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International Journal of Civil Engineering and Technology (IJCIET)
Volume 13, Issue 1, January 2022, pp. 1-12, Article ID: IJCIET_13_01_001
Available online at https://iaeme.com/Home/issue/IJCIET?Volume=13&Issue=1
Journal Impact Factor (2020): 11.3296 (Calculated by GISI) www.jifactor.com
ISSN Print: 0976-6308 and ISSN Online: 0976-6316
DOI: https://doi.org/10.34218/IJCIET.13.1.2022.001

NUMERICAL STUDY OF STRUCTURAL RELIABILITY OF LONG SPAN BRIDGE UNDER LATERAL LOAD

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ABSTRACT

Indonesia is classified as an earthquake-prone area due to its location above the convergence of active tectonic plates. There are, however, two bridges, Ponulele in Palu and Wu-shi in Taiwan, reported to have collapsed due to an earthquake, thereby, indicating the need to examine their reliability. It is, therefore, possible to conduct the analysis through the use of the Pushover Analysis Method.

This research showed the comparation between ATC-40 and FEMA 356 of the pushover and performance-based evaluation for the initial model that met the design criteria and discovered to be reliable against earthquakes were followed by the analysis of the test model which is a variation of the initial model with a cross-section of pillars and piles and different reinforcement ratio. The main model which have 2,5 m pier diameter and 1 m pile diameter was varied and named with code A-P1-TP1-1-3, A-P2-TP1-2-3, A-P2-TP1-3-3, A-P1-TP2-2-2 and A-P1TP2-2-3.

The result showed that the main model was reliable to earthquake design load, with ratio 1,25. The structural performance level based on ATC-40 was IO while FEMA 356 shows it was on Operation level. As compared to the one, model A-P1-TP1-1-3 showed reliability of 10% less after pier reinforcement ratio is reduced. Model A-P2-TP1-2-3 and A-P2-TP1-3-3 showed reliability of 20% less after reducing the main model's pier diameter, while model A-P1-TP2-2-2 and AP1-TP2-2-3 showed its reliability increased by 3% after adding the pile diameter. Nevertheless, the pier's diameter and reinforcement showed important result to bridge reliability under earthquake load more than the pile's. Plastic hinge mechanism of the models showed that the failure happened on the pier at first. Based on ATC-40, the structural performance level of model which had smaller pier diameter was DC, while others were on IO level. Based on FEMA-356, all of the model performance level was IO.

Keywords: Pushover Analysis; Performance Based; ATC-40, FEMA-356.

Cite this Article: Arini Novia Sari and Nursiah Chairunnisa, Numerical Study of Structural Reliability of Long Span Bridge Under Lateral Load, *International Journal of Civil Engineering and Technology (IJCIET)*, 13(1), 2022, pp. 1-12.
https://iaeme.com/Home/issue/IJCIET?Volume=13&Issue=1

1. INTRODUCTION

Indonesia is classified as an earthquake-prone area due to its location above the tectonic plate movement lane. This means the structures need to be planned to ensure they meet the design criteria for reliability against earthquakes. There are, however, two bridges, Ponulele and Wu-shi, reported to have collapsed due to an earthquake, thereby, indicating the need to examine their reliability. It is, therefore, possible to conduct the analysis through the use of the Pushover Analysis Method which is usually applied by providing and increasing lateral static load up to the moment a structure reaches the target displacement. This process produces a capacity curve to illustrate the relationship between the base shear and roof displacement as well as the behavior of a structure under elastic and plastic conditions up to the period its elements collapse.

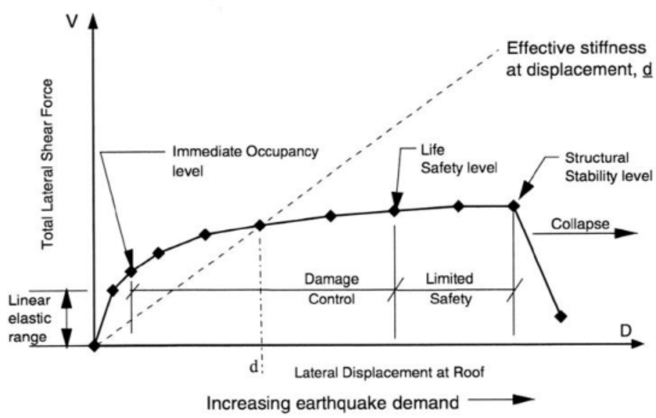


Figure 1 Capacity Curve (ATC-40,1996)

The performance level of the structure was evaluated based on ATC-40 to identify the earthquake hazards by setting the allowed level of damage (performance based). It was determined based on the capacity curve which illustrates the relationship between the lateral displacement and the magnitude of the force acting on the structure (Figure 1).

During the lateral response that occurs in the structure based on the capacity curve in Figure 1, there are certain performance levels. Performance levels based on ATC-40 are:

- Level SP-1 Immediate Occupancy (IO)
- Level SP-2 Damage Control (DC)
- Level SP-3 Life Safety (LS)
- Level SP-4 Limited Safety
- Level SP-5 Structural Stability (SS)
- Level SP-6 Not Considered

To get the level of structural performance, it is necessary to first check the lateral deformation of the maximum deviation and the maximum inelastic deviation according to the ATC-40 provisions as shown in Table 1.

Table 1 Lateral Deformation Limits (ATC-40, 1996)

Deviation Limits	Level of Structural Performance			
	IO	***DC***	***LS***	***SS***
Maximum Total Deviation	0,01	0,01-0,02	0,02	0,33 V_i/P_i
Maximum Inelastis Deviation	0,005	0,005-0,015	∞	∞

FEMA 356 defines structural performances levels from various combinations of structural and nonstructural performance levels. The structural performances levels are as follows may be illustrated in Figure 2.

- Immediate Occupancy (IO), the structure will be safe an in service after the earthquake
- Life Safety (LS), the structure is damaged but still remains at a marginal level of collapse
- Collapse Prevention (CP), the structure is able to resist the gravity loads but retains no margin agains collapse

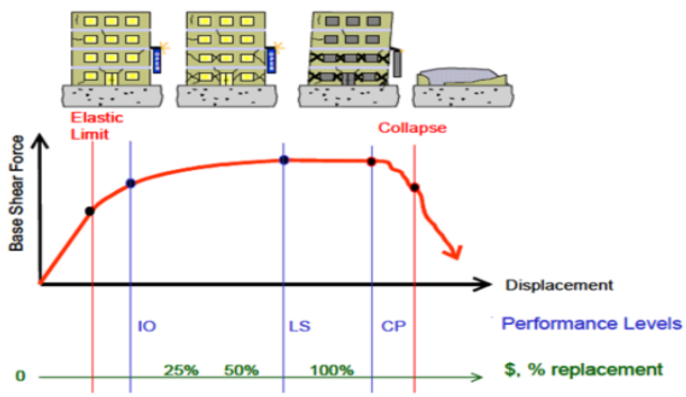


Figure 2 The structural performances levels (FEMA 356, 2000)

These performance levels were assessed by using damage variable as drift (the rooftop displacement) and plastic deformation (plastic hinges).

Table 2 Performance Levels of the Transfer Coefficient Methods (FEMA 356)

Earthquake Design Level	Concrete Structure Performance Target Level			
	Operational	**IO**	**LS**	**CP**
	Level (1-A)	Level (1-B)	Level (3-C)	Level (5-E)
Drift %	0 – 0,5 %	0,5 – 1 %	1 % - 2 %	2 % - 4 %

The nonlinear procedures of FEMA 356 plastic hinge rotation capacities require definition of the nonlinear load-deformation relation. Such a curve is given in Figure 3. The points (A, B, C, D and E) are used to define the load-deformation behavior of plastic hinge (hinge rotation behavior). Point A corresponds to the unloaded condition, point B corresponds to the nominal steel yield strength. The slope of line BC is usually taken equal between 0% and 10% of the initial slope (line AB). Point C has resistance equal to the nominal strength. The line CD represents the initial failure. It may be associated with fracture of the bending reinforcement, spalling of concrete or shear failure following initial yield. Line DE represents the residual strength of the member. It may be non-zero in some cases or practically zero in others. Point E corresponds to the deformation limit. Meanwhile the points IO, LS and CP are used to define the structural performances levels (acceptance criteria for the hinge).

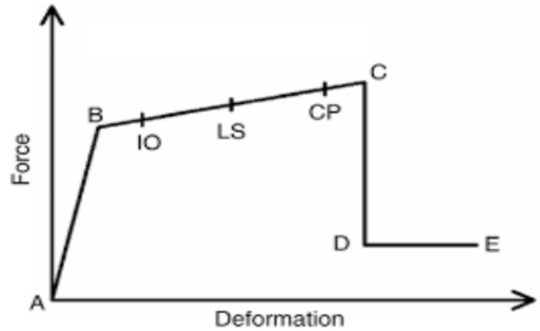


Figure 3 Typical Load – Deformation Relation and Target Performance Levels (FEMA 356, 2000)

The target displacement, δ_T shall be calculated in accordance with Equation 1:

$$\delta_T = C_0 C_1 C_2 C_3 S_a \left(\frac{T_e}{4\pi}\right)^2 g \qquad (1)$$

T_e = effective fundamental period under consideration (sec.)

C_0 =modification factors to relate spectral displacement into roof displacement. The modification factors are based on FEMA 356 (2000).

C_1 =modification factor to relate expected maximum inelastic displacement to displacement calculated for linear elastic response.

$$C_1 = 1{,}0 \text{ for } T_e \geq T_s$$

$$C_1 = \frac{\left[1+(R-1)\cdot\frac{T_s}{T_e}\right]}{R} \text{ for } T_e \geq T_s \qquad (2)$$

R = ratio of elastic strength demand to be calculated yield strength coefficient.

$$R = \frac{S_A}{V_y/W} C_m \qquad (3)$$

S_a = response spectrum acceleration, at the effective fundamental period and damping ratio under consideration.

V_y = yield strength

W = effective seismic weight

C_m = effective mass factor taken as the effective model mass calculated for the fundamental mode.

C_2 = modification factor to represent the effect of pinched hysteretic shape, stiffness degradation and strength deterioration on maximum displacement response.

C_3= modification factor to represent increased displacements due to dynamic P-Δ effects.

$$C_3 = 1{,}0 + \frac{|\alpha|(R-1)^{3/2}}{T_e} \qquad (4)$$

α= the ratio of post-yield stiffness to effective elastic stiffness, where the nonlinear force-displacement relation shall be characterized by a bilinear relation.

g= acceleration of gravity

The plastic joints in the concrete construction are usually formed at the position of the maximum moments and this is assumed, in this case, to be on the bridge pillar. Therefore, the restraints on these pillars are required to be good and sufficient for the bridge pillars to be plasticized due to bending (bending) and they are provided shear reinforcement to prevent plasticization caused by shear forces during an earthquake.

AASHTO stipulates that a bridge needs to be designed to ensure the plastic joints first form on the pillars before piling during an earthquake. This is necessary due to the difficulty in predicting the reliability of the structure when the plastic joints formed occur on the pile embedded in the ground which makes it precarious to ascertain determine whether it is broken. Response Spectra is used to predict the shear force value of the structure when an earthquake occurs. Spectra response is calculated based on parameters according to the location of the bridge structure built. The value of the basic shear force V based on this spectral response is used as a reference to determine the reliability of bridge structures that experience pushover forces.

2. METHOD

This research was conducted in three stages with the first being the production of the initial model and input of bridge structure data, the second is the structural analysis phase consisting of moment-curvature analysis, determination of lateral static pushover loads, and determination of plastic joint points while the third is the pushover and performance-based stage based on ATC-40 and FEMA 356.

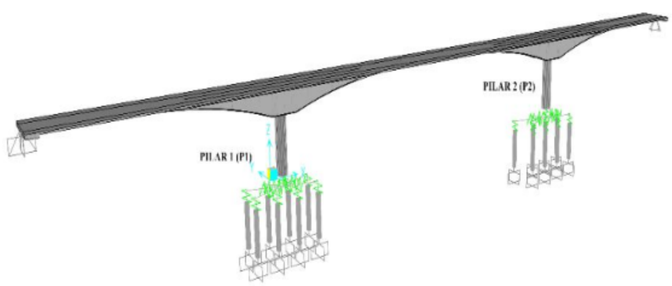


Figure 4 Long Span Bridge Model with Pushover Analysis

Figure 4 is an initial model in this study with material data of pillar and pile elements as follows:

a. Bridge Type = Balanced Cantilever
b. Concrete compressive strength quality (f_c') = 29 MPa
c. Quality of reinforcing steel (f_y) = 400 MPa
d. Pillar diameter = 2,50 m
e. Pile Diameter = 1,00 m
f. Main bar
   - Pillar = 124-D32
   - Pile = 30-D32
g. Cross bar = D13-150

The cross-section moment curves in Figure 5 (a) and (b), obtained from the results of cross-section analysis of pillars and piles with reinforcement details as above.

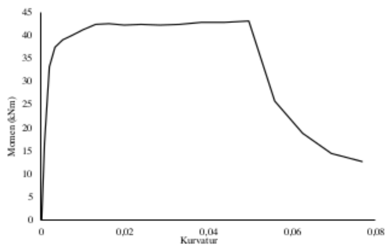


Figure 5(a) Pillar Cross-Moment Curve

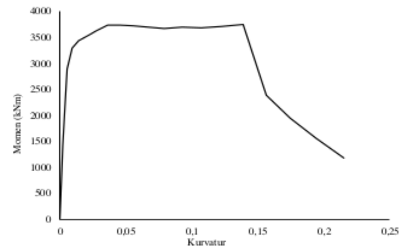


Figure 5(b) Momentary Curve of Pile Section

The plastic joints were modeled in this study to be distributed in cross-sections and the direction of the height zone of the pillar joints and piles while those modeled for the pillar elements were placed around both ends and in the middle of the pillar spans. Meanwhile, the earthquake load was acceleration-type based on the Jakarta earthquake response spectrum with the force obtained based on shear V of the bridge structure recorded to be 13364,685 kN and used as a reference to determine the reliability of bridge structures experiencing pushover forces.

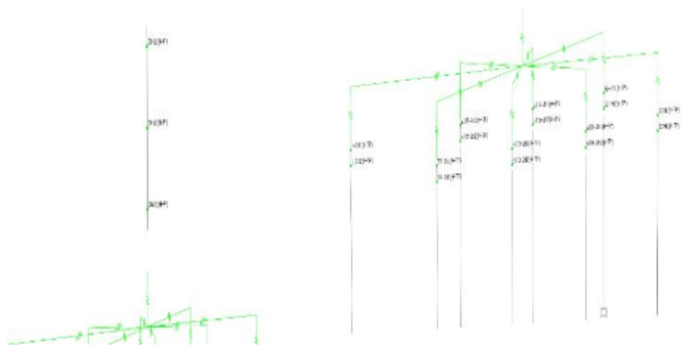


Figure 6 Location of Plastic Joints on Pillars and Piles

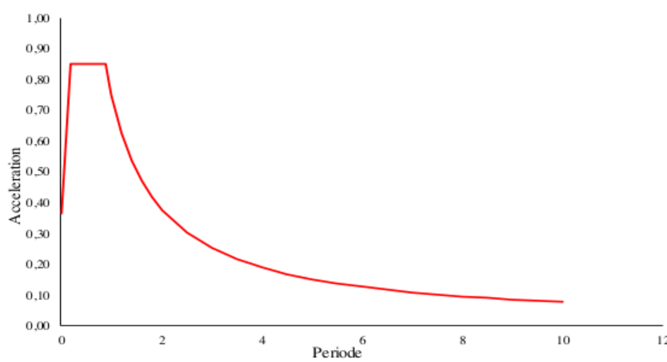


Figure 7 Jakarta Spectra Response

The pushover and performance-based evaluation for the initial model that met the design criteria and discovered to be reliable against earthquakes were followed by the analysis of the test model which is a variation of the initial model with a cross-section of pillars and piles and different reinforcement ratio as indicated in the following table. The model speciment was, however, expected to be a reference to design a new bridge structure.

Table 3 Model Speciments

No	Code	Pillar		Pile	
		D (m)	Ratio (%)	D (m)	Ratio(%)
1	Initial Model (A-P1-TP1-2-3)	2,5	2,032	1	3,072
2	A-P1-TP1-1-3	2,5	1,278	1	3,072
3	A-P2-TP1-2-3	1,5	2,185	1	3,072
4	A-P2-TP1-3-3	1,5	3,004	1	3,072
5	A-P1-TP2-2-2	2,5	2,032	1,5	2,002
6	A-P1-TP2-2-3	2,5	2,032	1,5	3,004

3. RESULT AND DISCUSSION

From the results of the initial model pushover analysis, the capacity curve and capacity spectrum based on ATC-40 are as follows:

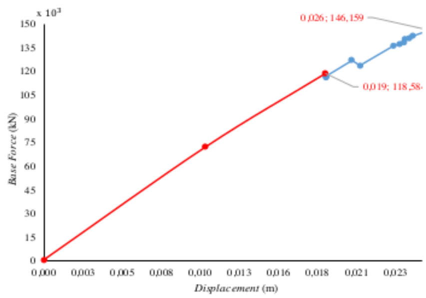


Figure 8(a) Capacity Curve X

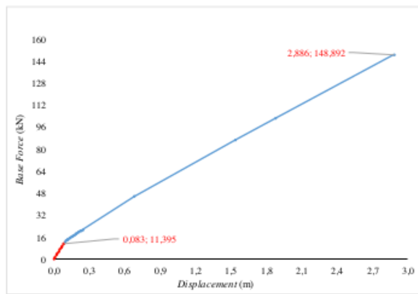


Figure 8(b) Capacity Curve Y

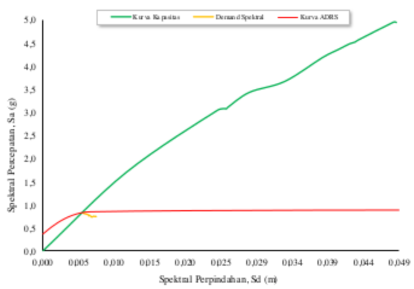


Figure 9(a) Capacity Spectrum X

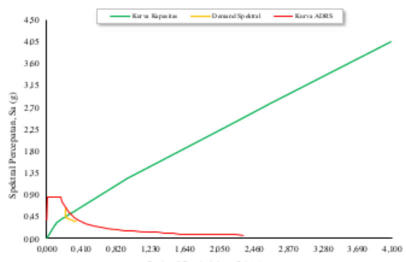


Figure 9(b) Capacity Spectrum X

Based on the ATC-40, the value of the X direction performance point consists of Base Shear (V) of 34640.85 kN, displacement (D) of 0.006 m, effective period value (Teff) of 0.165 seconds and effective attenuation (βeff) of 0.05 % As for the Y direction, Base Shear (V) of 16652.102 kN, displacement (D) of 0.159 m, effective period value (Teff) of 1.425 seconds and effective attenuation (βeff) of 0.091%. Shear force Y = 16652,102 kN <direction shear force X = 34640,850 kN. From the comparison of the shear force values it was concluded that the damage to the bridge structure was determined by the earthquake in the Y direction.

Table 4 Plastic Joint Mechanism Caused by Push Load In X

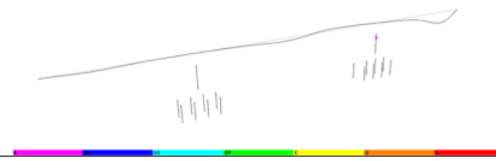
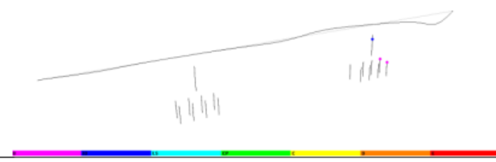
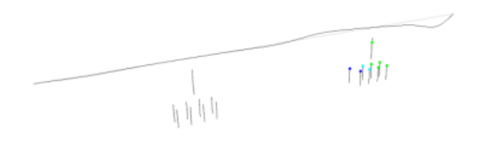
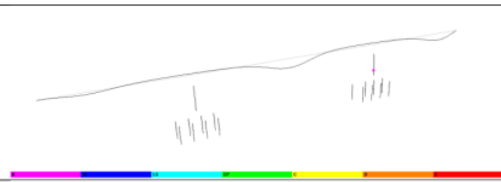
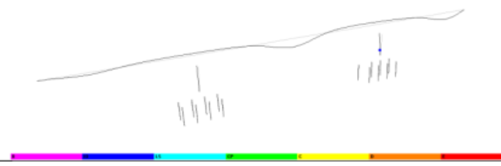
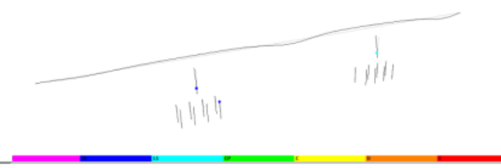
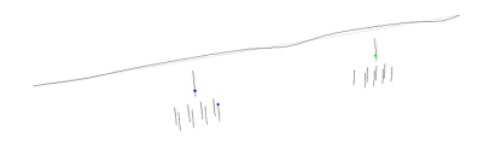
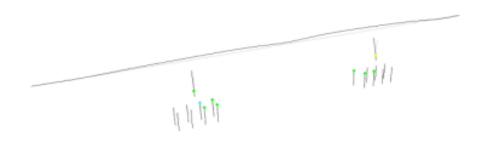
No	Plastic Joint Conditions	Load	Plastic Joint Location
1	A-B	Step 1	
2	B-IO	Step 2 - 5	
3	IO-LS	Step 6	
4	LS-CP	Step 8	
5	CP-C	Step 9	

Table 5 Plastic Joint Mechanism Caused by Push Load in Y.

No	Plastic Joint Conditions	Load	Plastic Joint Location
1	A-B	Step 1-8	

2	B-IO	Step 9 - 11	
3	IO-LS	Step 12	
4	LS-CP	Step 23	
5	CP-C	Step 26	
6	C-D	Step 69	

Target displacement based on FEMA-356 is calculated by the formulas (1) - (4) and the results are divided by the height of the bridge structure H = 16.8 m.

Table 6 Comparison of Initial Performance Base Models

Pushover Load Direction	**Parameter**	**Pushover Analysis Results**	
		ATC-40	**FEMA 356**
X	Displacement Targets (m) Drift Performance Level	0,006 0,00 *Operational*	0,012 0,07 % *Operational*
Y	Displacement Targets (m) Drift Performance Level	0,159 0,01 *Immediate Occupancy*	0,058 0,35 % *Operational*

FEMA-356 showed the bridge structure was at the operational performance level after the earthquake as indicated by the ATC-40 at the IO level. Moreover, the plastic joint mechanism showed the plastic joints with CP performance level for X-direction earthquake loads were formed simultaneously on pillars and piles while those with CP and C performance levels for Y direction earthquake loads occurred at the same point on the bottom pillar. This shows the failure of the plastic joint occurred first in the pillar and this fulfilled the design provisions based on the AASHTO, LRFD Bridge Design Specifications (2012). Furthermore, the earthquake load which determines the failure of the bridge structure based on ATC-40 is on the Y direction earthquake at 16652.102 kN and this is 25% greater than the base shear force Vbase. This indicates the initial model relied on earthquake plans with a reliability ratio of 1.25 and this means the initial bridge structure model met the design criteria and is considered reliable against earthquakes.

Table 7 Performance Base ATC-40

No	Pushover Load Direction	*Displacement* (m)	H (m)	Drift	Performance level
1	Test model A-P1-TP1-1-3: X	0,006	16,8	0,000	*Operational*
	Y	0,168	16,8	0,010	IO
2	Test model A-P2-TP1-2-3: X	0,006	16,8	0,000	*Operational*
	Y	0,235	16,8	0,014	DC
3	Test model A-P2-TP1-3-3: X	0,006	16,8	0,000	*Operational*
	Y	0,234	16,8	0,014	DC
4	Test model A-P1-TP2-2-3: X	0,005	16,8	0,000	*Operational*
	Y	0,147	16,8	0,010	IO
5	Test model A-P1-TP2-2-2: X	0,005	16,8	0,000	*Operational*
	Y	0,147	16,8	0,010	IO

The reliability test of the A-P1-TP1-1-3 model compared to the magnitude of the planned earthquake load showed a result of 15% while the structural reliability was found to have been reduced by 10% due to the reduction in the pillar reinforcement ratio by 1%. Meanwhile, the structural reliability was reduced by 20% due to a decrease in pillar diameter and pillar reinforcement ratio. Moreover, the reliability of the A-P1-TP2-2-3 and A-P1-TP2-2-2 test models against the planned earthquake was 28% and this increased the reliability by 3% when compared to the initial model. This shows the diameter and reinforcement ratio of the pillar has a greater effect on the reliability of the bridge than the diameter and reinforcement ratio of the pile. Furthermore, the structural performance in the test model was also influenced by the stiffness and reinforcement ratio of pillars and piles.

Table 8 Performance Base FEMA-356

No	Pushover Load Direction	Displacement (m)	H (m)	Drift	Performance level
1	Test model A-P1-TP1-1-3: X	0,016	16,8	0,000	*Operational*
	Y	0,159	16,8	0,010	IO
2	Test model A-P2-TP1-2-3: X	0,031	16,8	0,000	*Operational*
	Y	0,089	16,8	0,014	IO
3	Test model A-P2-TP1-3-3: X	0,017	16,8	0,000	*Operational*
	Y	0,085	16,8	0,014	IO
4	Test model A-P1-TP2-2-3: X	0,016	16,8	0,000	*Operational*
	Y	0,053	16,8	0,010	IO
5	Test model A-P1-TP2-2-2: X	0,020	16,8	0,000	*Operational*
	Y	0,059	16,8	0,010	IO

Based on ATC-40, the performance level of the test model is at the IO and DC level. The A-P2-TP1-2-3 and A-P2-TP1-3-3 test models with reduced pillar diameters show a lower level of performance ie DC level with greater deformation than other test models of 0.234 m and 0.234 m. Based on ATC-40, the test model with IO level shows the condition of the structure will be safe in the event of an earthquake with the risk of loss of life and structural failure that is not very significant. Whereas at the DC level, structural conditions are in transition between the IO and LS levels. In this condition, the structure is still able to withstand earthquakes that occur with very little risk of casualties.

4. CONCLUSION

- Based on the ATC-40, the base shear force of Vbase for the initial model of 16652.102 kN is greater than the earthquake value of the Vbase plan of 13364,685 kN. Shows that the initial model is reliable against planned earthquakes with a reliability ratio of 1.25.
- The displacement in the initial model produced a displacement drift of 0 on the x-axis and 0.01 on the y-axis. The roof displacement ratio according to ATC-40 shows this performance is at the level of the IO and the bridge structure is still safe during an earthquake with the risk of loss of life and structural failure found not to be very significant and can be re-function immediately.
- The target displacement in the initial model produced an x-axis drift ratio of 0.1% and a y-axis of 0.3%. These drift constraints show the level of structural performance at the operational level presents no significant damage to the structural and non-structural components.
- The reliability of the model against the planned earthquake load was 15% while the reliability of the structure was reduced by 10% due to the reduction of the pillar reinforcement ratio by 1% and by 20% due to a decrease in pillar diameter and pillar reinforcement ratio.
- The diameter and ratio of pillar reinforcement were found to have a greater effect on the reliability of the bridge than the diameter and reinforcement ratio of the pile and the structural performance in the test model was also influenced by the stiffness and reinforcement ratio of pillars and piles.

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