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Structural Failure Analysis of Building E ITERA Due to The Pounding Effect with Non-Linear Time History

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Abstract

Lampung, a province where Institut Teknologi Sumatera (ITERA) is located, is an area that has a high level of seismicity. This research takes a case study of the Building E ITERA which has a dilatation building concept. Due to dilatation, inter-buildings have the risk of collisions because of earthquake loads. The purpose of this study is to determine the value of joint displacement in adjacent buildings when given a dynamic load of Time History and determine whether the adjacent buildings experience a pounding effect. A Time History earthquake load data that has been matched with the Lampung region response spectrum by software is applied to the model of Building E. Building E is modeled according to the as built drawing data and the results of field checking. Structure is analyzed using software. The results of the study showed that the structure of the Building E which was loaded by Loma Prieta earthquake that has been matched would experience inter-building collisions. Further research using earthquake record data taken in areas within certain radius from ITERA is need to be conducted to obtain more accurate results.

Keywords: *Displacement, Matching, Pounding Effect, Response Spectrum, Time History*

INTRODUCTION

Infrastructure development in Indonesia is increasing rapidly nowadays, such as buildings. As a result, land availability is decreasing hence buildings are built vertically and closely spaced. When earthquake occurs at adjacent building, it can potentially experience collisions if the distance between buildings is smaller than the actual maximum vertical displacement.

Building E is an example of infrastructure that was built close at Institut Teknologi Sumatera which is used for education. This building was designed using earthquake standard [1]. The concept of dilatation is used in this building, which consists of 3 separate buildings close to each other. Structures with dilatation are vulnerable to collisions between buildings due to dynamic loads that can lead to structural failure.

Dynamic analysis can be performed on the design of earthquake resistant structures if more accurate results are needed for earthquake forces applied on the structure, as well as to determine the structural response due to earthquake loads. Analysis can be performed either elastic or inelastic. Elastic analysis can be divided into two, namely Time History Modal Analysis and Response Spectrum Analysis. In Time History analysis required earthquake acceleration data, whereas the analysis of the Response Spectrum required data from the Design Spectra so that the maximum response of each mode shape can be obtained. This elastic dynamic analysis is done by direct integration where this method is widely used because it is simpler [2].

In this study, structural behavior is analyzed in the form of structural displacement due to dynamic time history loads which has been carried out matching process with design response spectrum of Building E ITERA.

Research Purposes

This research aims to :

1. Analyze the structural failure that will occur due to non-linear time history earthquake load.
2. Determine whether the adjacent building at Building E ITERA experience pounding.

METHODOLOGY

Flow Chart

The research flow chart can be seen in Figure 1.

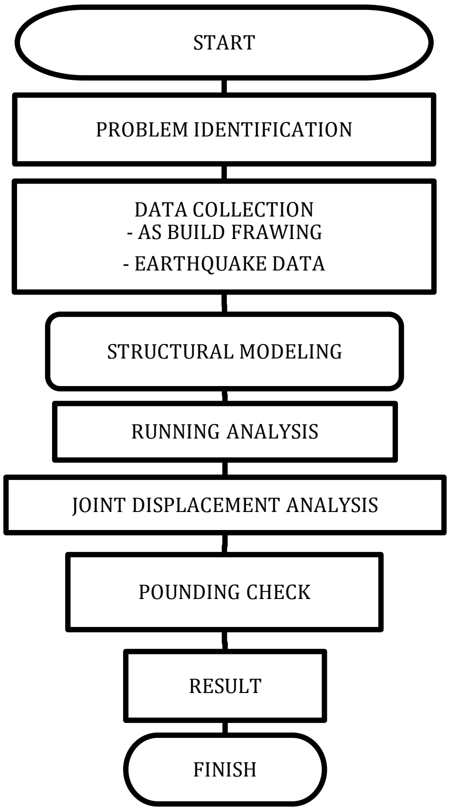

Figure 1. Flow Chart

Structural modelling by software can be illustrated in Figure 2.

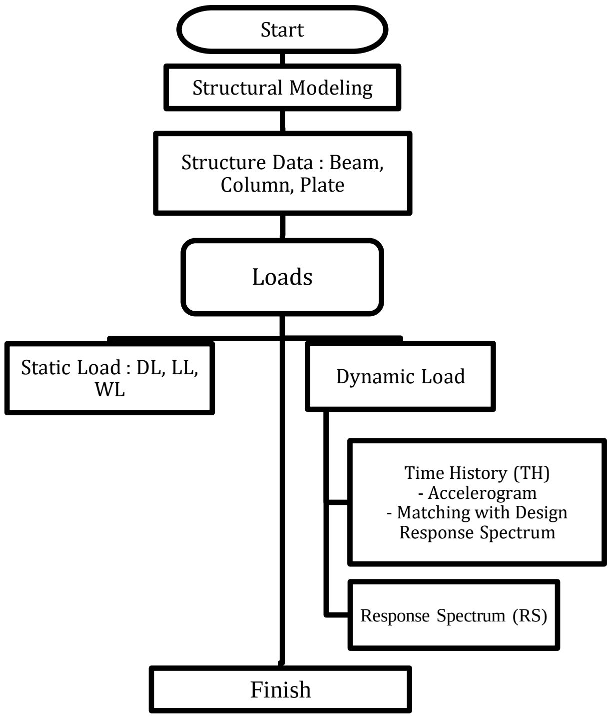

Figure 2. Structural Modeling Flow Chart

Problem Identification

Building E Institut Teknologi Sumatera has a basement because the field conditions have significant land contour differences. This building has a dilatation concept, where there are three buildings of different sizes. The two main buildings are used as lecture halls and a small building as a link between buildings. The concept of dilatation can be seen from the presence of cracks in the connecting column between adjacent buildings as in Figure 3.

Figure 3. Cracks in Building E

Data Collection

The data used in this research are as built drawings of Building E ITERA and earthquake data. The earthquake data used are design response spectrum data and time history The Loma Prieta earthquake data.

Structural Modeling

Structural modeling in this study uses structural analysis software. The Building E ITERA is modeled using a reinforced concrete system and open 3D frame with the dimensions of the structure of columns, beams and plates in accordance with the as built drawing. The data used as a reference in modeling in this study are:

1. Building Function : Lecture Room (Educational Facilities)
2. Soil Type : Medium (SD)
3. Building Location : South Lampung
4. Number of Floors : 5 (including basement and roof floor)
5. Height of Each Floor : 4,2 meter (Basement, 1, 2, 3)
6. Roof Floor Height : 3,0 meter
7. Concrete quality (fc') : 25 MPa
8. Elastic Modulus : 23500 MPa

Preliminary dimensions of Building E can be seen in Table 1.

Table 1. Structural Dimensions of Building E

Structural Element		**Dimension (mm)**	**Reinforcement (mm)**
Column	K1	500 x 500	4D22
	K3	500 x 500	4D19
Beam	Main Beam 1	350 x 700	
	Main Beam 2	250 x 500	
	Joist 1	200 x 400	
	Joist 2	200 x 300	
Floor	P1	120	

Building E ITERA is modeled as in Figure 4.

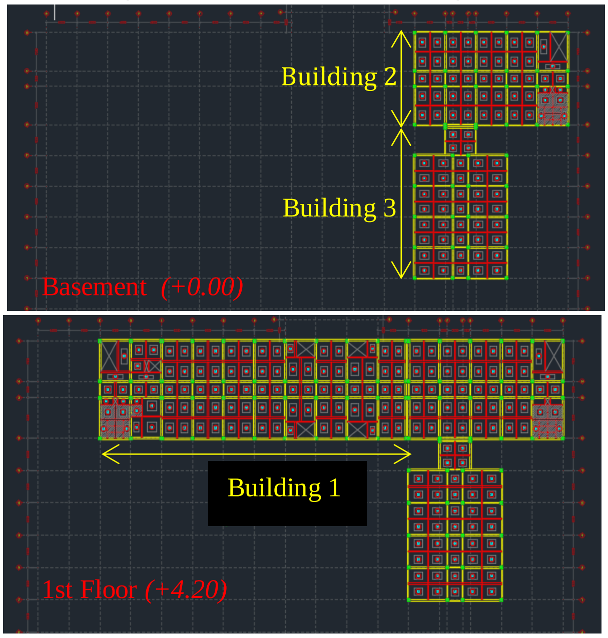

Figure 4. Separation of Building E

Separation of buildings is carried out in accordance with adjacent columns between buildings. Building 2 and Building 3 have a total height of 0 m to 15.6 m and Building 1 has a height of 4.2 m to 15.6 m.

Loads

The loading carried out in this modeling includes the load on the plate that are dead load and live load, wind load and earthquake load.

1) Floor Load
   The load that is added into the plate is SIDL (Super Imposed Dead Load) and live load according to code [3] and [4]. Life load used is 2.4 kN / m2 for offices and 100 kg / m^2 for roof life load. SIDL load used is equal to 48 kg / m^2.

2) Wind Load
   The loaded wind load has 4 directions. Winds that work in the positive x direction (WX+), winds that work in the negative x direction (WX-), winds that work in the positive y direction (WY+), and winds that work in the negative y direction (WY-). This wind load works on each side of the building structure and the wind load used is 40 kg / m^2.

In wind load modeling, for each wind it is assumed that there are compressive and suction winds that work perpendicular to the fields being reviewed. The wind coefficients regulated in [3] are as in Figure 5.

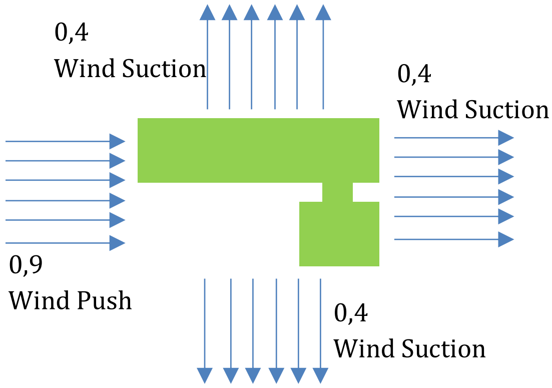

Figure 5. Wind coefficient

The coefficient also applies to the direction of each positive or negative wind direction acting on the structure. In the modeling of wind loads, for each wind load it is assumed that there are compressive and suction winds that work perpendicular to the fields being reviewed. The number of the suction and blowing wind acting on the plane of the structure is determined by multiplying the wind load by the wind coefficient.

3) Earthquake Load
a. Response Spectrum

Response Spectrum is the maximum response of the mass of Single Degree of Freedom structure both acceleration (Sa), velocity (Sv), and displacement (Sd) to the period of the structure based on specific damping and earthquake ratios [5]. According to [1], for determining the spectral response of MCE_R earthquake acceleration at ground level, a seismic amplification factor is needed at a period of 0.2 seconds and a period of 1 second with a probability of 2 percent exceeding in 50 years and expressed in decimal numbers to acceleration due to gravity. Amplification factors include vibration amplification factors related to acceleration in short period (Fa) vibrations and amplification factors related to acceleration that represent vibrations for 1 second period (Fv). The parameters of the acceleration response spectrum in the short period (S_{MS}) and 1 second period (S_{M1}) adjusted for the influence of the site classification, must be determined by the following formulation:

$$S_{MS} = F_a S_s \quad (1)$$

$$S_{M1} = F_v S_1 \quad (2)$$

Note :
Ss = Parameter of earthquake acceleration response spectrum MCE_R mapped for short period.
S_1 = Parameter of earthquake acceleration response spectrum MCE_R mapped for 1 second period.

In determining the value of S_S and S_1 can be seen based on Figure 6 and Figure 7 as follows :

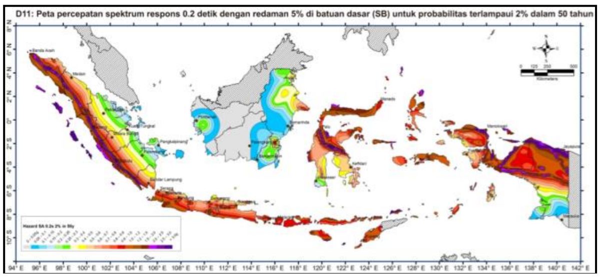

Figure 6. Ss map [6]

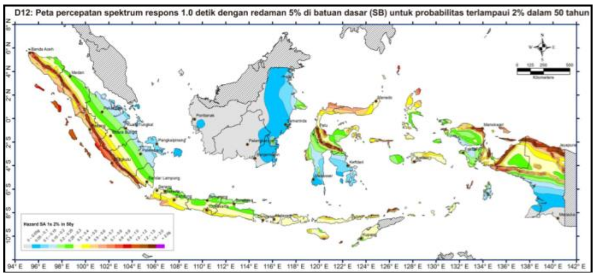

Figure 7. S_1 map [6]

To determine the site coefficients Fa and Fv, following table is used :

Table 2. Site Coefficient, Fa [1]

Site Class	Earthquake spectral response acceleration parameters (MCE_R) mapped at short periods, T = 0.2 seconds, Ss				
	Ss ≤ 0,25	Ss = 0,5	Ss = 0,75	Ss = 1	Ss ≥ 1,25
SA	0,8	0,8	0,8	0,8	0,8
SB	1,0	1,0	1,0	1,0	1,0
SC	1,2	1,2	1,1	1,0	1,0
SD	1,6	1,4	1,2	1,1	1,0
SE	2,5	1,7	1,2	0,9	0,9
SF			SS^b		

Table 3. Koefisien situs, Fv [1]

Site Class	Earthquake spectral response acceleration parameters (MCE_R) mapped at 1 second period, S_1				
	Ss ≤ 0,25	Ss = 0,5	Ss = 0,3	Ss = 1	Ss ≥ 1,25
SA	0,8	0,8	0,8	0,8	0,8
SB	1,0	1,0	1,0	1,0	1,0
SC	1,7	1,6	1,5	1,0	1,3
SD	2,4	2	1,8	1,1	1,5
SE	3,5	3,2	2,8	0,9	2,4
SF			SS^b		

The parameters of the design spectrum acceleration for the short period (S_{DS}) and the 1 second period (S_{D1}) are calculated using the following equation:

$S_{DS} = 2/3\ S_{MS}$ (3)

$S_{D1} = 2/3 S_{M1}$ (4)

For the purposes of analysis, an acceleration design response spectrum (Sa) must be made corresponding to local soil conditions, with the following Sa equation:

1) For period less than T_0, Sa is determined by equation :
   $Sa = S_{DS}\ (0,4 + 0,6\ T/T_0)$ (5)
2) For period $T_0 \leq T \leq T_s$, Sa is the same as S_{DS}
3) For period $T>T_0$, Sa is determined by equation :
   $Sa = S_{D1} / T$ (6)

T_0 dan T_s values are calculated by equation :

$T_0 = 0,2\ .\ S_{D1}\ /\ S_{DS}$ (7)

$T_s =\ S_{D1}\ /\ S_{DS}$ (8)

Data taken :
- Location : Building E ITERA
  Coordinate Lat. : -5.35941
  Coordinate Long. : 105.31149
- Soil Classification : Class D

From these data, the values of Ss, S_1, Fa, Fv, S_{DS}, and S_{D1} are as follows :

Ss = 0.718
S_1 = 0.311
Fa = 1.2264
Fv = 1.779
S_{D1} = 0.5870
S_{DS} = 0.3688

Based on the results using the data above, response spectrum can be obtained as in Figure 8.

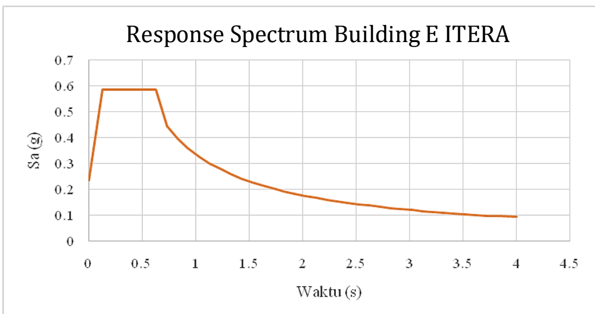

Figure 8. Response Spectrum Design Building E

b. Time History

The earthquake load used in this study is the dynamic load of time history analysis using accelerogram. Before implementing the accelerogram series in structural analysis, the data must be scaled to reduce the mismatch between characteristics and design parameters in an area based on standards or from certain hazard sites [7]. Accelerogram is used to represent earthquake motion. The accelerogram selected in the time history analysis at the level of earthquake plan must meet the requirements as stipulated in [1] sub chapter 11.1.3.2.

From earthquakes selected records produces a picture of the characteristics of ground motion, such as magnitude, distance to the center of vibration, and site classification. The selection is usually made more emphasis on efforts to obtain records that match with the response spectrum rather than the seismological parameters. Therefore, these records are selected based on consideration of the magnitudes of a strong ground motion, such as peak ground acceleration (PGA = peak ground velocity), peak ground velocity (PGV = peak ground velocity), and duration in accordance with the design response spectrum [8].

The following time history data along with response spectrum that have been made to the matching process.

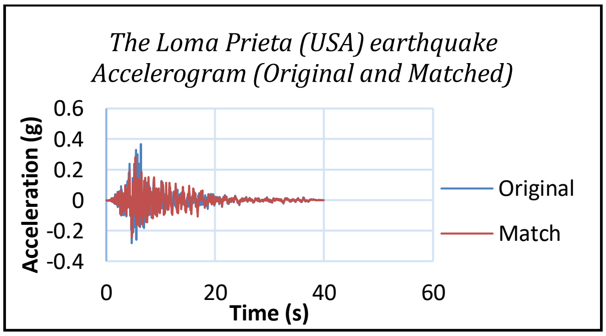

Figure 9. The Loma Prieta Earthquake Accelerogram

Pounding Effect

Collision between adjacent buildings due to a stimulus or seismic load are called the pounding effect [9]. This phenomenon has been found more than 20 years ago. A high level of complexity requires a thorough knowledge and how the building reacts if given a different magnitude but the strength of the structure remains the same. Generally, pounding categorized into floor-to-floor and floor-to-column pounding [10].

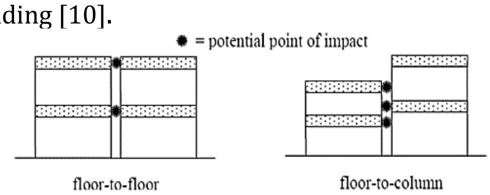

Figure 10. Pounding Category [10]

The main reason for seismic pounding is the determination of the distance between buildings [11]. The response of the external forces of an adjacent building is seen from the following conditions:

a. When the gap between adjacent buildings is inadequate.
b. When the building has a separation space but is connected to one or more members.
c. When adjacent buildings have different dynamic properties such as mass, height, orientation and geometry. If the dynamic properties are similar, pounding will not occur even if the distance is zero.
d. When the center of mass of the adjacent building is not axial.

Pounding might occur due to structural irregularities [12]. For example, eccentricity between center of mass causes torque. If the structure moves regularly, there will be no collisions between adjacent buildings (Figure 11). Concrete structure have a greater risk because poor design and construction implementation.

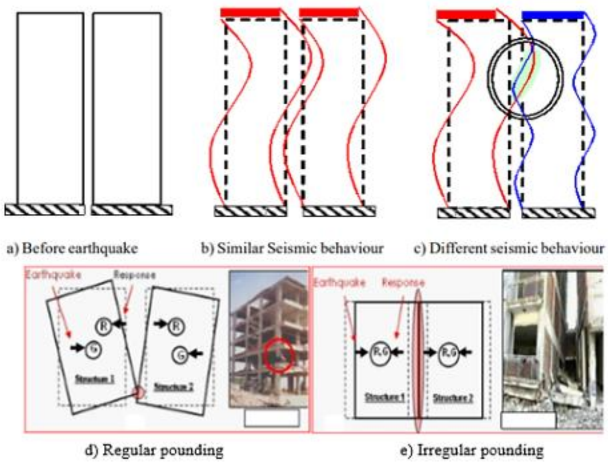

Figure 11. Seismic behaviour of adjacent buildings [12]

In accordance with regulation [13], the minimum separation distance is calculated by Absolute Sum (ABS) or Square Root of Sum of Squares (SRSS) as follows:

$$S = U_A + U_B \tag{9}$$

$$S = \sqrt{(U_A^2 + U_B^2)} \tag{10}$$

which :
S = separation distance
U_A, U_B = displacement each building A and building B.

Load Combination

Combination of loading used is the ultimate load in accordance with [4] chapter 2 as follows:

1. 1,4DL
2. 1,2DL +1,2 SIDL +1,6 LL+0,5 (Lr atau H)
3. 1,2DL +1,2 SIDL +1,6 (Lr atau H) + LL
4. 1,2DL +1,2 SIDL +LL+ W+0,5(Lr atau H)
5. 1,2DL +1,2 SIDL +E +LL
6. 0,9DL + W
7. 0,9DL + E

In this study, the load combination that does not use wind and earthquake loads will remain the same, while the combination that uses wind and earthquake loads will have varying coming directions therefore creating more combinations. The combination of loading becomes as follows:

1. 1,2DL +1,2SIDL +LL+ (WX+) +0,5Lr
2. 1,2DL +1,2SIDL +LL+ (WX-) +0,5Lr
3. 1,2DL +1,2SIDL +LL+ (WY+) +0,5Lr
4. 1,2DL +1,2SIDL +LL+ (WY-) +0,5Lr
5. 1,2DL +1,2SIDL +1ExH +0,3EyH+1EV +LL
6. 1,2DL +1,2SIDL +1ExH -0,3EyH+1EV +LL
7. 1,2DL +1,2SIDL -1ExH +0,3EyH+1EV +LL
8. 1,2DL +1,2SIDL +1EyH +0,3ExH+1EV +LL
9. 1,2DL +1,2SIDL +1EyH -0,3ExH+1EV +LL
10. 1,2DL +1,2SIDL -1EyH +0,3ExH+1EV +LL
11. 0,9D + (WX+)
12. 0,9D + (WX-) 13. 0,9D + (WY+)
14. 0,9D + (WY-)
15. 0,9D + 1ExH +0,3EyH+1EV
16. 0,9D + 1ExH -0,3EyH+1EV
17. 0,9D - 1ExH +0,3EyH+1EV
18. 0,9D + 1EyH +0,3ExH+1EV
19. 0,9D + 1EyH -0,3ExH+1EV
20. 0,9D - 1EyH +0,3ExH+1EV
21. TH combination

Inter-Story Drift

Based on [1] sub chapter 7.12.3 relating to the determination of design inter-story drift (Δ) must be calculated as the difference in deflection at the center of mass at the top and bottom levels reviewed. Deflection of the center of mass at the level (δx) (mm) must be determined according to the following equation:

$$\delta_x = \frac{C_d \ \delta_{xe}}{I_e} \tag{11}$$

Note:
C_d = deflection amplification factor
δ_{xe} = deflection at the required location determined by elastic analysis
I_e = earthquake importance factor

Plastic Hinge

Plastic hinge mechanism formed at the ends of the beam and at the base of the column produces a stable hysteresis behavior. The formation of plastic hinge should be dominated by flexural behavior. Plastic hinge can occur in a MDOF structure. Plastic hinge occurs gradually starting from forming at the ends of the beam evenly until it forms at the bottom of the column before the building collapse.

The relationship of force and displacement can be categorized into several criteria that indicate plastic hinge behavior [14]. The relationship of force and displacement can be described as Figure 12:

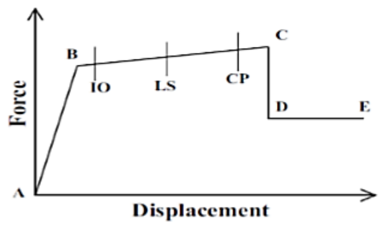

Figure 12. Plastic joint [15]

Each point determines the behavior of displacement vs force of the plastic hinge. Point A is the origin, B is the yielding point, C is the maximum point. D and E are measures of residual strength and displacement capacity. For the other three points namely IO, LS, and CP are structural behaviors that occur. IO (immediate occupancy) means the condition when there is no significant damage to the structure where the strength is about the same as the condition before an earthquake occurs, LS (life safety) means the condition when damage to structural components occurs, stiffness decreases, but still has sufficient threshold against collapse and CP (collapse prevention) means the condition of the occurrence of significant damage to the structural components, therefore strength of the structure is reduced significantly.

RESULTS AND DISCUSSION

Matching Process

The matching process is carried out by multiplying the Loma Prieta earthquake record with a number so that the spectrum response of the accelerogram approaches the SNI spectrum response for Lampung Province with medium soil.

The multiplication data which are acceleration and time data is drawn using the Seismosignal software so that the plot, called AGM02, can be obtained [16]. The output data from the seismosignal is an acceleration vs time data that has been adjusted to the response spectrum of Building E ITERA. Comparison between spectrum response curves based on SNI and time history that has been matched can be seen in Figure 13.

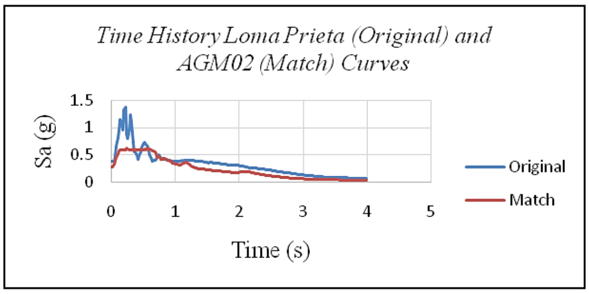

Figure 13. Time History and Response Spectrum Matching Curves

Plastic Hinge

In the process of modeling using software, auto-hinge parameters are given in each frame (beams and columns) at 0.5 and 0.95 of the length of each frame. The software output will be seen in the color indicator according to the interpretation in Figure 14.

Table 4. Structural Damage Level due to the Formation of Plastic Hinge [15]

Note	Symbol	Commentary
B	(pink)	Shows the linear limit followed by the first yield in the building
IO	(dark blue)	Minor or insignificant damage to the structure
LS	(light blue)	Damage to moderate levels cause reduced structural rigidity, but still has a large threshold to collapse
CP	(green)	Severe damage to the structure until the stiffness was reduced significantly
C	(yellow)	Maximum limit of shear force that is still able to be resisted by the building
D	(orange)	There is a great structural strength degradation so that structural condition is stable and almost collapse
E	(red)	Structure is not able to resist shear force (Total failure)

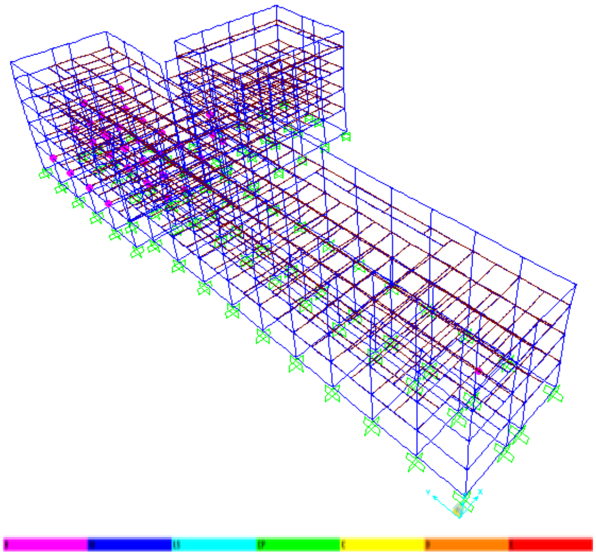

Figure 14. Plastic Hinge at Building E Caused by Loma Prieta Earthquake

Plastic hinges in Figure 14 at the last second of time history earthquake load have formed at the end of the beam (shown in pink). Therefore, the structure is still in an elastic state (point B in Figure 12 and Table 4). In other words, the structure does not suffer damage due to the earthquake load of nonlinear time history.

Inter-buildings Deflection

Three separate buildings with the concept of dilatation in building E modelled in a file. Displacement analysis is at the joint in each floor that becomes a meeting between buildings. The meeting point between buildings is defined as K1 to K4 which is the meeting between Building 1 and 2, and K5 to K6 which is the meeting between Building 2 and 3.

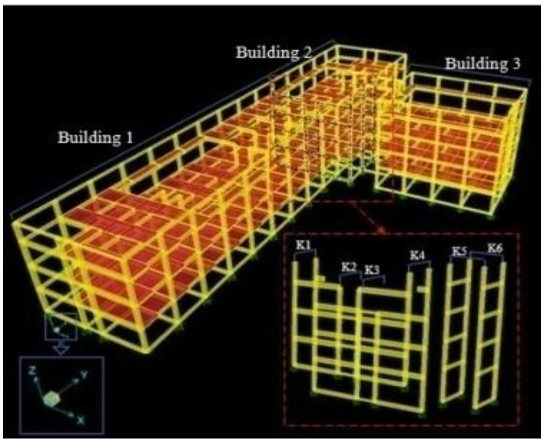

Figure 15. Meeting Points Definition Between Adjacent Buildings

Figure 15 indicates that there are 4 meeting zones between building 1 and building 2 along the Y axis, namely K1, K2, K3 and K4. Between buildings 2 and 3 towards the X axis, there are 2 meeting zones namely K5 and K6.

Building E ITERA consists of 3 buildings that are built close to the distance between buildings 50 mm from the outer side of the column or 550 mm from the column center line.

The inter-buildings displacement graphs (Figures 16 and 17) are adjusted to the distance in the as built drawing of Building E ITERA that is 50 mm, so that the movement of the joint displacement can be seen. Using equation (17) the displacement value can be seen from Table 5 to Table 8.

Table 5. Max. Displacement Loma Prieta Earthquake at Building 1 in Y Direction (t = 4.85 s)

K1				K3			
Building 2		Building 1		Building 2		Building 1	
No. Joint	dy(mm)	No. Joint	dy(mm)	No. Joint	dy(mm)	No. Joint	dy(mm)
542	2.6713	145	40.2310			157	38.3127
375	-3.3692	71	29.3691	374	-3.1912	70	29.3762
101	-14.7334	43	13.1882	100	-14.6405	42	13.2454
310	-31.4729	193	0	309	-31.3882	223	0
881	-50			879	-50		
K2				**K4**			
Building 2		Building 1		Building 2		Building 1	
No. Joint	dy(mm)	No. Joint	dy(mm)	No. Joint	dy(mm)	No. Joint	dy(mm)
		156	38.1779	559	3.12853	146	40.5439
373	-3.2292	69	29.3593	376	-3.14582	72	29.4695
99	-14.6721	34	13.2377	102	-14.5446	44	13.2337
308	-31.4069	222	0	311	-31.3618	224	0
880	-50			878	-50		

Table 6. Max. Displacement Loma Prieta Earthquake at Building 2 in Y Direction (t = 5.0 s)

K1				K3			
Building 2		Building 1		Building 2		Building 1	
No. Joint	dy(mm)	No. Joint	dy(mm)	No. Joint	dy(mm)	No. Joint	dy(mm)
542	19.7095	145	19.3978			157	17.5404
375	10.4257	71	14.2918	374	11.4825	70	14.3378
101	-5.4879	43	6.5044	100	-4.7497	42	6.55801
310	-27.2388	193	0	309	-26.824	223	0
881	-50			879	-50		
K2				**K4**			
Building 2		Building 1		Building 2		Building 1	
No. Joint	dy(mm)	No. Joint	dy(mm)	No. Joint	dy(mm)	No. Joint	dy(mm)
	-50	156	17.4302	559	22.1945	146	19.6894
373	11.1874	69	14.3378	376	12.1919	72	14.4242
99	-4.9643	34	6.5478	102	-4.2019	44	6.5655
308	-26.824	222	0	311	-26.5633	224	0
880	-50			878	-50		

There are two conditions of maximum displacement which are reviewed against the Loma Prieta earthquake load. The review are when the maximum displacement occurs in Building 1 (Table 5), and the maximum displacement occurs in Building 2 (Table 6). The maximum displacement in Building 1 occurs at 4.85 seconds of time history earthquake load, while the maximum displacement in building 2 occurs at the 5 seconds. The maximum displacement in Table 5 and Table 6 can be seen in the graph in Figure 16.

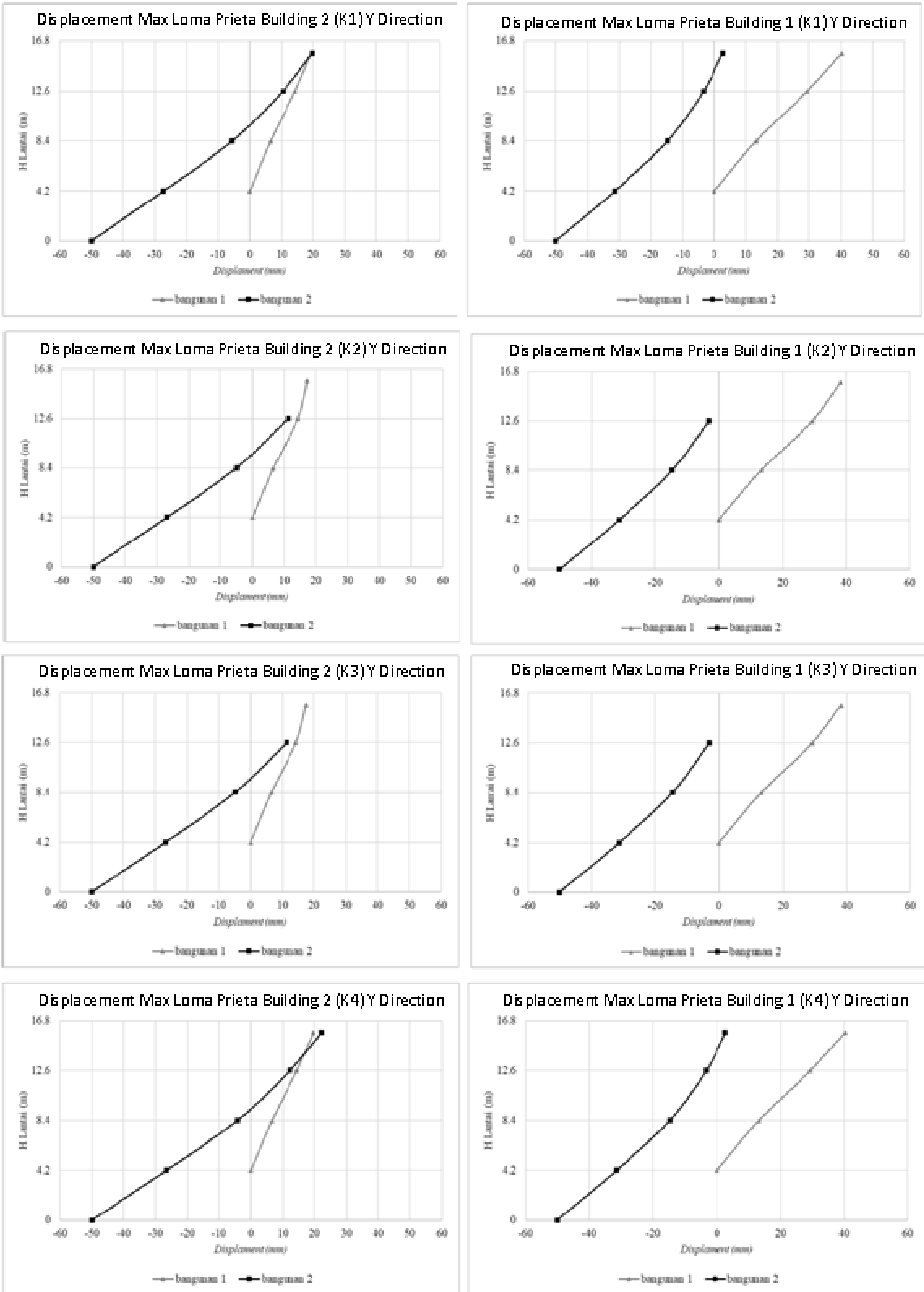

Figure 16. Displacement of Building 1-2, Loma Prieta Earthquake

Figure 16 indicates that the pounding effect occurs between Building 2 and Building 1 with the displacement difference of 0.312 mm in the K1 meeting zone in Y direction. Pounding effect also occurred between Building 2 and Building 1 with a displacement gap of 2.505 mm in Y direction at the K4 meeting zone. Both occur within 5 second of the time history earthquake load.

Table 7. Max. Displacement Loma Prieta Earthquake at Building 2 in X Direction (t = 4.93 s)

K5				K6			
Building 2		Building 3		Building 2		Building 3	
No. Joint	dx(mm)	No. Joint	dx(mm)	No. Joint	dx(mm)	No. Joint	dx(mm)
557	1.3316	558	47.2398	742	20.6151	743	70.6151
368	-1.9376	372	46.7900	739	9.10704	740	59.1070
94	-13.8734	98	35.2519	729	-7.0458	730	42.9542
303	-30.143	307	19.4451	736	-28.7351	737	21.2649
886	-50	882	0	760	-50	759	0

Table 8. Max. Displacement Loma Prieta Earthquake at Building 3 in X Direction (t = 5.01 s)

K5				K6			
Building 2		Building 3		Building 2		Building 3	
No. Joint	dx(mm)	No. Joint	dx(mm)	No. Joint	dx(mm)	No. Joint	dx(mm)
557	-4.4426	558	40.8215	742	26.1755	743	76.1755
368	-6.9739	372	41.0029	739	13.5778	740	63.5778
94	-17.6051	98	30.8952	729	-3.7117	730	46.2883
303	-32.0192	307	17.213	736	-26.9043	737	23.0957
886	-50	882	0	760	-50	759	0

Similar to Table 5 and Table 6, there are two criteria under consideration for the maximum displacement of Building 2 and Building 3. The review are when the maximum displacement occurs in Building 2 (Table 7), and the maximum displacement occurs in Building 3 (Table 8). The maximum displacement in Building 2 occurs at 4.93 seconds of time history earthquake load, while the maximum displacement in building 3 occurs at the 5.01 seconds. The maximum displacement in Table 7 and Table 8 can be seen in the graph in Figure 17.

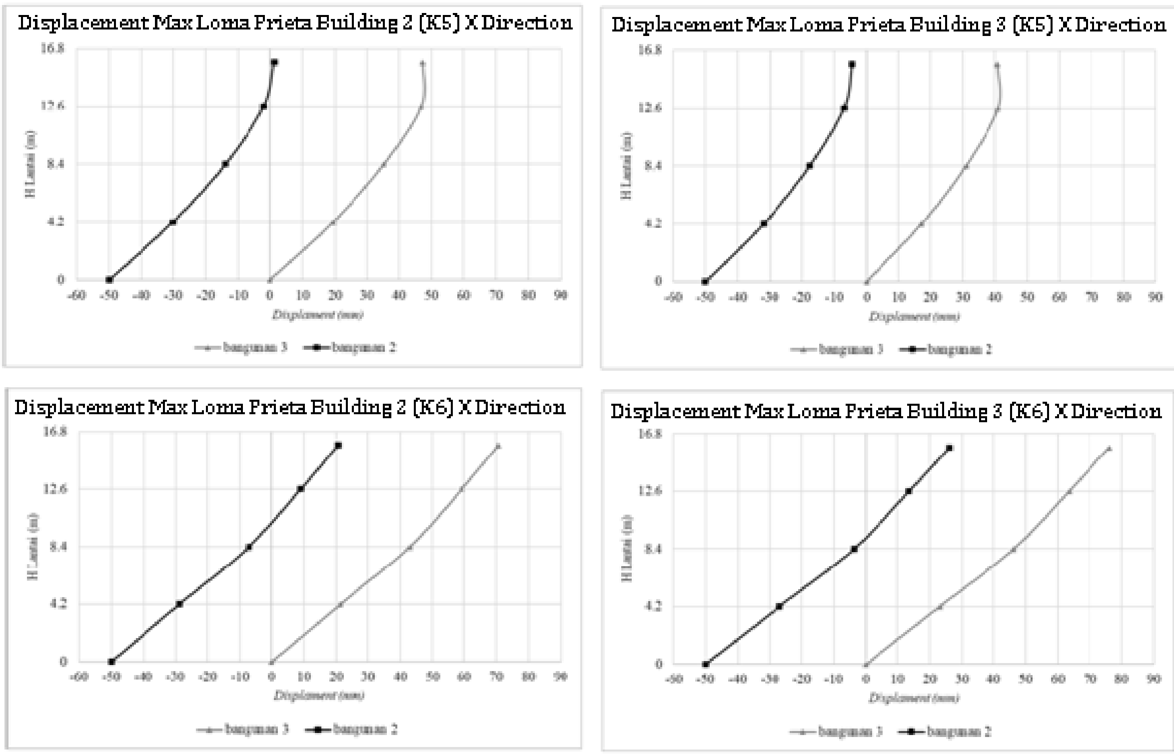

Figure 17. Displacement of Building 2-3, Loma Prieta Earthquake

Figure 17 demonstrates that in their maximum displacement at Building 2 and Building 3 in the K5 and K6 meeting areas, there is no pounding between Building 2 and 3. The building distance of 50 mm is therefore secure enough.

CONCLUSION

The conclusions that can be obtained from this study are: (a) plastic hinge have been formed in Building E due to the Loma Prieta earthquake load that has been matched with the response spectrum of local area. However, the plastic hinge formed are still in the elastic limit. Hence, the structure does not suffer damage due to the earthquake load of nonlinear time history; (b) by a building spacing of 50 mm, the pounding effect occurs at the meeting point of Buildings 1 and 2 due to the Loma Prieta earthquake load that has been matched with the response spectrum of local area. For the meeting points between Building 2 and 3 there was no pounding effect; (c) further research using earthquake record data taken in areas within certain radius from ITERA is need to be conducted to obtain more accurate results.

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