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# Seismic performance evaluation of a pile-supported pier in Aceh, Indonesia

**Y N Hanifah[1,2], A Budipriyanto[1,2] and I P Rahardjo[1]**
[1]The Department of Civil Infrastructure Engineering, Institut Teknologi Sepuluh Nopember, Kampus ITS Surabaya 60116, Indonesia.

[2]E-mail: yudnina13@mhs.ce.its.ac.id, agungbp@ce.its.ac.id

**Abstract.** The seismic performance of shore structures located in regions having high seismicity is still attracted the interest of researchers. Earthquakes could bring about serious damages on the structures which might lead to failure. Thus the structures' performance under the ground motions need to assessed. This paper describes results of study on performance of a pile supported pier which was constructed in Aceh, Indonesia under seismic motions. In this study the performance evaluation of the pier was conducted using displacement based method of ASCE 61-14 code. 3D numerical models of the pier structure were developed utilizing a structural analysis software package. To account for soil-structure interaction under simulated seismic loads, the nonlinear p-y spring model recommended by American Petroleum Institute was applied. Nonlinear static pushover analysis was performed to obtain structure's capacity. Respons spectrum analysis and nonlinear static demand analysis were carried out to obtain the structure's demand capacity. Results of this study showed that under the 475 and 950-year return periods of earthquakes, the pile-supported pier could be categorized as Minimal Damage. Nevertheless, when it was under 2475-year earthquake return period its performance could be categorized as Control and Repairable Damage; These met the requirements stated in ASCE 61-14.

## 1. Introduction

Seismic performance of shore structures situated in high seismicity regions is still attracted the interest of researchers. Earthquake events could cause important shore structures, such as pier and wharf structures, suffer serious damage that might lead to failure. Therefore, the behavior of the structures under the ground shakings need to be studied and their performances can be assessed. Methods for assessing quay walls and pile supported wharves under seismic loads were explained in [1]. The assessment was performed according to analysis methods recommended by Technical Standard for the results were presented in [2]. Eight specimen of connections widely used in practice were tested in the laboratory to investigate their performances under simulated seismic loads. They concluded that these types of connections could resist large cyclic loads and non-elastic deformations. However, there were substantial stiffness and resistance reductions in them. In addition, due to axial load on pile, moment capacity of the connections increased but it caused the resistance deteriorated more rapidly.

Mondal and Rei [3] reported damages happened on pile-supported harbor structures in Andaman Islands due to the 2006 Sumatra earthquake. They noted that mayor damages of the structure were

[2] To whom any correspondence should be addressed.

because of pounding between two segments of the structure, short pile effects on piles deployed on slope near shore, improper design and poor maintenance which led to corrosion of steel reinforcement and concrete cover spalling, and poor performance of some the structures under the earthquake. Tests on piles on rock fill under lateral loads were performed in [4] to study the relations of the piles' resistance and deformation when the soil particle size was large and might be comparable to the pile diameter. This would be typical open type wharf structure constructed in rock fill dike. The author proposed a curve of soil resistance and deformation, viz. p-y curve where the interlocking between rock particles was taken into consideration. Brunet and de la Llera [5] reported successful application of seismic isolation in pile-supported pier at Coronel port in Chile. The pier was in operational condition after an earthquake while other ports close to the pier were damaged significantly. Donahue et al. [6] conducted numerical analysis of a pile supported wharf using recorded strong motion data. The data were obtained from measurements of acceleration responses of the wharf during the M7.0 Loma Prieta earthquake. 3D numerical models of the wharf were utilized to investigate its responses during the earthquake. The model responses under the earthquake loads were compared with the measured responses. They pointed out that one of factors influencing the structure's model accuracy was the model representing pile-soil interaction. Doran et al. [7] presented evaluation of seismic performance of two pile supported wharves using Pushover analyses according to Turkish code. The results of the study concluded that these two wharves were met minimum requirement of the code.

This study was aimed at evaluating seismic performance of a pile-supported pier located in the high seismically regions in the province of Aceh, Indonesia, see the location in figure 1. The pier was designed for serving loading and unloading bulk coal of 10,000 DWT ship and Portland cement of ship ranging from 20,000 to 40,000 DWT. The structure was modelled numerically using a computer software package; effects of soil liquefaction and lateral spreading were not taken into consideration in this study. Nonlinear Pushover analysis was conducted for assessing the structure under earthquake loads using procedures recommended by the ASCE code for wharf and pier design.

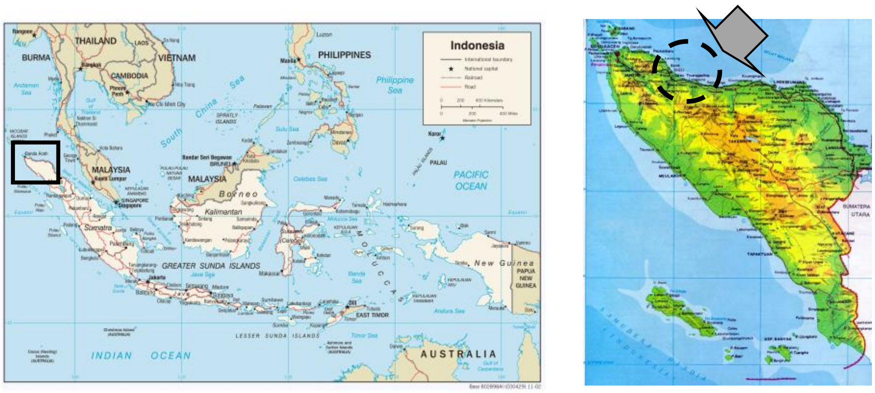


**Figure 1.** The pier location (http://www.maps-of-the-world.net).

## 2. Method

As stated earlier the pier evaluated for this study is located in high seismicity regions. It was designed for berthing of bulk carriers, especially for coal and cement. This important structure has to be in operational condition after being subjected to designed earthquakes or at least damages occurred in the structure can be repaired shortly. In the following sections steps conducted in this study are described.

### *2.1. Data collecting*

The piled-supported pier has dimension of 102m in length and 25m in width. It was constructed on steel pipe piles with the diameters of 1.0m and 0.8m. The steel pile had yield strength of 310 MPa. The pier deck was concrete having compressive strength of 29.05 MPa and rebar yield strength of 400 MPa. Figures 2, 3, and 4 show the plan view, longitudinal and transverse cross sections of the pier.

The connection of pile and pile-cap was composite material of steel pipe and reinforced concrete; It spanned from the pile cap to about -1.0 m below the low seawater level. Reinforcement details of the connection are presented in figures 5 and 6. Available geotechnical data on site was standard penetration test (SPT); the soil profile below the sea bed in term of SPT values is presented in table 1.

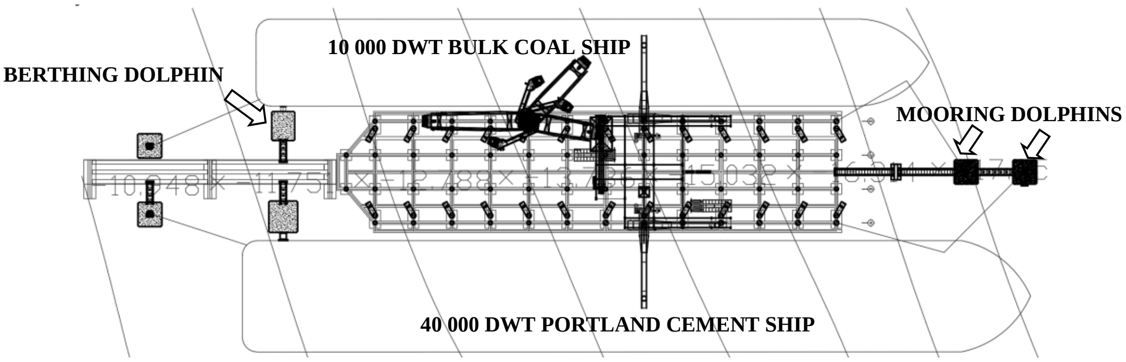


**Figure 2.** Plan view of the pier.

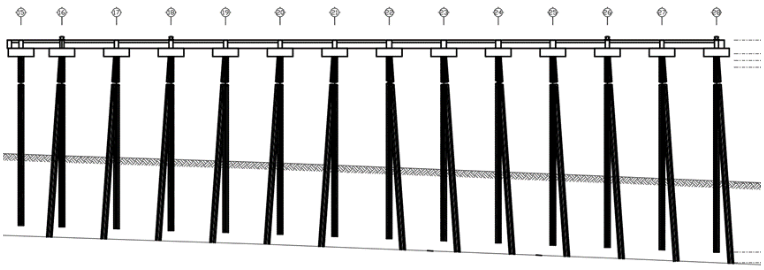

**Figure 3.** Longitudinal cross section of the pier structure.

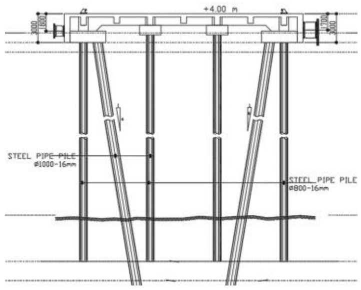

**Figure 4.** Transverse cross section of the pier structure.

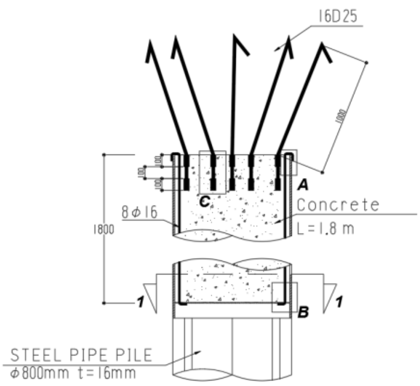


**Figure 5.** Detail of pile to pile-cap connection.

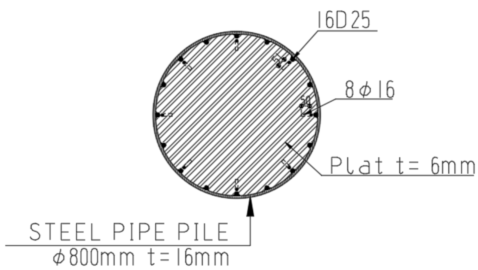


**Figure 6.** Cross cection of pile to pile-cap connection.

**Table 1.** Typical standard penetration test (SPT) values at the pier location.

| Depth below the seabed (m) | 2 | 4 | 6 | 8 | 10 | ≥12 |
|---|---|---|---|---|---|---|
| N SPT Value | 17 | 18 | 26 | 35 | 45 | 60 |

*2.2. Structural modelling*
Three-dimensional models of the pier were developed using SAP2000 structural analysis software [8]. Soil-structure interaction was modelled using nonlinear p-y spring method recommended by American Petroleum Institute [9]. Figures 7 and 8 show curves of soil resistance as function of deflection for the piles of the pier at different depths below the seabed. Lower-bound and upper bound spring values were computed to take into account uncertainties of the soil characteristics and soil variability [10]. In this study response spectra were obtained using two different codes, i.e., RSNI 2833-201X [11], the Indonesia's bridge seismic design code, and ASCE 7-05 [12], the code for minimum load recommended for building and other structure. These spectra for 475, 950, and 2475-year earthquake return periods are presented in figures 9 and 10.

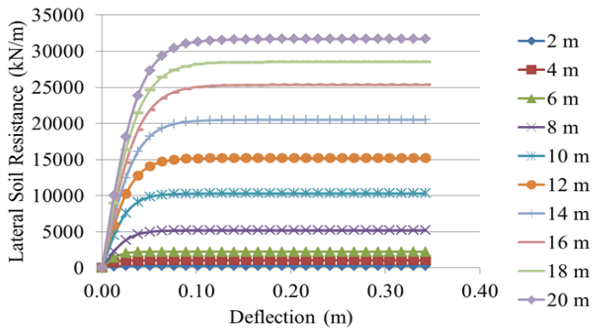


**Figure 7.** p-y curves of 0.8m-diameter pile at different soil depths below the seabed.

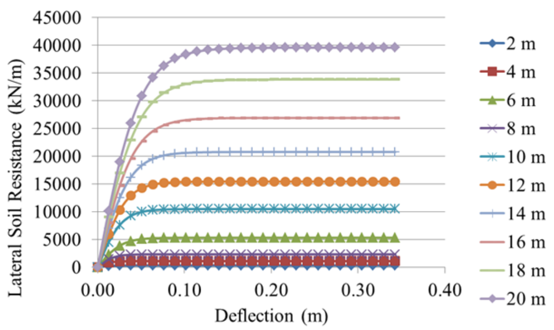


**Figure 8.** p-y curves of 1m-diameter pile at different soil depths below the seabed.

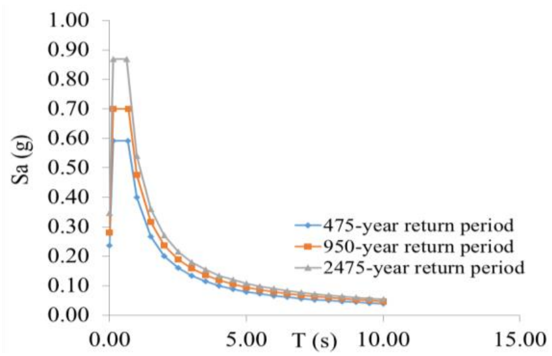


**Figure 9.** Response spectra for 475, 950, 2475–year return periods using RSNI 2833-201X.

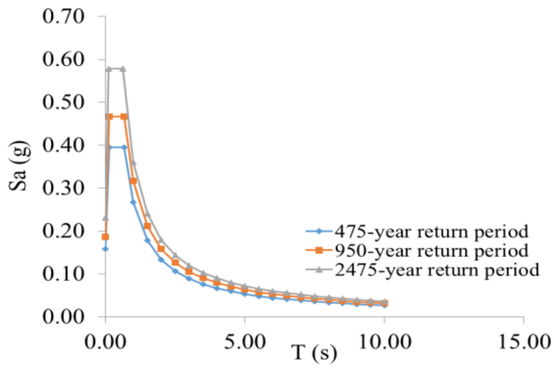


**Figure 10.** Response spectra for 475, 950, 2475–year return periods using ASCE 7-05.

2.3. *Structural capacity analysis*
Nonlinear Static Pushover analysis were performed to obtain structural capacity under the seismic loads. The analysis was conducted following steps described below,

*2.3.1. Expected material strength.* Expected material properties were determined to assess Structural capacity and Demands under the earthquake loads. The expected strength for each material was defined according to ASCE 61-14 [13].

*2.3.2. Plastic hinge and strain limit.* In many cases plastic hinges were formed at (a) connection of pile to pile-cap, and (b) pile beneath the seabed where the large bending moment occurs. To define the

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plastic hinge properties, in this study cross section analyses were conducted to obtain the moment curvatures of plastic hinge at the connection of pile to pile-cap and the one below the seabed.

In the connection of pile to pile-cap, the steel pile was considered to be embedded; it implied that it was a full-moment connection. ASCE 61-14 [13] permits Method B for moment-curvature analysis for this type of connection. The allowable strain limits for different performance levels of pier structure are presented in table 2 [13].

**Table 2.** Strain limits for different performance levels [13].

| Performance level | Strain limit | |
|---|---|---|
| | Top of pile | In ground |
| Minimal Damage | $\varepsilon_s \leq 0.010$ | $\varepsilon_s \leq 0.010$ |
| Controlled and Repairable Damage | $\varepsilon_s \leq 0.035$ | $\varepsilon_s \leq 0.025$ |
| Life Safety Protection | $\varepsilon_s \leq 0.050$ | $\varepsilon_s \leq 0.035$ |

*2.3.3. Loads applied on the structure.* During the earthquake, the loads applied on the pier were the structure self-weight, weight of equipment that was attached on the structure constantly, and 10% of the live load. The design loads for the pier were computed according to POLB [10] and AASTHO LRFD [14]. The seismic mass of the pier considered in this study were therefore made up of mass of these loads.

*2.3.4. Lateral load distribution pattern.* To model lateral load pattern during earthquake events, load patterns used in this study were uniform acceleration and modal patterns. In uniform acceleration pattern, the force pattern was assigned proportional to the mass distribution in the corresponding direction. Whereas in modal pattern, modal analysis results were to be assigned to the corresponding mode load case; In this study, the first two modes of the structure were considered in the analysis. Figures 11(a) and (b) depict the deformed shape of longitudinal and transverse modes.

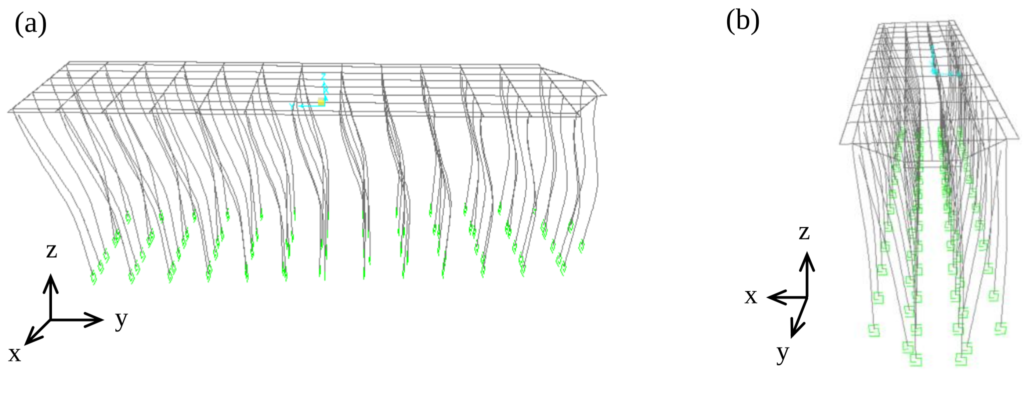


**Figure 11.** Deformed shape of (a) longitudinal mode (in the-y axis direction), and (b) transverse mode (in the-x axis direction).

## 2.4. *Demand capacity analysis*

Response spectrum analysis and nonlinear static demand analysis were used to obtain demand capacity. These analyses were performed by applying the procedure described in ASCE 61-14 [13].

## 3. Results and Discussion

Results of this study are reported and discussed in the sections below.

### *3.1. Capacity curve*

Capacity curve represents the function of base shear and displacement of the pier deck. In this study capacity curves of the pier were obtained by applying Nonlinear Static Pushover analysis under load applied in the transverse and longitudinal directions. Figures 12(a) and (b) present capacity curves of the pier structure obtained using upper bound and lower bound spring models for two lateral load distribution patterns, i.e., uniform acceleration and modal load distribution patterns. It can be seen from these figures that for the same deformation values, the base shear forces obtained by the uniform acceleration pattern were higher than those computed by the modal pattern. In addition, the load applied on the tranverse direction resulted in larger displacements on the structure for the same base shear. Plastic hinges developed under loads applied during the Nonlinear Static Pushover analysis could also be monitored. Figures 13(a) and (b) show the first yield plastic hinge occurred in the pier structure for loads applied in the transverse (the–x axis) and the longitudinal (the–y axis) directions using lower bound spring model. The results for upper bound spring model was similar therefore it is not presented in this paper.

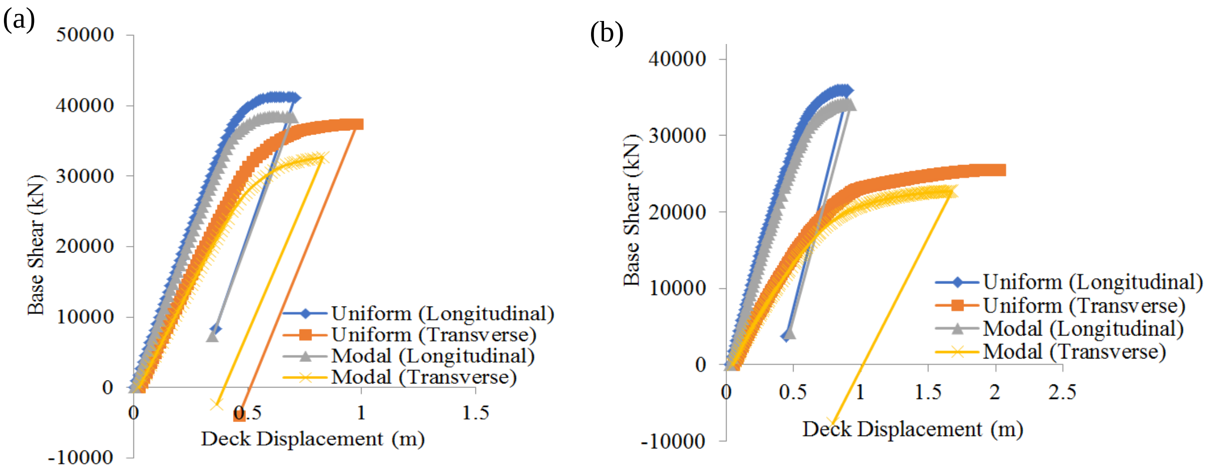


**Figure 12.** Capacity curves of pier obtained using (a) upper bound, and (b) lower bound spring models in the transverse and longitudinal directions by applying uniform acceleration load pattern (uniform) and modal load pattern (modal).

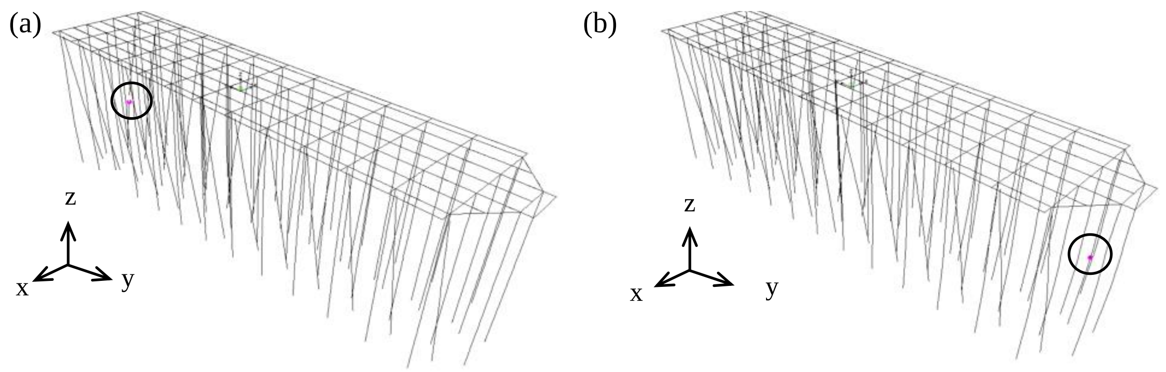


**Figure 13.** First yield-hinge under the load applied in the (a) transverse (the –x axis) , and (b) longitudinal (the –y axis) directions of the pier using lower bound spring model.

### *3.2. Seismic demand*

To obtain the seismic demand, first the respons spectrum analysis was conducted to investigate wether the structure was still in its elastic state under the ground motions or not. Then when the structure behaved beyond its elastic limit, the nonlinear static demand analysis was performed. The nonlinear static demand analysis was conducted by using capacity spectrum approach available in SAP2000; numbers of iterations might be needed to satisfy the conditions required in ASCE 61-14 [13].

### *3.3. Performance levels of the structure*

The pier was subjected to 475, 950, and 2475-year earthquake return periods calculated using procedures recommended by RSNI 2833-201X [11] and ASCE 7-05 [12]. The minimum performance level of the pier required the ASCE 61-14 [13] was Controlled and Repairable Damage under 475-year return period of ground motion and Life Protection under 950 and 2475-year return period of ground motions. When the response spectra of the ground motions having 475 and 950 year return periods were computed using procedures described in [11] and [12] it was observed that performance of the pier structure could be categorized as Minimal Damage. Pier's performance level under 2475-year earthquake return period was also Minimal Damage when the spectrum was calculated using procedure in [12]. Minimal Damage can be desribed as the damage that can be identified with initial cracking and spalling of the pile and/or pile cap [13]. Tables 3 and 4 present the performance levels of the pier under 2475-year return period of the ground motion in the transverse and longitudinal directions using two load distribution patterns when the spectrum was obtained using procedure stipulated in [11]. The pier's performance levels under the 2475-year earthquake return period are also

**Table 3**. Performance levels of the pier in the transverse direction under the 2475-year return period seismic ground motion for different lateral load distribution patterns.

| Lateral load pattern | Displacement capacity limit (m) | | | | Seismic Demand (m) | Strain at yield (m/m) | Performance Level |
|---|---|---|---|---|---|---|---|
| | MD [e] | CRD [f] | LSP [g] | Collapse | | | |
| Uniform-UB [a] | 0.4441 | 0.5545 | 0.6079 | 0.6612 | 0.4090 | 0.0081 | MD |
| Uniform-LB [b] | 0.7061 | 1.0081 | 1.1126 | 1.2200 | 0.6450 | 0.0051 | MD |
| Modal-UB [c] | 0.3986 | 0.4936 | 0.5419 | 0.5953 | 0.3960 | 0.0042 | MD |
| Modal-LB [d] | 0.5830 | 0.8740 | 0.9741 | 1.0711 | 0.6750 | 0.0171 | CRD |

[a] Uniform acceleration load pattern with upper bound spring model
[b] Uniform acceleration load pattern with lower bound spring model
[c] Modal load pattern with upper bound spring model
[d] Modal load pattern with lower bound spring model
[e] Minimal Damage
[f] Controlled and Repairable Damage
[g] Life Safety Protection

**Table 4.** Performance levels of the pier in the longitudinal direction under the 2475-year return period seismic ground motion for different lateral load distribution patterns.

| Lateral load pattern | Displacement capacity limit (m) | | | | Seismic Demand (m) | Strain at yield (m/m) | Performance level |
|---|---|---|---|---|---|---|---|
| | MD [e] | CRD [f] | LSP [g] | Collapse | | | |
| Uniform-UB [a] | 0.2986 | 0.4119 | 0.4719 | 0.5392 | 0.2970 | 0.0079 | MD |
| Uniform-LB [b] | 0.3734 | 0.5357 | 0.6135 | 0.6930 | 0.3950 | 0.0139 | CRD |
| Modal-UB [c] | 0.2982 | 0.4112 | 0.4712 | 0.5382 | 0.3050 | 0.0125 | CRD |
| Modal-LB [d] | 0.3727 | 0.5329 | 0.6077 | 0.6930 | 0.4020 | 0.0156 | CRD |

[a] Uniform acceleration load pattern with upper bound spring model
[b] Uniform acceleration load pattern with lower bound spring model
[c] Modal load pattern with upper bound spring model
[d] Modal load pattern with lower bound spring model
[e] Minimal Damage
[f] Controlled and Repairable Damage
[g] Life Safety Protection

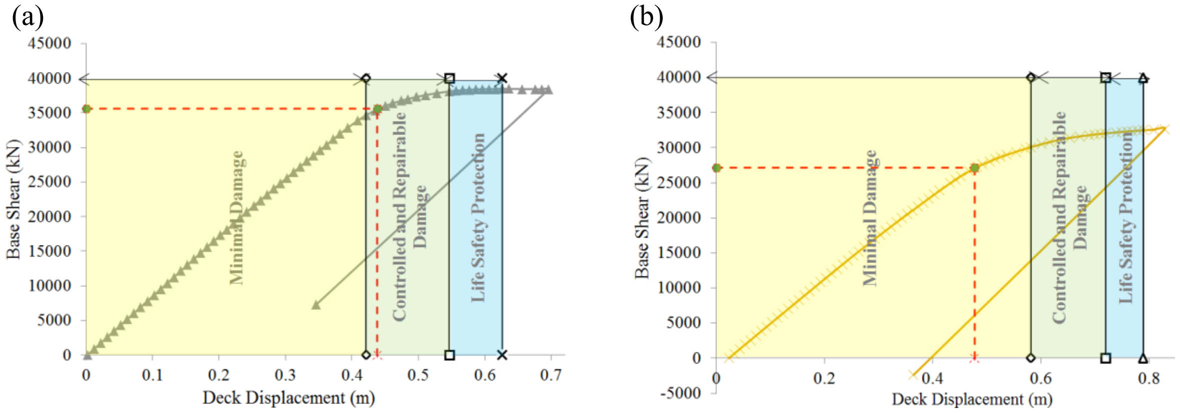


**Figure 14.** Performace levels of the pile-supported pier under the 2475-year earthquake return period obtained using upper bound spring model for modal load pattern applied in the (a) longitudinal (the-y axis), and (b) transverse (the-x axis) directions.

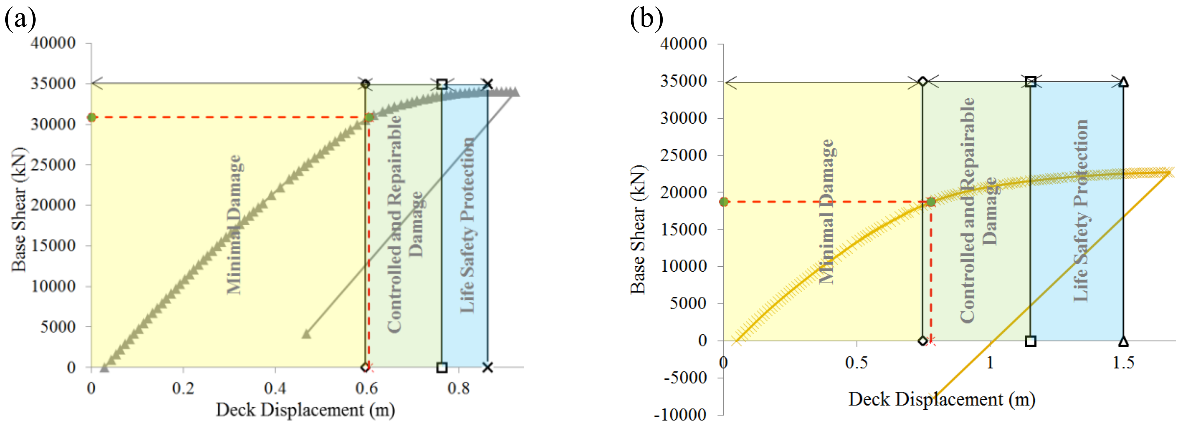


**Figure 15.** Performace levels of the pile-supported pier under the 2475-year earthquake return period obtained using lower bound spring model for modal load pattern applied in (a) the longitudinal (the-y axis), and (b) the transverse (the-x axis) directions.

presented in figures 14 and 15 for clarity; Modal load pattern was applied for obtaning the performance levels presented in these figures. The performance level of the pier structure under the earthquake was Controlled and Repairable Damage. This type of damage could be indicated by exposure of reinforcement bar of the deck due to considerable spalling occurred in the deck particularly at the region adjacent the pile. Thus the performance levels of the pile-supported pier met the requirement stated in the code [13].

## 4. Conclusion

In this study, a pile-supported pier structure located in Aceh - a high seismicity region in Indonesia – was evaluated for its seismic performance. 3D models of the pier were developed using SAP2000 structural analysis software. To account for soil-pile interaction, nonlinear spring model of API [9] was applied. The structural capacity was obtained using nonlinear static Pushover analysis and demand capacity was computed using respons spectrum analysis and nonlinear static demand analysis described in [13]. The pile-supported pier was subjected to 475, 950, and 2475-year return periods of earthquakes which spectra were calculated using two different methods, viz., RSNI 2833-201X [11] and ASCE 7-05 [12].

IOP Conf. Series: Materials Science and Engineering **267** (2017) 012022 doi:10.1088/1757-899X/267/1/012022

Using response spectrum analysis along with nonlinear static demand analysis, it was observed that under the 475 and 950-year earthquake return periods the structure's performance could be categorized as Minimal Damage. However, under 2475-year earthquake return period which spectrum computed using procedure in [11], the pier's performance could be categorized as Controlled and Repairable Damage. Therefore the seismic performance level of the pile-supported pier structure met the requirement stated in the ASCE 61-14.

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