

The effect of tsunami loads on Pasar Raya Inpres Block III building in Padang City based on FEMA P-646

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Abstract. Pasar Raya Inpres Block III building in Padang City is a traditional market building that located in the high seismic zone and prone to tsunami. The building consists of five floors that planned to be used as a vertical evacuation for the surrounding community. However, in designing the building structure, it is only planned to strongly against the earthquake load and it does not take into account the tsunami load. In this study, the effect of tsunami load on the building was investigated. This building will be analyzed and modeled by using ETABS v.9.7.1 software. The applied earthquake load in the building refers to SNI 1726:2012, while the tsunami loads refer to FEMA P-646/2012. From the analysis results, it was found that the columns of the building are capable of restrained the tsunami loads, but the beams were not strong enough to withstand the tsunami loads. The beam failure occurred due to the small load-bearing capacity of the beams against the tsunami loads includes buoyant force, additional gravity load, and uplift hydrodynamic force. Furthermore, the retrofiting of the building should be carried out, especially on the beams of the building before being used as a tsunami vertical evacuation building.

1 Introduction

Padang City is the third largest city in Sumatera Island that located along the west coast of Sumatera. It appears before the border of Eurasia and Indo-Australia plates, and the area is at a high risk of earthquake and tsunami disaster. This condition makes Padang City very vulnerable to tsunami hazard [1].

There are more than fifty percent of Padang people live in the lowland areas close to the beach. If a large tsunami happens, those in the area most need evacuation sites. Padang City is situated on the very flat liquefiable ground. Everyone must move more than 3 km from the beach line to reach a height of 5 meters above sea level (masl). It is very complicated to evacuate many people (more than 400,000 population) in a short period to the tsunami safe zone. The residents would not have enough time to reach high ground. The transportation facilities would not help due to the public panic met with the number of vehicles will only cause traffic [2,3].

To overcome this problem, Padang City government has implemented various rescue strategies; one of them is to build a vertical evacuation to avoid tsunami waves by providing buildings for the shelter. Some of them are the market buildings that have more than four stories, in which the roof is served as a tsunami evacuation. The tsunami evacuation buildings are primarily required to have the capacity to resist anticipated tsunami loads without collapse, overturning, or lateral movement for the life safety of evacuees.

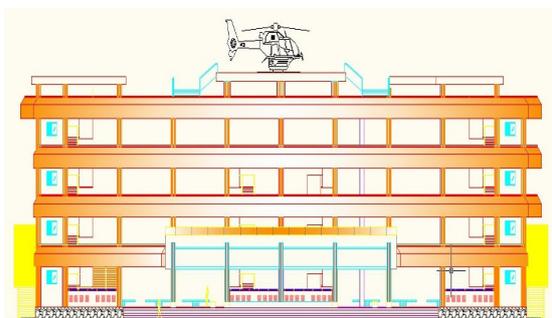


Fig. 1. The front view and photo of Pasar Raya Inpres Block III building.

One of the areas that prone tsunami and needed the shelter is Central Market. This location is quite close to West Padang coastal area. It is one of the dense areas of

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society because it serves as a center to buy and sell in Padang City.

Currently, one of the market buildings, Pasar Raya Inpres Block III is located in Pasar Raya, Padang, as shown in Fig. 1, where the building was previously a collapsed building due to the West Sumatera earthquake in 2009. Then, the building was destroyed and built again in the same location. These buildings are initially designed to resist the earthquake load in accordance with SNI 1726-2012 [4]. The building design does not take into account the tsunami loads. In order to build a shelter or temporary evacuation site located in the red zone (tsunami prone) area, the building must be able to withstand earthquake loads and strong impact of tsunami loads. Therefore, the buildings are then subjected to tsunami loads following the proposed the FEMA P-646 in order to investigate the effect and what improvements need to be made to the buildings to ensure their safety in the event of a tsunami [5-7].

2 Structural analysis

Feasibility evaluation of the structure consists of serviceability limit performance evaluation, ultimate limit performance that determined by the story drift, the column capacity using P-M interaction diagram, and the load-bearing capacity of the beam. Based on Google Earth, the distance of the building was approximately 500 m from the seacoast and at the height of 10 m above the sea level, as shown in Fig. 2. The building of Pasar Raya Inpres Block III is located in Pasar Raya, Padang that can accommodate around 3750 people. The building shelter is in the Northern region of Padang City, which is located in red (no safe) zone area. The population in this area is very dense and it has a better chance for the residents who live on around the shelter, to escape from tsunami.

The data needed for this study are:

- Technical data consist of the data about the condition of existing research object. It was obtained from the field survey and contacting the relevant parties.
- Non-technical data consist of a map of tsunami hazard in Padang City.

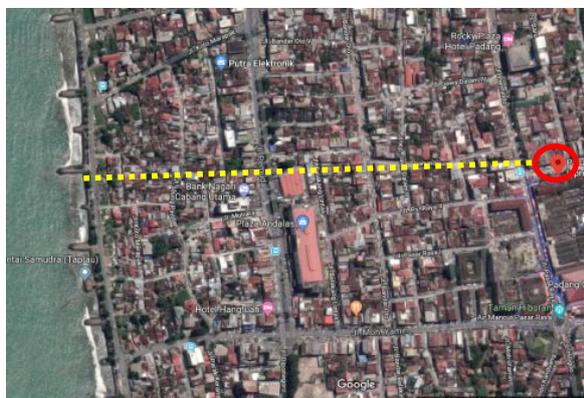


Fig. 2. The location of Pasar Raya Inpres Block III building in Google Earth.

In this study, the market building is designed for earthquake load and evaluated for additional tsunami loadings. The building is designed for both reinforced concrete special moment resisting frames (SMRF) and lateral force resisting systems. These buildings are considered at locations: Padang, Indonesia by coordinates of Latitude -0.947 N and Longitude 100.417 W. The details of the building are described in Table 1. The market was a five-story reinforced concrete building, which is designed by using current seismic code (SNI 1726-2012).

The building is modeled and analyzed by using commercial software, ETABS v9.7.1 [8]. Columns and beams are modeled as a frame element, whereas the slabs are modeled as a shell element. Modeling of the building structure is performed based on the drawing of the design consultant. Fig. 3 shows the structural model of Pasar Raya Inpres Building.

Table 1. Details of Pasar Raya Inpres Block III building.

Structural Elements	Properties
Beam size	B1 (35 cm x 70 cm); B2 (30 cm x 60 cm); B3 (25 cm x 50 cm); B5 (40 cm x 80 cm); BL1 (15 cm x 70 cm); BL2 (15 cm x 80 cm); BR1 (20 cm x 60 cm)
Column size	K1 (D-70 cm); K1A (D-80 cm); K2A (D-100 cm); K2 (D-120 cm); K3 (30 cm x 40 cm); K3A (40 cm x 200 cm); K4A (35 cm x 35 cm)
Number of floors	5
Total height of frame	29.2 m
Strength of concrete	K-350 ($f'_c = 29.05$ MPa)
Yield strength of reinf. (f_y)	240 MPa, $D < 13$ mm 400 MPa, $D > 13$ mm

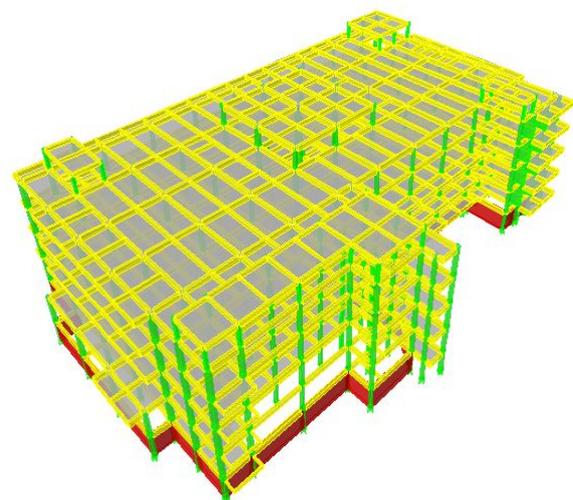


Fig. 3. The 3-D modeling of Pasar Raya Inpres Block III building using ETABS.

The procedures of linear analysis are carried out considering the linear behavior of the material with a linear stress-strain relationship. The procedures are adjusted according to the overall building deformations and material behavior criteria, to consider the nonlinear behavior of earthquake seismic response.

2.1 Design of loads

The standard code of the minimum load for design buildings and other structures, SNI 1727-2013 [9], was used to design the dead and live loads of the building. The dead load includes all components of the building structure, namely beams, columns, plates, and load-bearing walls. Live load used was 250 – 600 kg/m², where the function of the building is as a store and evacuation building.

Seismic load based on SNI 1726-2012 using response spectrum was obtained from the website http://puskim.pu.go.id/Aplikasi/desain_spektra_indonesia_2011/ by entering the name of a city. Fig. 4 shows the response spectrum of Padang City with soft soil type. The value of spectral design S_{D1} and S_{DS} are 0.6g and 0.932g, respectively [10].

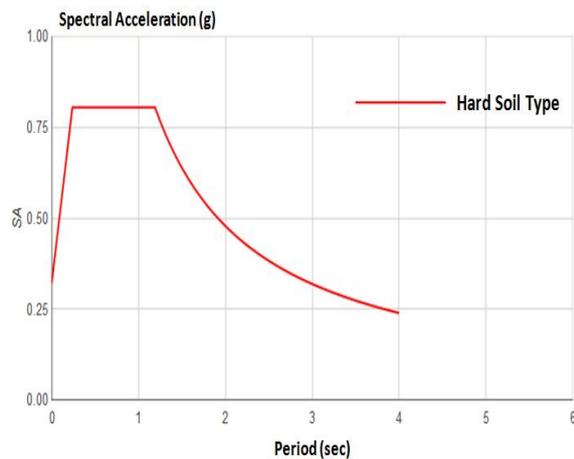


Fig. 4. Response spectrum graph for Padang City based on SNI 1726-2012.

Tsunami loads are calculated based on the FEMA P-646 (2012). The detail (equation and input) of the loads from the code is presented as follows:

2.1.1 Hydrostatic force

The hydrostatic force was calculated by using equations 1 and 2:

$$F_h = p_c \cdot A_w = 0,5 \cdot \rho_s \cdot g \cdot b \cdot h_{\max}^2 \quad (1)$$

$$h_{\max} = 1,3 \cdot R^* - z_w \quad (2)$$

These loads are given by using the action line of the force resultant, as seen in Fig. 5.

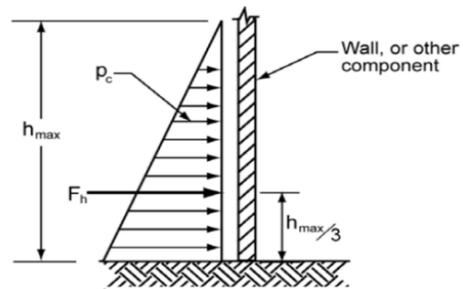


Fig. 5. Distribution of hydrostatic force [6].

2.1.2 Buoyant force

The buoyant force was computed using equation 3:

$$F_b = \rho_s \cdot g \cdot V \quad (3)$$

The buoyant force on an overall building is shown in Fig. 6.

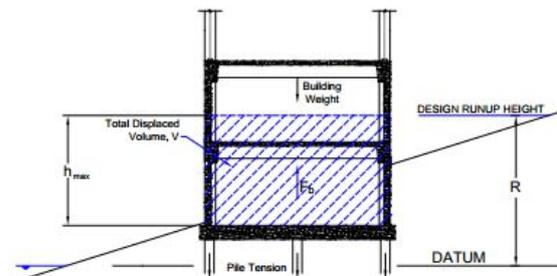


Fig. 6. The acting of buoyant force on the shelter structure [6].

2.1.3 Hydrodynamic force

The hydrodynamic force was calculated by using equation 4:

$$F_d = 0,5 \cdot \rho_s \cdot C_d \cdot B \cdot (hu^2)_{\max} \quad (4)$$

The hydrodynamic force is applied approximately at the centroid of the wetted surface of the component, as seen in Fig. 7.

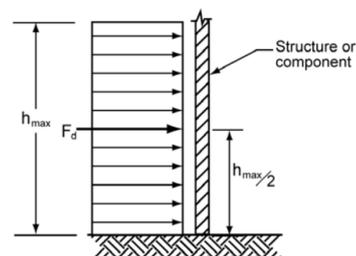


Fig. 7. Distribution of hydrodynamic load on the structure [6].

2.1.4 Impulsive force

This force is taken approximately 1.5 times the subsequent hydrodynamic force, as shown in equation 5:

$$F_s = 1,5 \cdot F_d \quad (5)$$

Impulsive forces act on members at the leading edge of the tsunami bore, as shown in Fig. 8.

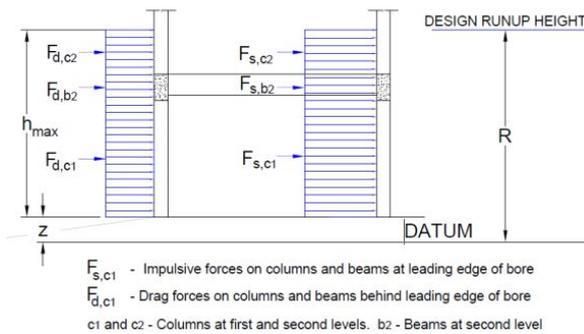


Fig. 8. Impulsive force on the structure [6].

2.1.5 Impact force

As an approach, this value was calculated by using equation 6:

$$F_i = 1,3 \cdot u_{\max} \sqrt{k \cdot m_d (1 + c)} \quad (6)$$

The impact forces are assumed to act locally on a single member of the structure at the elevation of the water surface, as seen in Fig. 9.

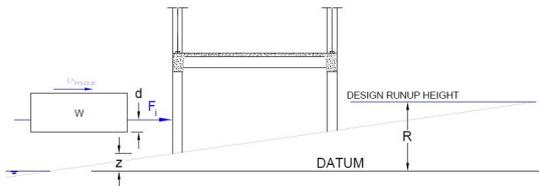


Fig. 9. Impact load on the structure [6].

2.1.6 Debris force

To calculate the debris force, use equation 7:

$$F_{dm} = 0,5 \cdot \rho_s \cdot C_d \cdot B_d \cdot (hu^2)_{\max} \quad (7)$$

2.1.7 Extra gravity load

Water retained on the top of elevated floors will apply additional gravity loads which is shown in Fig. 10. The maximum potential downward load per unit area was calculated using equations 8 and 9:

$$f_r = \rho_s \cdot g \cdot h_r \quad (8)$$

$$h_r = h_{\max} - h_l \leq h_{bw} \quad (9)$$

2.1.8 Uplift hydrodynamic force

Uplift forces will be applied to floor levels of a building that are submerged by tsunami inundation. This load was calculated by using equation 10:

$$F_u = 0,5 \cdot \rho_s \cdot C_u \cdot A_f \cdot u_v^2 \quad (10)$$

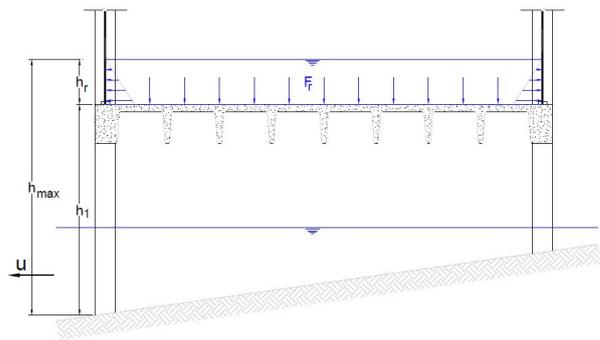


Fig. 10. Extra gravity load on the structure [6].

The load's value is calculated based on the prediction of tsunami waves high, as shown in Fig. 11. The assumptions in predicting the height of tsunami wave is based on the building ground elevation and the distance from the shoreline. The simplest way to express the magnitude of a tsunami load is based on its wave height or depth of inundation at a given location [11]. The height of the tsunami wave assumption was obtained from the calculation as follows:

- Elevation basic structure (Z_w) = 10 masl
- Run-up (tsunami hazard map) (R^*) = 14 masl
- Run-up design (R) = 18.2 masl
- Maximum depth of puddle (h_{\max}) = 8.2 m

The calculation results of tsunami loads are given in Table 2. Tsunami loads are applied to these structural members including columns, beams, floor slab, and structural walls.

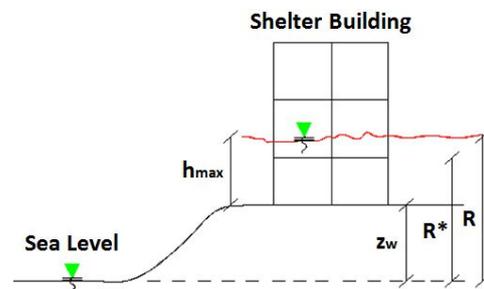


Fig. 11. The maximum height of the tsunami wave.

Table 2. Calculation results of tsunami loads on Pasar Raya Inpres Block III building.

Code	Type of Tsunami Load	Value of Force
F_h	Hydrostatic	25887,4 kg
F_b	Bouyant	4400 kg/m ² (1 st & 2 nd floors) 220 kg/m ² (3 rd floors)
F_d	Hydrodynamic	7418,95 kg
F_s	Impulsive	11128,43 kg
F_i	Impact	55238,6 kg
F_{dm}	Debris	7418,95 kg
F_u	Uplift Hydrodynamic	2,501849 kg/m ²
F_r	Extra Gravity	4400 kg/m ² (1 st & 2 nd floors) 220 kg/m ² (3 rd floors)

2.2 Combinations of loads

In this study, the load combinations are provided from current Indonesian seismic code, SNI 1726-2012 and guidelines for design of structures for vertical evacuation from tsunamis, FEMA P-646 (2012), i.e.

- 1.4 DL
- 1.2 DL + 1.6 LL
- 1.2 DL + 1 LL ± 0.3 (ρQEx + 0.2 S_{DS} DL) ± 1 (ρQE_y + 0.2 S_{DS} DL)
- 1.2 DL + 1 LL ± 1 (ρQEx + 0.2 S_{DS} DL) ± 0.3 (ρQE_y + 0.2 S_{DS} DL)
- 0.9 DL ± 0.3 (ρQEx - 0.2 S_{DS} DL) ± 1 (ρQE_y - 0.2 S_{DS} DL)
- 0.9 DL ± 1 (ρQEx - 0.2 S_{DS} DL) ± 0.3 (ρQE_y - 0.2 S_{DS} DL)
- 1.2 DL + 1.0 TS + 1.0 L_{REF} + 0.25 LL
- 0.9 DL + 1.0 TS

where:

DL = Dead load

LL = Life load

L_{REF} = Life refugee load

ρ = Redundancies factor

QEx = Earthquake load in X-direction

QE_y = Earthquake load in Y-direction

TS = Tsunami load

3 Results and discussion

The analysis of the structure was carried out using ETABS v9.7.1. The analysis results on the effect of tsunami loads on the building will be presented in term of the inter-story drift, internal forces, load-bearing capacity of the structural elements and foundation capacity.

3.1 Inter-story drift

Table 3 shows the comparison of the inter-story drift value of building structure analyzed without and with tsunami loads in x and y-directions. From the table, it can be seen that the maximum increase of inter-story drift due to the presence of tsunami loads occurred in the 1st and 2nd floors at x-direction, around 11.7 % and 28.7%, respectively.

3.2 Internal forces on columns and beams

Tables 4 and 5 show the increasing percentage of internal forces on columns and beams on building with and without tsunami loads, respectively. It can be seen from the table that there is a significant increase in columns internal forces in the basement and 1st floors, as well as the beams on the 1st and 2nd floors. The increase

of axial, shear and bending moment of the column around 25 – 98 %, 0 – 80 %, and 20 – 69 %, respectively, while on the beam, the increase of shear and bending moment of the beam around 0 – 72 % and 0 – 86 %, respectively.

Table 3. The increasing percentage of inter-story drift in the structure.

Story	Displacement in X dir. (mm)		Percentage of Increasing (%)
	Without Tsunami Load	With Tsunami Load	
Base	0.000	0.000	0.000
1 st floor	3.294	3.731	11.704
2 nd floor	8.589	12.043	28.680
3 rd floor	19.986	19.987	0.003
4 th floor	29.725	29.727	0.004
Roof	36.216	36.218	0.005
Helipad	40.017	40.018	0.004

Story	Displacement in Y dir. (mm)		Percentage of increasing (%)
	Without Tsunami Load	With Tsunami Load	
Base	0.000	0.000	0.0000
1 st floor	9.738	9.738	0.0000
2 nd floor	22.340	22.340	0.0004
3 rd floor	31.749	31.749	0.0003
4 th floor	38.826	38.826	0.0005
Roof	43.308	43.308	0.0009
Helipad	45.195	45.196	0.0009

3.3 Load-bearing capacity

3.3.1 The Load-bearing capacity of columns

The axial – bending (P-M) interaction diagram is used describe the capacity of the column to resist the working loads. Figs. 12 and 13 show the P-M interaction diagram obtained from structural analysis results of the building with and without tsunami loads, respectively.

As seen in the figures, all the axial forces and bending moment are inside the diagram that means the columns of the building with applying tsunami loads is strong enough capacity to resist the loads taht is applied in the structure. Similarly, the columns also have enough shear capacity in resisting the working loads. The shear capacity of the column due to the tsunami loads are shown in Table 6 [12].

Table 4. The increasing percentage of internal forces in columns.

Story	Column/ Position	The Internal Force in Column (Without Tsunami Load)			The Internal Force in Column (With Tsunami Load)			The Percentage of Difference		
		Axial Force (kN)	Shear Force (kN)	Bend. Moment (kNm)	Axial Force (kN)	Shear Force (kN)	Bend. Moment (kNm)	Axial Force (%)	Shear Force (%)	Bend. Moment (%)
Base- ment	K1 Ext.	596.12	24.94	101.03	796.25	31.35	162.49	25.13	20.45	37.82
	K1 Int.	344.80	73.02	137.30	5717.16	200.78	321.61	93.97	63.63	57.31
	K1A Ext.	380.60	33.69	73.15	763.41	50.59	152.76	50.14	33.41	51.88
	K1A Int.	111.89	67.00	179.10	5313.98	140.79	269.06	97.89	52.41	33.44
	K2A Int.	57.48	79.27	267.73	4027.42	79.28	379.17	98.57	0.01	29.39
	K2 Int.	1210	100.54	405.11	2485.17	107.70	855.03	50.78	6.65	52.59
	K3A Ext.	439.06	2.92	5.95	1151.12	5.35	19.57	61.85	45.52	69.58
	K3 Int.	44.87	43.49	88.79	436.24	100.96	174.65	94.37	68.94	65.92
1 st floor	K1 Ext.	585.1	160.92	413.91	2234.56	367.37	740.34	73.82	56.20	44.09
	K1 Int.	318.57	170.33	393.28	2923.27	350.42	709.23	89.10	51.39	44.55
	K1A Ext.	372.4	266.43	696.86	1471.53	481.58	968.32	74.69	44.68	28.03
	K1A Int.	112.7	114.66	348.70	2622.54	281.41	523.76	95.70	59.26	33.42
	K2A Int.	57.73	144.93	496.28	1701.23	391.59	689.23	96.61	62.99	27.99
	K2 Int.	0.83	80.96	571.38	19.34	408.93	716.20	95.71	80.20	20.22
	K3A Ext.	263.43	124.02	295.63	1173.80	527.31	923.76	77.56	76.48	68.00
		K3 Int.	44.87	43.49	88.79	436.24	100.96	174.65	89.71	56.92

Table 5. The increasing percentage of internal forces in beams.

Beam	Story	The Internal Force in Beam (Without Tsunami Load)		The Internal Force in Beam (With Tsunami Load)		The Percentage of Difference	
		Shear Force (kN)	Bend. Moment (kNm)	Shear Force (kN)	Bend. Moment (kNm)	Shear Force (%)	Bend. Moment (%)
B1	1	210.18	377.65	654.21	1298.70	67.8	70.9
	2	390.91	376.10	653.92	1300.243	40.2	71.0
	3	354.48	441.02	354.48	441.03	0	0
	4	247.52	403.88	247.52	403.85	0	0
	Roof	225.98	397.32	225.98	397.32	0	0
B2	1	38.17	35.79	132.81	260.88	71.2	86.3
	2	47.95	55.97	63.24	157.21	24.2	64.4
	3	46.95	53.08	46.95	53.08	0	0
	4	55.15	60.41	55.15	60.42	0	0
	Roof	64.14	39.33	64.14	39.33	0	0

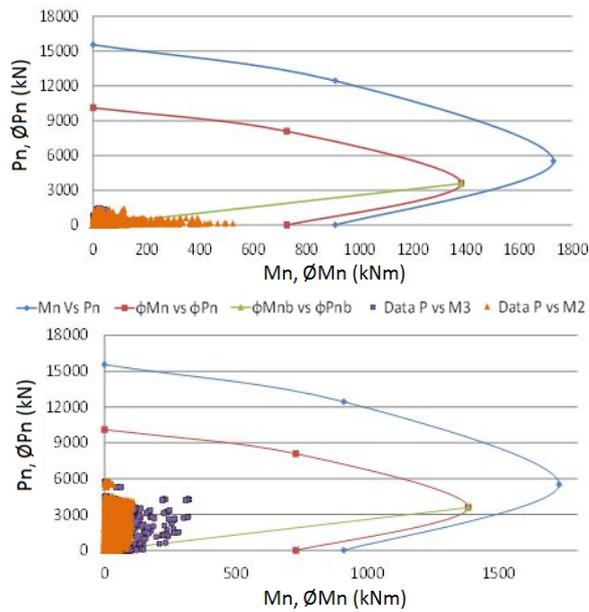


Fig. 12. P-M interaction diagram of column K1 in the basement floor.

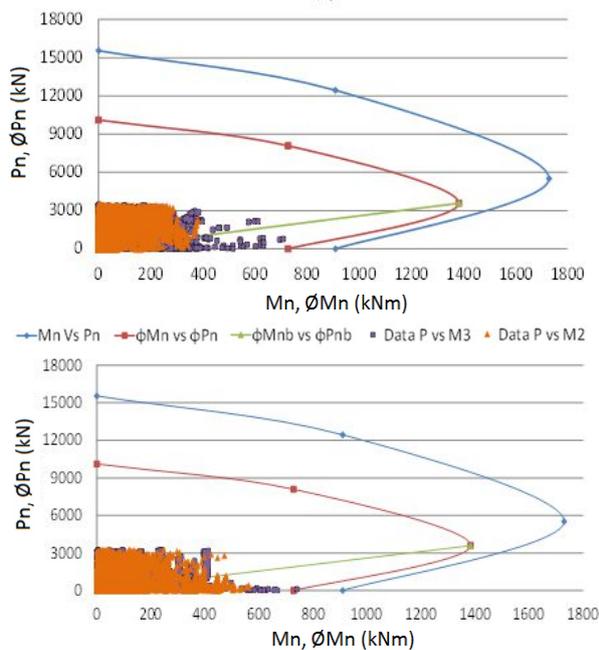


Fig. 13. P-M interaction diagram of column K1 in the 1st floor.

3.3.2 The Load-bearing capacity of beams

The bending moment and the shear force on the beams due to the tsunami loads that can be seen in Tables 7 and 8. From these Tables, it is concluded that some beams on the 1st and 2nd floors are not able to withstand the load due to the tsunami loads, either on the bending or shear capacity. This is due to the small capacity of the beams in resisting the buoyant force, the additional gravitational load, and the uplift hydrodynamic forces.

Table 6. The shear capacity of the columns in the basement and 1st floor.

Story	Column/ Position	ØVn (kN)	Vu (kN) from ETABS, with tsunami load	Capacity
Base- ment	K1 Ext.	701.174	68.480	OK
	Int.	701.174	200.780	OK
	K1A Ext.	859.917	50.590	OK
	Int.	859.917	140.790	OK
	K2A Ext.	1217.070	79.280	OK
	K2 Ext.	1627.111	107.700	OK
	K3A Ext.	8411.260	523.673	OK
1	K3 Ext.	1334.981	42.230	OK
	K1 Ext.	701.174	367.370	OK
	Int.	701.174	350.420	OK
	K1A Ext.	859.917	481.580	OK
	Int.	859.917	281.410	OK
	K2A Ext.	1217.070	391.590	OK
	Int.	1627.111	408.930	OK
1	K3A Int.	8411.260	902.414	OK
	K3 Int.	1334.981	100.960	OK

Table 7. The flexural capacity of the beams in the 1st and 2nd floors.

Beam	Story	Position	ØMn (kN)	Mu (kN) with tsunami load	ØMn ≥ Mu
B1	1 st floor	Ped.	636.438	1298.7	NK
		Mid.	466.435	1298.7	NK
	2 nd floor	Ped.	636.438	1300.2	NK
		Mid.	466.435	1300.2	NK
B2	1 st floor	Ped.	344.388	260.8	OK
		Mid.	241.215	260.8	NK
	2 nd floor	Ped.	344.388	157.2	OK
		Mid.	241.215	157.2	OK
B3	1 st floor	Ped.	245.160	206.6	OK
		Mid.	159.051	206.6	NK
	2 nd floor	Ped.	245.160	265.5	NK
		Mid.	159.051	268.4	NK
B5	2 nd floor	Ped.	938.445	1075.7	NK
		Mid.	586.561	1075.7	NK
BL1	1 st floor	Ped.	116.928	93.3	OK
		Mid.	116.928	93.3	OK
	2 nd floor	Ped.	116.928	188.2	NK
		Mid.	116.928	188.2	NK
BL2	2 nd floor	Ped.	135.074	387.6	NK
		Mid.	135.074	387.6	NK
BR1	1 st floor	Ped.	249.196	23.2	NK
		Mid.	249.196	26.5	NK

Table 8. The shear capacity of the beams in the 1st and 2nd floors.

Beam	Story	Position	ϕV_n (kN)	V_u (kN)	$\phi V_n \geq V_u$
B1	1 st floor	Ped.	691.5	654.2	OK
		Mid.	513.7	654.2	NK
	2 nd floor	Ped.	691.5	653.9	OK
		Mid.	513.7	653.9	NK
B2	1 st floor	Ped.	569.1	132.8	OK
		Mid.	417.8	132.8	OK
	2 nd floor	Ped.	569.1	66.2	OK
		Mid.	417.8	66.2	OK
B3	1 st floor	Ped.	458.2	102.1	OK
		Mid.	332.1	102.1	OK
	2 nd floor	Ped.	458.2	131.7	OK
		Mid.	332.1	131.7	OK
B5	2 nd floor	Ped.	820.7	429.1	OK
		Mid.	616.3	429.1	OK
BL1	1 st floor	Ped.	426.5	90.7	OK
		Mid.	426.5	90.7	OK
	2 nd floor	Ped.	426.5	91.1	OK
		Mid.	426.5	91.1	OK
BL2	2 nd floor	Ped.	489.7	163.4	OK
		Mid.	489.7	163.4	OK
BR1	1 st floor	Ped.	535.4	15.85	OK
		Mid.	382.7	15.8	OK
	2 nd floor	Ped.	535.4	24.0	OK
		Mid.	382.7	24.0	OK

3.4 Analysis of foundation (base support) capacity

The strength of the base structure, similar to that of the upper structure, must be designed according to the standard code. The effects of soil and foundation should be considered in applying the buoyancy force to the buildings because the force acts on the bottom of the structure. The examination bearing capacity of the foundation aims to determine the ability of the foundation to bear the load due to the addition of tsunami load. Foundations checked including consideration of the effects of overturning on the vertical forces: (bearing pressure, uplift, lateral force).

3.4.1 Foundation and soil data

The type of foundation used is a bore pile with a 60cm diameter and 30m depth. The one of foundation group (one column) has three piles with material properties; the compressive strength of concrete (f_c') = 27.5 MPa and the yield strength of reinforcement bar (f_y) = 400 MPa. Each pile has 9 - $\phi 8$ steel rebar in 350 mm hollow round section.

Soil data were obtained from the N-SPT test, where the soil consists of five layers.

3.4.2 Supporting capacity of foundation on axial forces

From calculating the bearing capacity of the foundation on the pile group, the bearing capacity of the foundation is greater than the maximum load value. The supportability of the pile group is 10570.69 kN, and the maximum axial load is 1950.231 kN.

3.4.3 Supporting capacity of foundation on uplift forces

Based on the capacity calculation of the foundation on the uplift force, it is found that the tensile pull resistance capacity is higher than the lifting force. The tensile resistance of piles was obtained from the yielding force of steel bars, which was smaller than pull out strength based on the boring site test in surrounding soil.

Then, concerning the tension pile, it has to be confirmed that the value is lower than the ultimate tensile capacity (it should be the lesser value in the tensile capacity of the pile or friction of the area surrounding the pile). Concerning the compression pile, the value has to be higher than the ultimate axial bearing capacity of the pile. The tensile capacity at the pile group is 7464.448 kN while the lift force is 5717.16 kN.

3.4.4 Supporting capacity of foundation to lateral forces

From the calculation of the lateral carrying capacity of the soil, it is obtained that the foundation is able to withstand lateral loads that occur due to the tsunami, where the ultimate lateral load value on one pile is greater than the lateral load occurring. The ultimate lateral load value is 334.4 kN, while the occurred lateral load is 19.023 kN. Meanwhile, if the ability of the drill pile to withstand the lateral load due to the tsunami is calculated based on the strength of the material, the resultant capacity of the pile moment is greater than the moment that acts on the pile. The maximum moment result due to lateral load is 56127 kNm, while the result of moment pile group capacity is 177515 kNm.

Overall, under seismic and tsunami loads, the bearing and uplift capacity of the foundations is sufficient.

4 Conclusions

1. Pasar Raya Inpres Block III Padang building was strong against earthquake load, but it is not strong enough against tsunami loads. The beams structural elements on the first and second floors have not been strong enough capacity in resisting the additional tsunami loads. However, on the column structural elements, all types of columns are strong against the all working loads, including both earthquake and tsunami loads.

2. The addition of tsunami loads on the building increases the internal forces in the columns up to 98%, 80%, and 69% for axial, shear, and bending moment, respectively.
3. The increase of internal forces occurred in the beams due to the tsunami loads, with the incremental value up to 72% and 86% for shear and bending moment, respectively.
4. The pile group foundation has been strong enough capacity and able to withstand the maximum axial load, uplift, and lateral force on the site caused by the addition of tsunami loads.
5. The retrofitting of the building structure should be done especially on the beams on the first and second floors of the building before using the building as an evacuation (shelter) building.
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