whole main jetty area.

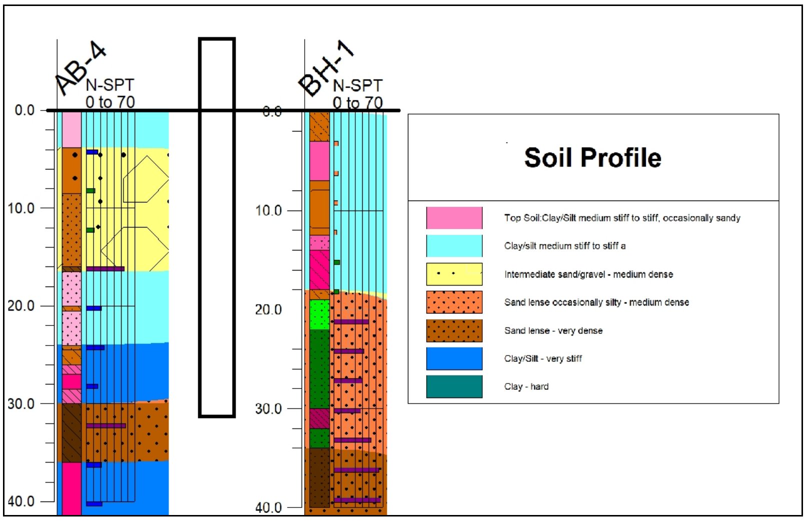

Fig. 3. Side by side comparison of design assumption and the actual soil condition

4.3 Seabed Level Evaluation From 2013 to 2016

During the initial design phase in 2013, prior to the construction phase, the deepest sea bed level in the main jetty location based on the bathymetric survey is at -25 m+LWS. During the construction from 2013 - 2014, it is recorded on the pilling records that the seabed levels can reach up to -26 m+LWS. Here it can be seen that during a period of one year, there is erosion in the seabed level.

During the assessment of the jetty in 2016, a measurement of the seabed level is also be conducted. Based on the latest measurements by the divers, the main jetty location is dominantly located on the -26 to -27 m+LWS. Hence, the seabed is eroded about 1 to 2 m thick during 3 years period.

Some areas in the jetty location have topsoil of clay with sand and gravel mix. Based on laboratory tests conducted from new boreholes, the first 7 m has the mean particle size (D50) on the range of 0.04 to 1.5 mm. Although in this project, no comprehensive sediment transport analysis has been carried out due to the lack of time and data, the seabed erosion causes might be preliminary identified. It is recorded in the area that the maximum current speed near-surface is 1.2 m/s. The seabed erosion might be caused due to relatively high current on the floor area with respect to mean grain size (Hjulström, 1935) as well as the critical shear stress on the bed (Shield, 1936). Another possibility is that the sediments are transported due to the combination of waves and current (Van Rijn, 1995).

5 Structural Assessment

5.1 Field Instrumentation Using the Accelerometer

The vibration test is carried out to the structure by producing initial excitation by hitting the structure with a small tugboat. The natural vibration properties of the structure are measured by putting accelerometer instruments to record the response of the structure during the collision. One of the vibration properties, the natural frequency, is useful information in order to identify the stiffness of the structure. In this project, the natural frequency obtained from the test will be compared with the remodeling result.

The mission of this investigation is to answer whether the main jetty can be operational or otherwise, and hence, it is needed to evaluate the main jetty performance, especially during the berthing of 50,000 DWT vessels. The instrumentation data is used as a benchmark for the product of the back analysis model. A close value between the back analysis model and field measurement will bring a good confidence level to conduct further predictive analysis.

5.2 Finding on Layout Configuration and Actual structure Stiffness

It can be seen on the main jetty layout presented in **Fig. 4**, the longitudinal pile spacing of 11.5 meters (approximately 12.6D) is considered too large. The consequences of having an excessive spacing between piles resulting in insufficiency on the number of piles to withstand design loads as follow:

- High displacement of upper structures
- Result in pile overstressed.

The required maximum pile distance is determined through structural analysis. Since it is known that the actual soil condition is much worse than determined on the design; therefore, 12.6D is simply considered insufficient to withstand loads proven with the back analysis result that is further discussed in section 7.

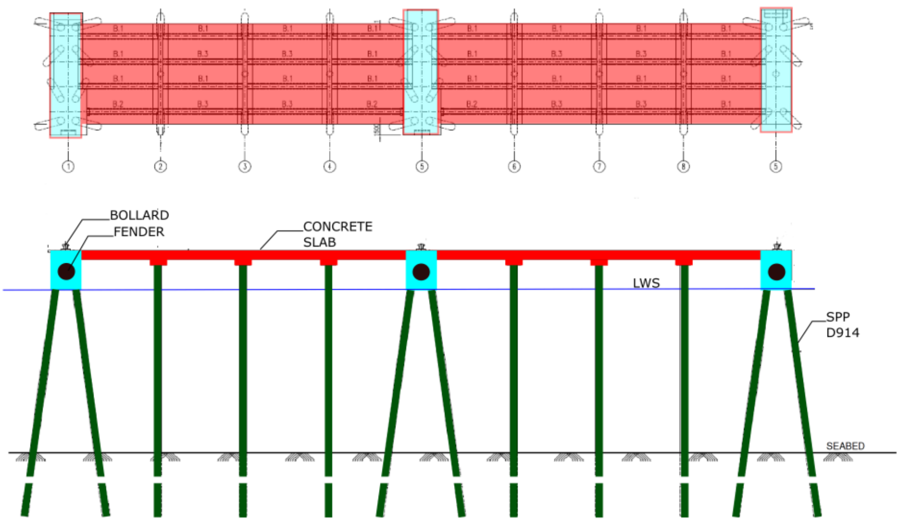

Fig. 4. Main jetty configuration sketch

5.3 Actual Raking Pile Inclination Versus Design Inclination

Based on the underwater inspection conducted, the condition of inspected piles is generally reasonable. There is no indication of deflection, buckling, and ripped. All HDPE (High-Density Poly Ethylene) wrap protection is in good condition, and cathodic protection is in good condition also. However, the inclinations of the raking piles are not similar to the designs. The piles are installed steeper than what was suggested in the design. Most of those piles can not achieve the designed inclination of 1:5 (Horizontal:Vertical) rake. Most of the piles have an inclination of 1:10 (H:V), which is steeper than the design. The raking piles are meant to increase the stability of the jetty structures mainly due to horizontal loads such as berth loads, wave loads, etc. The cause of this inclination problem is most likely due to the slenderness of the piles that are not able to maintain its orientation, and tend to bend by its self-weight when the pile is being installed. The consequence of having lower pile inclination is discussed in section 7.

6 Remodeling of the As-built Main Jetty

6.1 Structural Parameter for Modeling

Structural analysis on the main jetty is carried out by modeling structure using 3D FEM (Finite Element Method) with a software called Nemetschek SCIA engineer version 14. The geometry of the structure is developed based on the actual geometry of the as-built structure. Loads and load combinations have been applied in the model accordingly, and the results shown in this section are chosen based on the appropriate/relevant condition, not only the worst-case scenario.

As-built Main jetty has 100.5 m long and 17 m wide structure with pile foundation. The representative seabed level is at -26 m+LWS, and the representative piles toe elevation is taken at -59.0 m+LWS. The pile tips are connected to the pile cap / beam at their center lines at a level of approximately +3.00 m+LWS.

The longitudinal and transversal beam are precast concrete elements with a rigid connection to the pile cap/beams. The slab is modeled as a 2D member. The connection between beams and slabs is modeled as a rigid connection, and the beams themselves are modeled as continuous beams.

6 vertical piles and 76 inclined (1:10) piles with 914 mm diameter and 14~16 mm thickness are used in the model. The inclination in the model is based on actual field measurement. The jetty consists of a 300 mm thick concrete slab (consist of 150 mm precast and 150 mm in-situ) supported by grids of concrete beams.

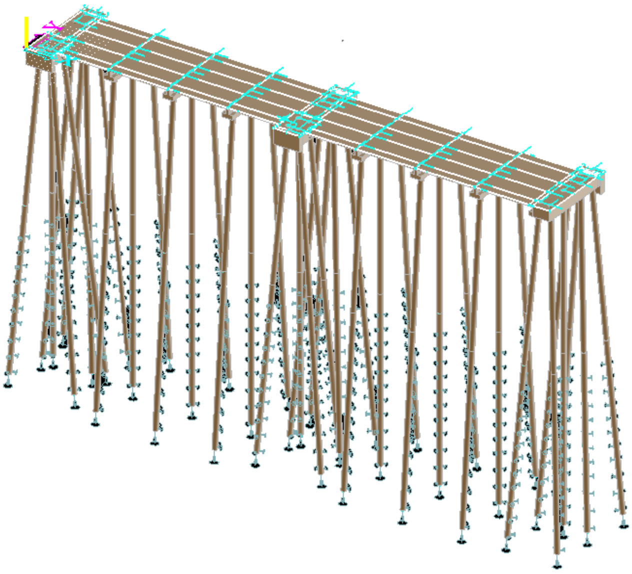

Fig. 5. Structural geometry of back analysis model of the main jetty

6.2 Soil Parameter for Structural Modeling

The spring stiffness method is used to model the soil-structure interaction. The general method, such as the fixity point method, is not likely to be applicable to estimate the true deformation of the structure. Although both methods have their unique limitations, the spring stiffness method is more conservative than the fixity point method; however, more realistic as well since at the plastic limit of the soil due to horizontal loading can be implemented with modification to the linear springs. The horizontal spring stiffness that is determined by using the Menard-Brinch Hansen method (Geodelft, 2004), will provide a linear response and the plastic limit of the soil. Besides that, the vertical spring is based on API RP 2A (American Petroleum Institute, 2010). Soil horizontal and vertical stiffness is presented in **Table 2** and **Table 3**, respectively.

In the structural model, the interaction of pile and soil is taken into account two different soil stiffness values to get the worse effect of the structure. Two models are developed to determine the force distribution in the structure. These models are distinguished by the value of soil lateral stiffness (k_H and k_V, horizontal and vertical stiffness, respectively) as follow:

- The higher value of soil lateral stiffness is used to introduce maximum internal forces to the pile;
- the lower value of soil lateral stiffness is used to get maximum deformation.

Table 2. Horizontal spring stiffness for back analysis

Soil type	Top	Bot.	k_{hor};Menard	
			Low	High
	[m+LWS]	[m+LWS]	[kN/m^3]	[kN/m^3]
Top soil: Clay/Silt medium stiff to stiff, occasionally sandy*	-4	-4	3000	5600
Clay/Silt medium stiff to stiff	-4	-8	3000	5600
Intermediate sand/gravel medium dense	-8	-16	13000	45000
Clay/Silt medium stiff to stiff	-16	-18	3000	12000
Sand lense - medium dense	-18	-33	12000	30000
Clay/Silt very stiff	-33	-67	7000	12000
Clay hard	-67	-90	12000	17000

Table 3. Vertical spring stiffness for back analysis

Pile type	Pile diameter	Thickness	Vertical Spring Stiffness (k_v)	
	(mm)	(mm)	(MN/m)	
steel pipe pile	914	14-16	lower boundary = 105	upper boundary = 170

7 Back Analysis Result

7.1 Discussion on Jetty Structure's stiffness

Structural element resists lateral loads subject to horizontal displacement and deforms within their elastic limit. The lateral motion of the jetty structure by external excitation (e.g., mooring, berthing force, and ground motion during an earthquake) generates initial displacement than released and permitted to vibrate freely. The structure will vibrate back and forth about its initial equilibrium position. The displacement occurs with differentiation of acceleration and velocity with regards to time.

Based on BS 6439 Part 1-4 (ref.[3]), jetty structure in this project shall be classified as a rigid structure since all the horizontal loadings are carrying by mainly as direct compression or tension action of the piles. Rigid structure by this definition shall have high stiffness, thus has a high natural frequency and should not likely experience large amplitude deflections due to dynamic loadings.

Dynamic analysis has been performed by using a three-dimensional FEM model structure. The dynamic analysis result from the back analysis is compared with the result obtained from the vibration field test. The comparison results are presented in **Table 4**.

Table 4. Comparison of natural frequency obtained from initial design, field measurement, and back analysis result.

Jetty structure	Back Analysis		Field Measurement	Initial Design
	Low	High		
Main jetty	0.31	0.32	0.4	0.66

The natural frequency f_n represents the stiffness of the structure — the lower of f_n, the lower the structural stiffness. From **Table 4**, the natural frequency of the main jetty from the analysis is likely in the range of field measurement results. The frequencies obtained from the analysis between low and high stiffness soil have no big difference (0.31 vs. 0.32) since it only checks the superstructure stiffness itself. The "zero displacement level" on the structural model is likely at the same level for both soil stiffness.

The higher natural frequency in the initial design means that the structure is more rigid in theory, but not in reality. This issue can be understood since the initial structure is modeled and analyzed based on the ideal geometry as designated (e.g., correct pile raking 1:5, with the ideal free length of piles). The structure was initially designed using the fixity point method, in which the pile zero displacement level is very shallow; hence, it is very optimistic. In the actual situation, many piles are not raking as designated and have longer free length due to deeper eroded seabed. Besides the superstructure structural rigidity, the interaction between the structure and the ground is also important. In this project, the weak ground condition will support the superstructure badly, resulting in a deeper actual zero displacement level. This can be explained that soil behavior plays an important role in determining the zero-displacement level (fixity point) in the actual condition. Under a certain load, the soil will still behave elastically until a specific load has been reached that makes the soil to enter the plastic state. Once the soil enters a plastic state, theoretically, the strength contribution will be given by the passive soil pressure of the soil itself (Geodelft, 2004). After the load relief, in short term condition, the soil condition will not revert back to the initial condition due to the excessive strains occurred. In other words, the pile loses the grips on the plastic zone. In long term condition, the erosion occurred on the seabed will increase the normative bending moment since the free length of the pile is longer, the actual fixity point will be shifted downward resulting in a deeper point than before. This condition is simply illustrated in **Fig. 6**.

These multiple issues affect the global stiffness of the structure. The structure will deform relatively high even with a small load (e.g., <10 kN), and if it is repetitive loads, such as wave and current, swaying effect may occur.

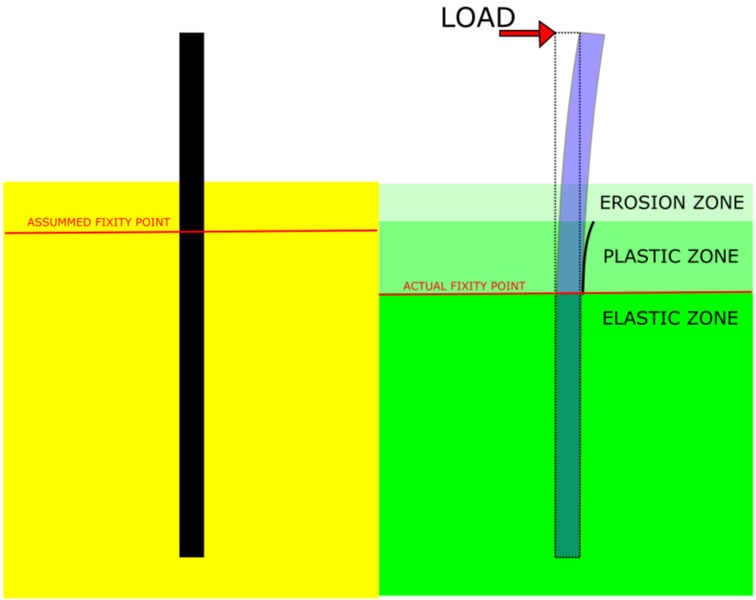

Fig. 6. Actual deeper fixity point resulting in higher displacement and bending moment

A swaying or vibrating structure due to dynamic loads does not mean that the structure is unsafe. Furthermore, the aim of this investigation is to answer whether the jetty can be operational due to the berthing or mooring vessels. The operational condition or the service condition can be predicted by doing an additional structural analysis using the current structure strength/condition. A close value of natural frequency parameter between back analysis model and field measurement; 0.31/0.32 and 0.4 respectively brings a good confidence level to conduct further predictive analysis.

7.2 Predicted Displacement Due to 50,000 DWT Berthing and Pile Stress Ratio on the Main Jetty

The maximum displacement in-service limit state (SLS) of structure for two chosen load combinations based on BS-6349 (ref.[3]) in the transversal direction is presented in **Table 5**. The first scenario is due to environmental loading included with self-weight with superimposed dead load and live load on the main jetty. The second scenario is similar to scenario one but included with berthing load. Both of two scenarios given are not taken in to account seismic forces. The allowable deflection (δ) is set to 100 mm.

Table 5. Structure deflection result of back analysis and initial design under SLS condition

Load combination	Back Analysis (low soil stiffness)	Initial design
	(mm)	(mm)
DL +LL+Environmental load	lower than 20 mm	lower than 15 mm
DL+LL+Environmental load + berthing load	more than 200 mm	lower than 25 mm

Based on two scenarios given, the deflection criteria in SLS condition with berthing load is unsatisfied, and therefore mitigation measures such as additional jetty reinforcement need to be planned if needed. The deflection result of more than >200 mm under SLS condition occurred in transversal (Y) direction parallel with the berthing line, is illustrated in **Fig. 7**.

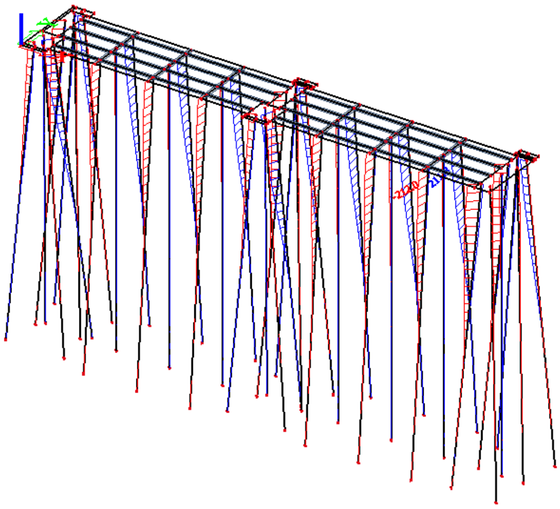

Fig. 7. Maximum deflection in Y direction under SLS condition with berthing load

For jetty structure, having some raking piles are essential for most cases or projects. Raking piles will be designed to have a certain inclination angle, and the inclined amount is up to the Designer. The consequence of having a steeper actual inclination than the design is that the bending moment of the pile tends to be higher. The lateral loads resist by the rake piles primarily in axial compression and tension action. With steeper inclination or near-vertical pile, a high bending moment is induced due to the inability of the pile to transfer the lateral load into compression and tension action. The piles tend to absorb the lateral load by its bending moment capacity, and at the same time, the internal forces are being transferred to the ground due to soil-structure interaction. If the bending moment is high, then the lateral displacement will be high as well. In this jetty case, the displacement obtained from structural modeling is found to be excessive.

Excessive displacement produced by a high bending moment will result in overstressed pile conditions. From structural modeling, the pile stress ratio is ranging from 0.8 to 1.3 maximum (**Fig. 8**). According to Eurocode 3 (ref. [8]), the maximum allowable pile stress is 1, and obviously, some piles are in overstressed condition. The overstress ratio of 1 means that the occurred stress is equal to the yield of steel. In short-term condition, allowing an overstress ratio to equal to 1 is fine. However, in the long-term situation, this is unfavorable since the steel thickness will decrease over time due to corrosion; and hence, corrosion thickness allowance needs to be taken into account to accommodate the future stress induced on the pile.

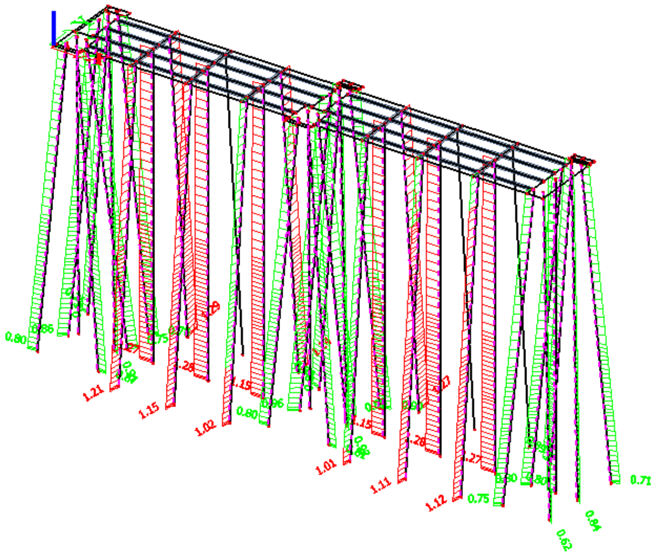

Fig. 8. Pile overstress ratio in SLS condition with berthing load

8 Conclusion

Initial borehole used for design is not representative data for the Jetty due in no small distance (e.g., BH-1 with 90 m). In the waterfront area, especially Dumai, the inhomogeneity of soils are high, and designers shall carefully estimate the representative soil profile for design boundary condition (Nugroho et al., 2019).

In this location, the seabed floor is also still being eroded over time, from 2013 to 2016, and the seabed elevation is reduced about up to 2 m. This reduction of topsoil will negatively impact the overall structure stiffness since the piles' free length increases. Also, most of the installed piles in this project were driven with steeper inclination than as designated. Rake piles have a better carrying capacity of lateral loads than a vertical one, particularly when the lateral loads are high (e.g., berthing vessel load) included with large free, unsupported length. Accumulating all the issues that are found in the actual condition, these conditions drastically reduce the stiffness of the structure, and thus introduced significant horizontal displacement even with small loads. If the loads are occurring periodically, such as waves and current, the jetty may sway.

In most jetty design cases, the fixity point method will be used as a method to determine the pile zero displacement level. However, using this method at a deep seabed level (e.g., 20 m or more) with weak ground strength (e.g., average NSPT $\leq$ 8) and highly erodible seabed are not practically recommended unless high safety factor in fixity point level is applied. Designers shall be aware that the fixity point method is not taking into account the fact that the soil will deform plastically at a particular load. Once the soils are deformed plastically, the piles' free length will increase; thus, the bending moment will be higher. Assumption of the future seabed level due to seabed erosion is also needed during the design process, in order to accommodate the increases of the bending moment due to deeper seabed. This assumption is similar to sheet pile design practice based on CUR166 (ref. [6]), whereas a morphological change due to hydraulic conditions needs to be taken into account such as lowering the passive side ground of the sheet pile.

From the remodeling result of the existing structure, the 50,000 DWT berthing vessel will induce more than 200 mm displacement to the main jetty. This displacement is exceeded the displacement criteria of 100 mm. A reinforcement effort of the existing structure can be implemented in order to make the jetty operational. The reinforcement can be carried out, especially on the right side of the main jetty, whereas space is available to conduct additional pilling works to create the new berthing structures.

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