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Earthquake Vulnerability of Port Structures in Indonesia

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Earthquake Vulnerability of Port Structures in Indonesia

H Widyastuti¹, E Lumantarna², M Sofi², M I Ramli³, Y. Oktavianus² and A Rajabifard²

¹Civil Engineering Department, Institut Teknologi Sepuluh Nopember, Surabaya, Indonesia

²Department of Infrastructure Engineering, The University of Melbourne, Melbourne, Australia

³Department of Civil Engineering, Hasanuddin University, Makassar, Indonesia

E-mail: hera.widyastuti@yahoo.co.uk

Abstract. Indonesia is located in a high seismic region and is in one of the most vulnerable countries likely to experience earthquakes. The impact of earthquakes on port structures can have an enormous impact on the economy if the earthquake hazard is not acknowledged, essential elements of the transportation system not identified, and damage prevention procedures not applied. In this project, the seismic performance of critical infrastructures, such as port structures as designed and constructed, are assessed. Outcomes of the assessment enable vulnerable elements to be identified leading to design recommendations. The project is conducted based on data collection, field survey, site investigations, experiments, and computer modelling and simulations.

3

1. Introduction

Seaports are very important infrastructures for local and international transportation networks and play a vital role in the economic activity of a nation. In most countries, international trade through sea transportation is the most common mode compared to other modes such as trade through land and air. Hence, their reliability and effectiveness are crucial for the regional and national economy. Approximately 90% of Indonesia's international traded goods are distributed using sea transportation.

The downtime of seaports due to natural disasters such as earthquakes can result in significant economic loss. For example, the 1993 Koshiro-Oki earthquake resulted in significant economic loss, with a significant proportion of the loss estimated for port-dependent industries [1]. The 2004 Banda Aceh earthquake and tsunami has significant impact due to damage to the infrastructure and facilities supporting the fisheries sector [2]. A significant economic loss can occur despite minor physical damage suffered by port structures. Damage to transportation systems, especially ports, require much longer time-frames for restoration [3]. Damage to ports and disruption to transportation accessibility have been reported to be the main contributor to long-term economic loss in the 1995 Kobe earthquake. Often, the damage suffered by ports inhibits the delivery of relief supplies to the affected area as experienced in Haiti and Andaman Islands [4, 5].

Experience has demonstrated the possible devastation of earthquakes on seaports. For example, the magnitude 7 Haiti earthquake resulted in the collapse of the North Wharf of Port de Port-au-Prince. The magnitude 7.2 Kobe earthquake caused collapse of hundreds of quay walls in Port Kobe. More modest magnitude earthquakes have also been reported to have caused damage to ports (e.g., the magnitude of 6.4 Lefkada earthquake in Greece, the magnitude of 6.3 Christchurch earthquake in New Zealand).



2
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Poor foundation and backfill soil, which are common to waterfronts environment and liquefaction phenomena in the loose saturated sand beneath the port structures have been reported to be the common reasons for poor seismic performance of ports. Poor foundation soil has been found to have caused damage to piles supporting the wharf structures in the Kobe earthquake [6] and the Haiti earthquake. Beside poor soil conditions, damage to port structures due to inadequate design and poor maintenance are equally common to seaports. Inadequate shear reinforcement, improper detailing, and corrosion of transverse ties have contributed to the damage of piles supporting the port structures in Andaman Islands during the 20²⁷ Sumatra earthquake.

Indonesia is one of the most seismically active regions where the tectonic boundary stretches from Sumatra in the east and West Papua in the west. Within the past decade alone, close to 20 above seven magnitude earthquakes have occurred in the country. In light of recent earthquake activities, the seismic hazard map of Indonesia has been updated, resulting in an increase in seismic hazard values [7, 26].

Because of the increase in the seismic hazard values and the significance of seaports, there is a need to assess the seismic performance of port structures. This paper presents seismic vulnerability assessment of a bridge structure, connecting the berth and mainland, at Terminal Peti Kemas, Surabaya. Fragility curves have been constructed for typical steel girders in the bridge structures based on non-linear time-history analysis. The details of the structure and finite element modelling approach are presented in Section 2. The selection of ground motion inputs for the analyses is presented in Section 3. The development of the fragility curves for the typical piles are presented in Section 4.

2. Structural details and modelling approach

An RC bridge at Terminal Peti Kemas (TPS) in the seaport of Tanjung Perak was selected for a case study. Figure 1 shows the RC bridge in TPS viewed from the domestic berth. The bridge has a total number of the span of 100, and each span has a length of 15 meters [9]. Each span is constructed out of nine RC T-beams that are supported by an RC cross beam. The crossbeam is supported by three steel piles that are wrapped with a corrosion protection coating layer. Figure 2 shows the typical cross-section of the bridge (adapted from [10]). The concrete has a design strength of 40 MPa, and the steel bars have yield strength of 400 MPa. The steel piles were made up of the circular hollow section with an outer diameter of 711 mm and a thickness of 12 mm. The steel pile has a grade of 50C following BS 4360 [11]. It corresponds to yield and ultimate strength of 350 MPa and 500 MPa, respectively.



Figure 1. RC bridge investigated.

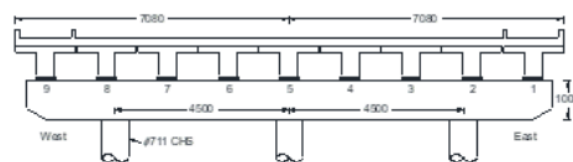


Figure 2. Cross-section of the RC bridge.

33
10 non-linear time-history analyses were conducted using program ETABS v13.2.2 [12]. Only one pier with a group of three steel piles was modelled as shown in figure 3. The T-beam was not modelled since it is supported by the cross-beams, and hence the earthquake lateral force will be resisted by the steel piles only. The self-weight of the T-beam (340 kN for one T-beam) was taken into account by taking into account by adding a lumped mass into the system. The cross-beam was assumed to remain elastic, and only the steel piles can be excited into inelastic region under earthquake excitations. Plastic hinges were defined considering P-M2-M3 interaction for steel elements following FEMA 356 [13]. These plastic hinges were applied to both top and bottom of the steel pile.

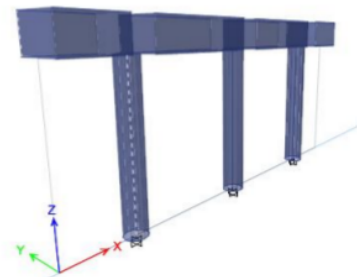


Figure 3. Model of a typical pier in the TPS bridge.

The length of the pile varies depending on the distance to the land. In the analyses, the length of the pile was varied from 4 to 12 m to investigate the effect of varying the length on the seismic performance of the piles. Though the steel piles are wrapped with a corrosion protection coating layer outside the steel tube and site investigation conducted have not detected any sign of deterioration to the piles, three different levels of corrosion were considered, i.e. 0, 10%, and 20%. The corrosion was assumed to occur globally from inside of the tube wall.

3. Selection of ground motion inputs for analyses

Ground motion inputs were obtained by collating ground motion records on rock and use the ground motions on the rock to generate ground motion inputs taking into account the effect of soil sediment. The ground motion on rocks consists of a combination of recorded and generated accelerograms that have peak ground velocity (PGV) that are representative of the seismic hazard of Surabaya. The relationship between the design return period (RP) and the peak ground acceleration on the rock (PGA) under the latest Indonesian seismic hazard map [8] is presented in table 1. The recorded accelerograms on rock (with $V_{s,30} \geq 750$ m/s) were selected from PEER ground motion database [14]. The generated accelerograms on rock were simulated using program GENQKE [15] using the Atkinson's attenuation model [16]. The recorded and generated motions represent earthquake events with magnitude ranging from 6.5 to 9. The PGV of the records is plotted against its associated design RP in figure 4.

The recorded and generated ground motions were applied to various soil layers using program DEEPSOIL [17] to generate ground motion taking into account the effect of soil sediment. The generated ground motions on soil were used for the analyses of the structure. Three sets of soil data (table 2) obtained nearby the bridge was used to generate the ground motion inputs.

Table 1. Relationship between RP and PGV.

RP (years)	PGA (g)	Notional PGV* (mm/sec)
95	0.05 – 0.1	37.5 – 75
195	0.1 – 0.15	75 – 112.5
475	0.15 – 0.2	112.5 – 150
750	0.2 – 0.25	150 – 187.5
1034	0.25 – 0.3	187.5 – 225

RP (years)	PGA (g)	Notional PGV* (mm/sec)
2475	0.3 – 0.4	225 – 300
4950	0.4 – 0.5	300 – 375
9950	0.5 – 0.6	375 – 450
>9950	1.3 – 1.7	975 – 1275

* PGV = PGA × 750

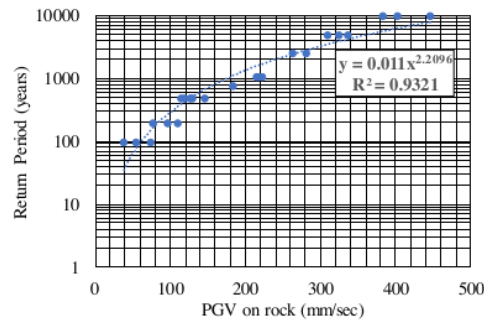


Figure 4. PGV on the rock of the recorded and generated ground motions and the associated RP.

Table 2. Three sets of representative soil layers.

Soil type	Unit weight (kN/m ³)	V _s (m/s)	Depth from top surface (m)		
			S1 (T=2.15s)	S2 (T=1.38s)	S3 (T=2.43s)
Very soft sandy (silty) clay (+shell)	17	75	0 – 28	0 – 17.8	0 – 31.7
Soft silty clay	18	125	28 – 35.3	17.8 – 22.2	-
Soft to firm silty clay	19	150	-	22.2 – 25	-
Firm silty clay	20	175	-	25 – 31.1	-
Medium to stiff silty clay	21	200	35.3 – 40	-	31.7 – 35
Stiff to very stiff silty clay	22	225	40 – 41	31.1 – 34.2	-
Very stiff silty clay	23	250	41 – 60.5	-	35 – 43
Very stiff to hard silty clay	23.5	275	-	-	43 – 80.5
Very dense sandstone	24	350	-	-	-

4. Development of fragility curves

Seismic fragility functions define the building's probability of exceeding a damage limit state as a function of ground motion intensity measure (IM). The fragility function is defined by Equation (1):

$$P[Y > 1 | IM] = \phi \frac{\ln(\eta_{Y|IM})}{\sqrt{\beta_{Y|IM}^2 + \beta_C^2 + \beta_M^2}} \tag{1}$$

where, $\eta_{Y|IM}$ is the median critical demand-to-capacity ratio as a function of IM, $\beta_{Y|IM}$ is the dispersion (logarithmic standard deviation) of the critical demand-to-capacity ratio as a function of IM, β_C is the capacity uncertainty and β_M is the modelling uncertainty. In this paper, the dispersion was assumed to have been caused by the variation in ground motions. Hence, both β_C and β_M were set to zero.

4.1. Ground motion Intensity Measure (IM)

The development of fragility curves involves conditioning the structural response on the ground motion intensity measure (IM). The IM selected shows a strong correlation between the seismic intensity and the structural response to reduce the uncertainty in the seismic assessment. Besides, the IM needs to represent the level of seismic hazard, i.e., it needs to correlate well to earthquake return periods [18]. Traditionally, the IM that has been commonly used for seismic assessment has been peak ground acceleration (PGA). It is the parameter that is typically used to represent hazards on seismic hazard maps, including the latest Indonesian hazard map. However, peak ground velocity (PGV) is considered to provide a better indication of the level of structural damage since it is related to the energy in the ground motion [19, 20]. In this paper, PGV was used as ground motion IM. The PGA values were calculated by dividing the PGV values by a factor of 750 [20]. The PGA values were correlated with the design return period (RP) by the regression equation shown in figure 4.

4.2. Performance levels and demand to capacity ratio

Many different performance levels are defined in the literature and codes (e.g., [21, 22]), each with different acceptance criteria. This study adopts only one performance level, which is a slight damage level (operational limit state). This limit was selected due to the requirement from the stakeholder for the bridge to remain operational after a significant earthquake event. The limit associated with the performance level was set to be equal to the yield capacity of the steel pile.

Figures 5 to 7 present the D/C ratio (Demand to Capacity ratio) for 4 m, 8 m, and 12 m piles, and different soil layers (S1, S2, and S3) plotted against the PGV values on the rock. Only results for 0% corrosion level are presented, although similar trends were observed for other corrosion levels. The D/C values that are larger than 1 indicate that the structure yielded (exceeded the performance level). Regression analyses were conducted to fit each curve to a power-law function. It was performed to obtain a constant standard deviation of the natural logarithm ($\beta_{Y|IM}$ in Equation (1)) [23]. It is shown that in general, the D/C value for Y-direction is higher than that of X-direction. It means that the structure is more flexible and yields higher displacement demand in Y-direction than in X-direction. Soil layer S2 is generally shown to result in higher demand (larger D/C ratio) in comparison with soil layers S1 and S3. It is because soil layer S2 has a site period that is the closest to the fundamental natural period of the piles. The longer piles were also found to be subjected to higher displacement demand (D/C ratio increases with the increase in pile's length).

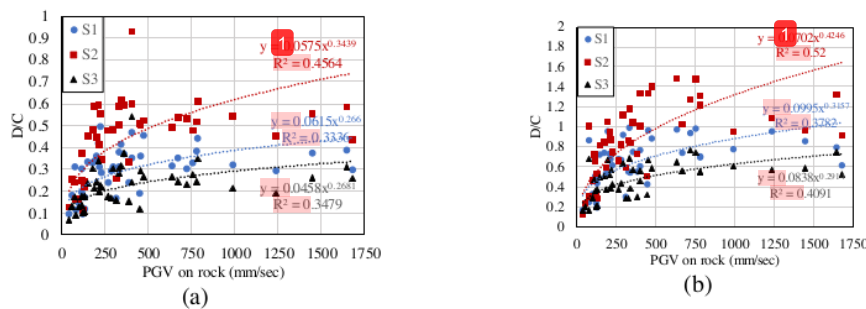


Figure 5. D/C ratio for 4 m length pile and 0% level of corrosion, (a) x-direction, (b) y-direction.

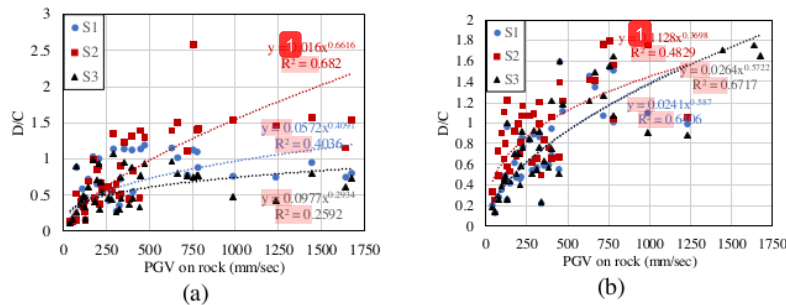


Figure 6. D/C ratio for 8 m length pile and 0% corrosion, (a) x-direction, (b) y-direction.

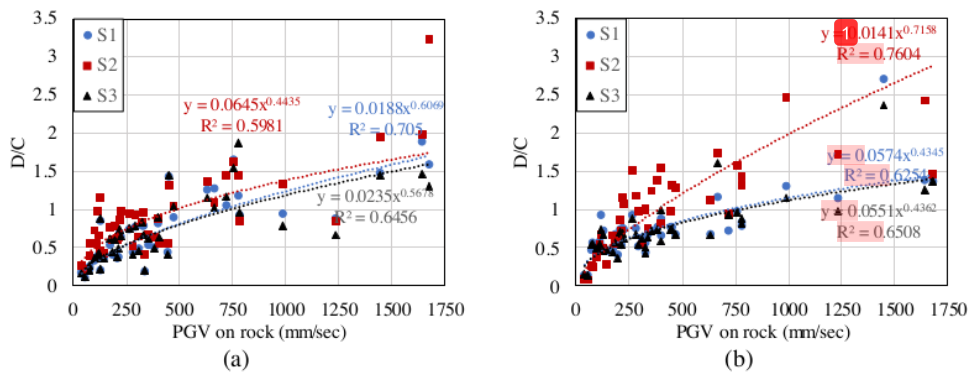


Figure 7. D/C ratio for 12 m length pile and 0% corrosion, (a) x-direction, (b) y-direction.

4.3. Fragility curves

Based on the D/C graphs presented in figures 5 to 7, the fragility curve can be derived using Equation (1). The equation calculates the probability of exceeding D/C (defined as Y in equation (1)) of 1 as a function of ground motion intensity measure (IM). $\beta_{Y/IM}$ Was obtained by calculating the logarithmic standard deviation of the D/C ratio as a function of IM. The fragility curves for 4 m, 8 m, and 12 m pile length are presented in figures 8 to 10.

Figures 8 to 10 indicate that the structure is more likely to fail in the y-direction than in the x-direction. The observed trends are expected as the structure is more flexible in the y-direction than the x-direction resulting in larger displacement demands. A longer pile is shown to be more vulnerable to damage indicating that soil erosion could have detrimental effects on the pile. Further, corrosion on the structure is shown to have resulted in a higher probability of damage, although the effect is generally less significant compared to the effect of other parameters (such as the length of piles and soil layers).

From the fragility curves presented in Figures 8 to 10, the return period of an earthquake event, which results in the damage limit state being exceeded can be estimated. In this paper, the estimation has been conservatively based on a 5% probability of exceedance, i.e., the earthquake event has 5% probability of causing damage to the structure. The PGV value associated with 5% probability of exceedance for each curve was correlated to the return period using the regression equation shown in Figure 4. The return period (RP) values are presented in table 3.

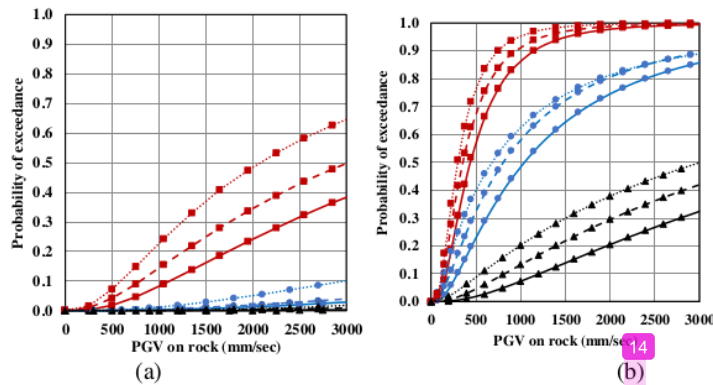


Figure 8. Fragility curve for 4 m length pile, (a) x-direction, (b) y-direction.

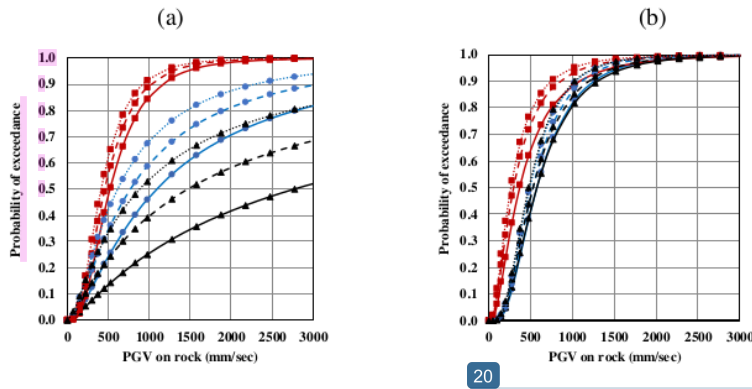


Figure 9. Fragility curves for 8 m length pile, (a) x-direction, (b) y-direction.

Table 3. The RP (in years) of the structures at the probability of exceedance of 5%.

Pile length (m)	Level of corrosion (%)	x-direction			y-direction		
		S1	S2	S3	S1	S2	S3
4	0	1117355	27136	12560322	1456	483	28975
	10	715819	12417	3025648	631	290	11475
	20	218559	6684	1622548	246	229	4289
8	0	984	1129	1584	1456	232	1337
	10	514	850	496	1321	150	1101
	20	375	668	280	1119	121	1030
12	0	2466	613	2007	1766	910	2288
	10	1864	363	1748	1173	844	1526
	20	1569	222	1315	571	749	791

It is shown that the return period is significantly affected by all parameters investigated (i.e., pile length, direction, and soil layers). It was found that structural degradation due to corrosion can increase

19 the vulnerability of the structure in an earthquake. However, 18 the length of the pile was found to have a significant impact on the vulnerability of the structure. The damage limit set for the structure can be exceeded by 500 year return period event (nominally defined as a rare event) and 2500 year return period (nominally defined as a very rare event) for the most onerous case.

29 Conclusion

The seismic performance of the bridge connecting the berth and the mainland at Terminal Peti Kemas Surabaya has been assessed. Fragility curves were constructed based on non-linear time-history analyses using ground motions inputs that are representative of the earthquake excitations expected at the location, taking into account the effects of soft soil layers. Ground motion records on the rock were selected from PEER database and generated using program GENQKE, which were then used as input to generate the ground motion records on top of 5oft soil layers using program DEEPSOIL.

The results from the analysis demonstrate that the vulnerability of the structure is dependent on various parameters. The stiffer soil generally results in higher displacements due to the less energy being absorbed by the soil layers. It has, in turn, caused an increase in vulnerability of the structure. Structural degradation has been shown to increase the vulnerability of the structure. However, the length of the piles has been shown to have the most significant impact on the vulnerability of the structure. It has been found that the damage limit set for the structure can be exceeded by 500 year return period event and 2500 year return period for the most onerous case.

17 Acknowledgements

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