Structure Behavior Analysis with Time History Levelling Method (Case Study in Building E ITERA)

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Abstract. Mitigation of structural failure can be based on analyzing the behavioral of the structure by giving a nonlinear dynamic earthquake load of time history. Because Indonesia doesn't have any data of time history record, it is necessary to match the time history data with the design spectrum response. Gedung E Itera is modeled on reinforced concrete structures with open frame models like as built drawing. Then can be evaluated the structural behavior that is like mass participation, plastic design, displacement, rotation, and base shear with time history load that will be levelling (levelling time history). With levelling the time history load , it can be found the first structural part that collapses and the maximum load that can be retained by the structure for one of the mitigation effort. The analysis uses a nonlinear dynamic time history analysis with the most dominant combination is earthquake combination. The results of this study show structural performance, the location of structural failure, and the maximum load that can be retained by the reinforced concrete structure of gedung E Itera.
Keywords : Structural performance, time history.

1 Introduction

The acceleration of development in the field of technology by the central government for readiness for human resources in Sumatra is the forerunner to the formation of the Sumatra Institute of Technology (ITERA) which is the only state institute on the island of Sumatra. Since it was inaugurated in 2014, the Institute of Technology in Sumatra has had several main buildings used both for lecture halls and campus administration rooms. One of the buildings that was inaugurated in 2017 is ITERA Building E which is a building grant from the city government of Bandar Lampung to the Sumatra Institute of Technology to support the smoothness of lectures.

The acceleration of development really requires maintenance control especially for the existing building structures that have been built. In terms of structural maintenance, generally checking conditions in the field are often carried out, besides the method of predicting structural behavior can also be used. This prediction method by looking at the behavior of the inner structure receives the maximum load to see which part of the structure has a gradual failure. The method used in this study is Leveling Time History, which is by gradually increasing the Aog or PGA Time History until the structure begins to fail until the overall structural failure occurs. Earthquake load Time History is matched with the Spectra Response of the Bandar Lampung region in order to cause a nonlinear earthquake load that approaches the predicted real earthquake that will occur. This is done to be able to take preventive measures so that the life of the building can run longer.

2 Literature Review

2.1. General review

2.1.1. Reinforced Concrete Structure

Concrete is a mixture of portland cement or other hydraulic cement, coarse aggregates, fine aggregates, and water, with or without additional mixtures forming solid masses (SK SNI T-15-1991-03). This mixture will form an artificial stone whose strength varies depending on the planned mixture.

Reinforced concrete itself is a combination of two materials, namely: concrete and steel (reinforcement) which in its planning must refer to the standards in Indonesia SNI 2847-2013 concerning the requirements of structural concrete for buildings. The advantages of concrete material are strong compressive resistance, while steel (reinforcement) is a very good material to resist tensile and shear. The combination of these two materials is expected to be able to withstand tensile forces, compressive forces and shear forces so that a building structure remains strong and safe. The use of reinforced concrete in building structures includes: foundations, beams, columns, plates, shearwall walls.

2.1.2. General Concept

Ductility is the ability of a building structure to experience large post elastic displacements repeatedly and back and forth due to earthquake load above the earthquake load which causes the first melting while maintaining sufficient strength and stiffness so that the building structure remains standing even though it is in on the verge of collapse (Budiono and Supriatna, 2011: 17). Deformed structures can mean elongated, shortened, and bent. The ductility factor of a building structure is the basis for determining the seismic load that works on the building structure, therefore achieving the level expected to be well guaranteed. This can be achieved if the beam has to melt before the damage occurs in the column (the concept of strong column weak beam).This means that due to the influence of the earthquake plan, plastic joints in the building structure are only at the ends of the beam and on the foot of the column.

In a more detailed sense ductility is the ability of a structure to experience large post elastic displacements repeatedly and back and forth due to an earthquake load which causes the first melting, while maintaining sufficient strength and stiffness, so that the structure remains standing even when it is at condition of the threshold of collapse. In planning earthquake resistant buildings, the formation of plastic joints that are expected to occur in the structure when a large earthquake occurs needs to be controlled and its location is limited to the structural components. In the frame structure it would be better if the dissipation of earthquake energy through melting (plastic joints) on the beam and column components are expected to provide strength, stiffness and stability when holding forces acting through bending, shearing and axial action. The space frame system inside the structural components and their joints withstand forces acting through flexural, shear and axial action is called the Moment Resisting Frame System.

2.2. Structure Dynamics

Structural dynamics is one part of mechanics that specifically discusses structural responses to dynamic loads, for example due to earthquakes. In the discussion of structural dynamics, the load and response structure is not only determined by the direction, location, and magnitude, but also by the time variable. In particular, the magnitude of the structural response in the form of internal forces is a function of time, as a form of response to disturbances or external loads, whose formula is determined by the parameters of the structure in question, the mass, stiffness and attenuation affecting the vibration experienced by the structure.

2.2.1. Degree of Freedom (Degree of Freedom)

According to Widodo (2001) the degree of freedom is the degree of independence needed to state the position of a system at any time. If a point is observed experiencing a horizontal, vertical and sideways displacement, for example, the system has 3 degrees of freedom.

According to Mario Paz (1996), in general a continuous structure has an infinite number of degrees of freedom. But with the process of idealization or selection, an appropriate mathematical model can reduce the number of degrees of freedom into a discrete number and for some circumstances can become a single degree of freedom.

2.2.2. Degree of Single Freedom (Single Degree of Freedom)

A mass system that moves in one direction, namely the horizontal direction, is called a single degree of freedom (SDOF) system. In SDOF systems, the structure is modeled with a single mass and a single displacement coordinate. The elements that influence this system are mass, structural stiffness, attenuation, and external force. The following in Figure 1 is an example of a mathematical model of SDOF.

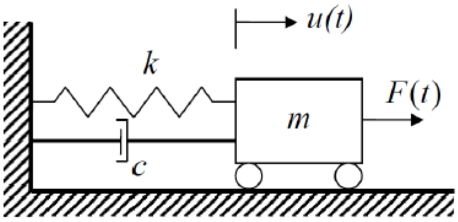

Figure 1 Mathematical Model of SDOF
Source: Dynamics of Theory & Calculation Structure, Mario PAZ (1996)

2.2.3. Many Degrees of Freedom (Multi Degree of Freedom)

Mathematical models that represent a system of multiple degrees of freedom (MDOF) can be seen in Figure 2.

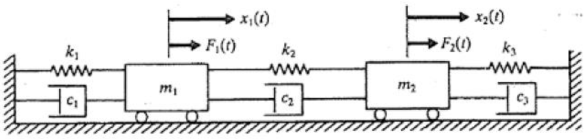

Figure 2 Mathematical Model of MDOF
Source: Dynamics of Theory & Calculation Structure, Mario PAZ (1996)

A structure has a natural frequency as much as the degree of freedom it has and if the dynamic load received by the structure has a frequency close to the natural frequency of the structure, there will be a resonance which will result in collapse or collapse of the structure.

2.3. Irregular and Irregular Building Structure

According to SNI 1726-2012 article 7 structure of buildings must be classified as regular building structures and irregular building structures. For regular building structures procedures can be usedequivalent static analysis and for irregular building structures, seismic effects of the plan must be reviewed as the effect of dynamic loading. The analysis that can be used for irregular building structures is spectrum response variance analysis and dynamic response analysis of linear and nonlinear time histories. In this researchis used irregular building structure planning with the analysis used is a time history dynamic response analysis.

3 Research Methodology

3.1. General description

The method of predicting structural behavior can also be used in terms of structural maintenance. This prediction method by looking at the behavior of the inner structure receives the maximum load to see which part of the structure has a gradual failure. The method used in this study is Leveling Time History, which is by gradually increasing the Aog or PGA Time History until the structure begins to fail until the overall structural failure occurs. Earthquake load Time History is matched with the Spectra Response of the Bandar Lampung region in order to cause a nonlinear earthquake load that approaches the predicted real earthquake that will occur. This is done to be able to take preventive measures so that the life of the building can run longer.

3.2. Research sites

In this study researchers conducted a case study in ITERA Building E which was inaugurated in 2017. Building E ITERA which is a building grant from the city government of Bandar Lampung to the Sumatra Institute of Technology.

3.3. Research Data Collection Method

Data collection methods in this study are divided into two ways, namely:
1. Primary data
Primary data is obtained using architectural drawings as built from Building E ITERA.
2. Secondary data
The nonlinear earthquake load used is the Time History earthquake which has been matched with the response of the Lampung region spectra.

3.4. Data Processing Analysis

The method used in this study is Leveling Time History, which is by gradually increasing the Aog or PGA Time History until the structure begins to fail until the overall structural failure occurs. In this study structural modeling uses structural analysis software, SAP2000 version 14.

From every failure that occurs, the behavior of the structure is analyzed in parameters where the structure fails. Then the part of the structure that first fails from any increase in Aog or PGA Time History is analyzed so that it can be concluded which structure is critical as a mitigation step if the structure experiences nonlinear loads.After running leveling time history results in a complete structural failure, the researcher can deduce the maximum nonlinear earthquake load that can be retained in the structure of the E ITERA Building.

3.5 Research Flow Chart

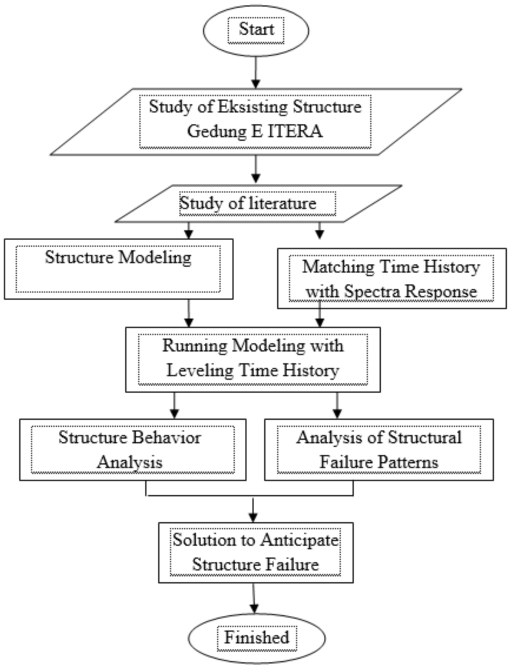

Figure 3 Flowchart of Research Methodology

4 Results and Discussion

4.1. Structure Description

The reinforced concrete structure design is adapted to the as built drawing of gedung e ITERA. The structure is modeled as shown in Figure 4, where the structure is divided into 2 main parts. The first structure that extends the y direction and the second structure is a smaller structure and is next to the first structure.

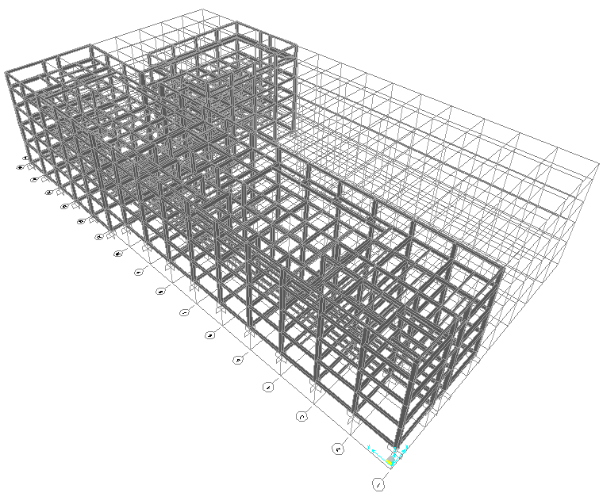

Figure 4 3D View of the Concrete Building Structure of the E ITERA Building

4.2. Description of loading

One of the objectives of this study is to evaluate the performance of the behavior of the structure being reviewed. Therefore, structure that have been modeled will be given loads to see the performance of the structure. At the time of modeling the structure of the beam and column is modeled as open frame so that the load that works will be directly received by the structural elements.

The most dominant combination of loading in this study is earthquake load. The earthquake load that will be received by the structure is an earthquake load time history that will be modified later. In addition to earthquake loads, the combination of loading used in the case study of the ITERA E building is a burden that is regulated in the applicable loading regulations in Indonesia based on SNI 03-1726-2012. And for the earthquake load that will be used in this structural analysis is the time history that will be adjusted to the response of the spectrum of the region and the condition of the soil where the structure is located.

4.2.1. Time History

Time history or time history data is obtained from records of earthquake accelerometers when an earthquake occurs in an area. In this case the Loma Prieta E-W accelerogram (Figure 5) was recorded on October 17, 1989 in the northern part of California.

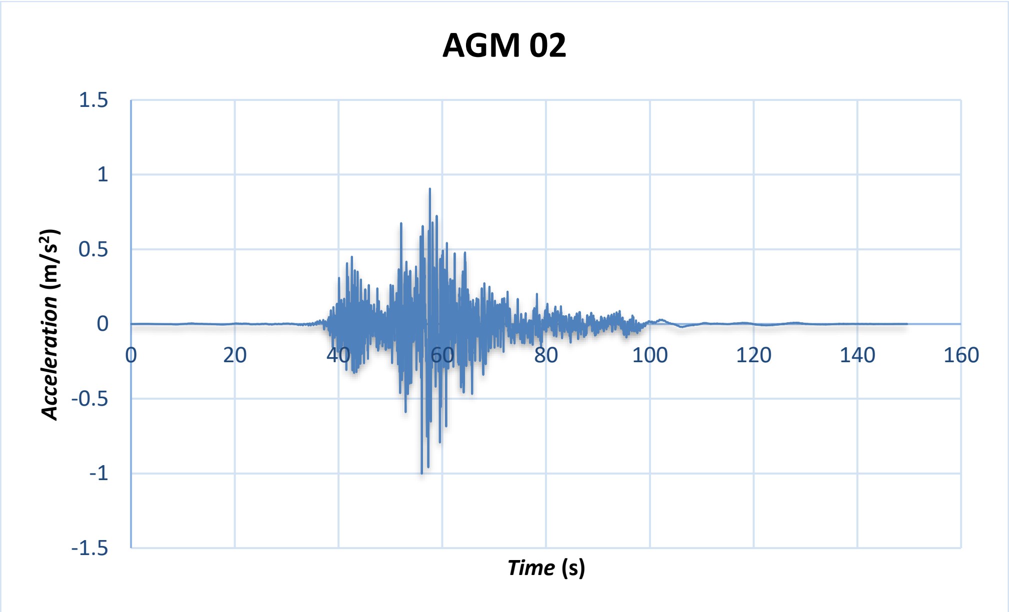

Figure 5 AGM02 Acceptor (Loma Prieta E-W, October 17, 1989)

This acceleration data will be adjusted in response to the design spectrum for the South Lampung region.

4.2.2. Spectrum Response

Analysis of calculation of response spectrum design takes data from Puskim Ministry of Public Works PUPR website (puskim.pu.go.id) in the form of Ss, S1, Fa, and Fv data which then after calculation in accordance with SNI-1726-2012 with earthquake source and hazard maps 2017 will produce a spectrum response curve.

The results of the calculation of the design spectrum response can be seen in Figure 6 that is the spectrum response curve with abscissa time and ordinate acceleration.

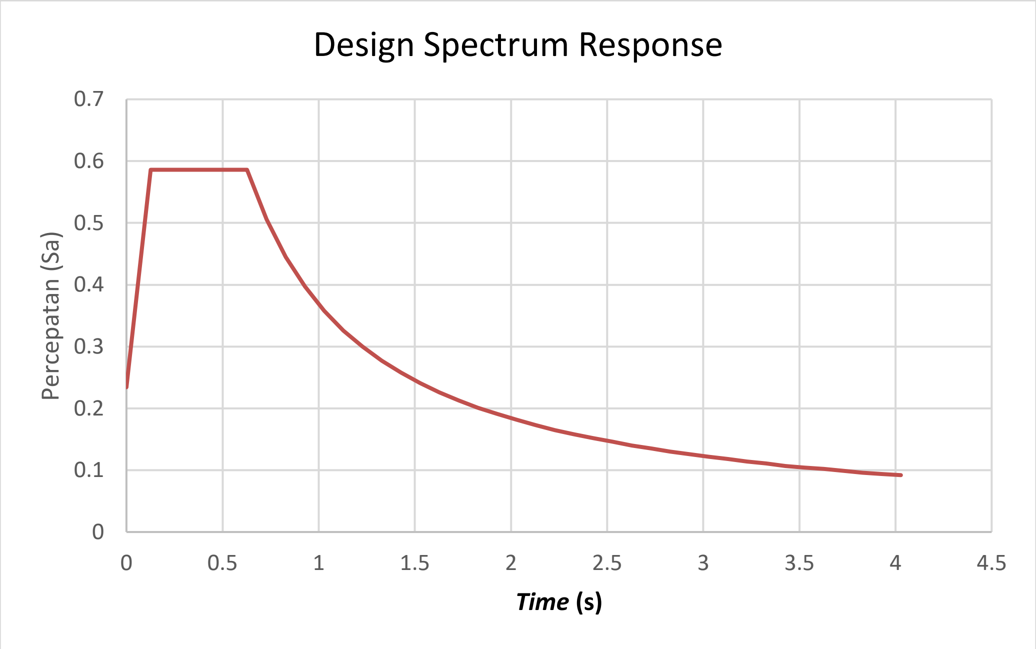

Figure 6 Spectrum Response Curve

4.2.3. Matching and Leveling Process

In SNI 1726-2012 it is stated that ground motion must be scaled in such a way that in the range of 0.2T to 1.5T the average value of acceleration is not less than the ordinate value associated or adjacent to the design spectrum response. In this study matching earthquake recordings were first multiplied by a number so that the spectrum response of the accelerogram approached the spectrum response based on SNI for the Lampung region with soft soil conditions.

Multiplication data which is the acceleration and time data will be drawn using Seismosignal software from Seismosoft so that the AGM02 recording plot results can be obtained as shown in Figure 7. The output data from the seismosignal is the acceleration of the time that has been adjusted with the spectrum response of E building location ITERA is located. Display of comparison between spectrum response curve based on SNI and time history that has been matched can be seen in Figure 8.

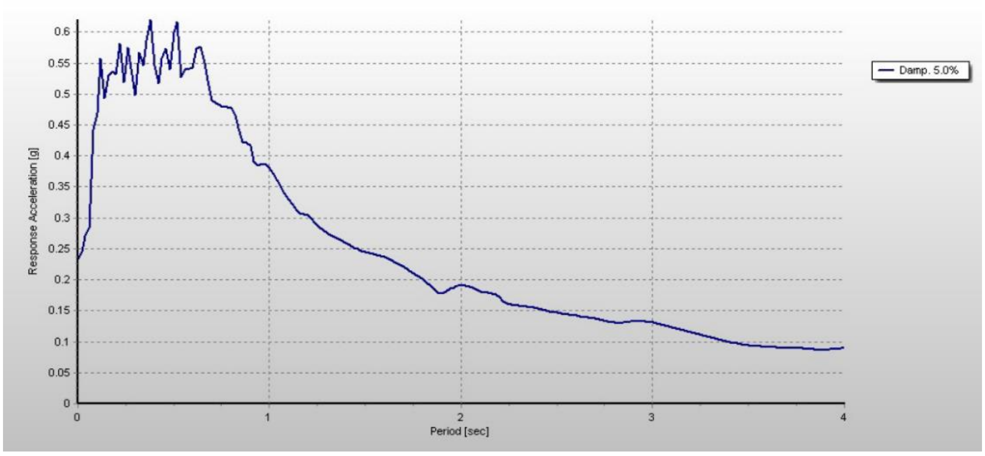

Figure 7 Spectrum Response Image Output from Seismosignal Software

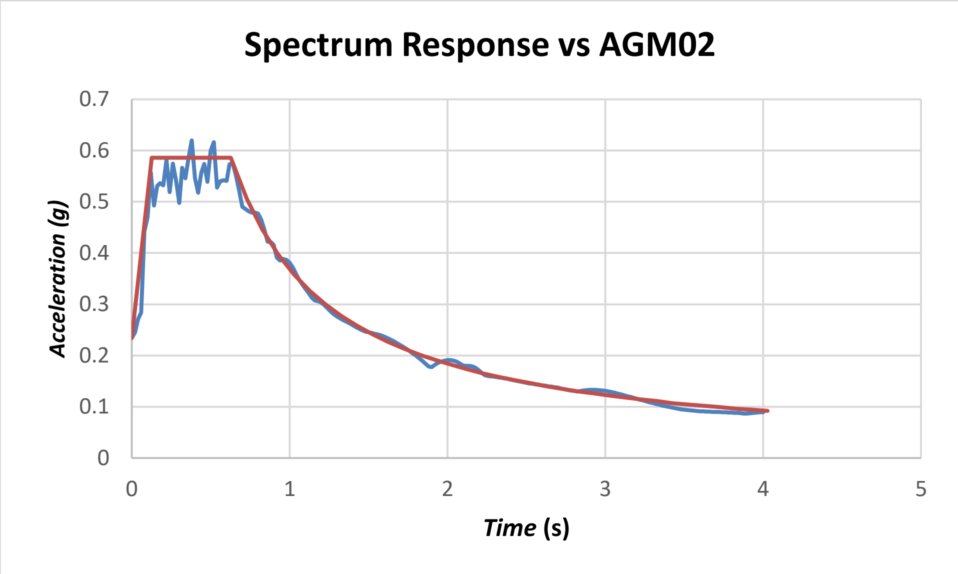

Figure 8 Comparison of Time History and Spectrum Response

In the analysis using SAP2000 earthquake record AGM02 that has been matched with the response spectrum of Lampung region will be increased Aog (earthquake acceleration) several times and examples can be seen in Figure 9. Increased initial earthquake acceleration aims to see the performance (structural behavior) of gedung e ITERA in every increase in Aog.

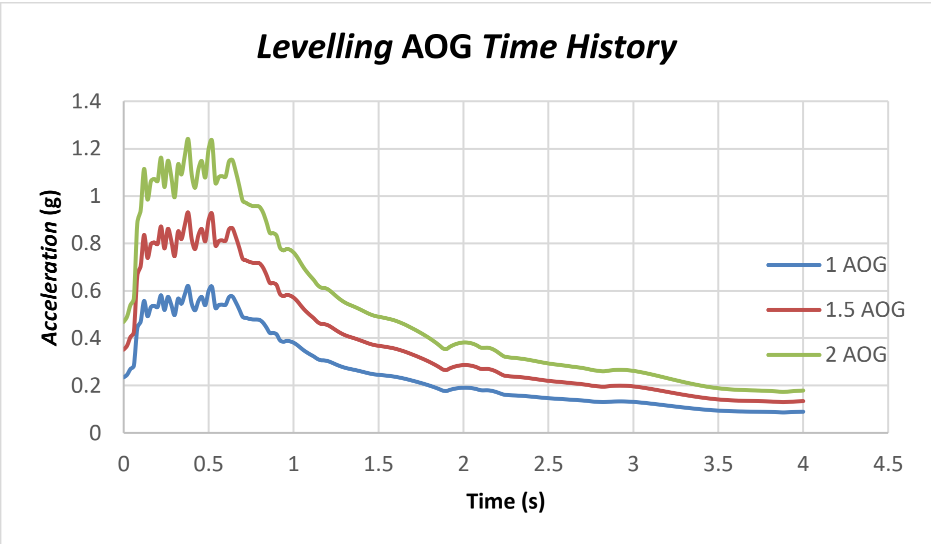

Figure 9 Leveling Results of Aog Time History

4.3. Structural Analysis

Structural analysis is carried out using the help of SAP2000 application to obtain the results of structural behavior that will be reviewed in the next section, namely mass participation, check plastic design, displacement, rotation, and base shear. After modeling on SAP, structural analysis will be carried out in this study, the structural analysis stage is carried out in accordance with the initial acceleration of earthquake time history that has been matched with the design spectrum response. The analysis used is time history nonlinear dynamic analysis.

4.3.1. Check Mass Participation

Checking the participation of the masses was carried out to find out about the mode conditions (vibrational variance) where some structures experienced mass participation reaching 90% (in accordance with SNI 1726-2012). Where the mode is used as a reference for determining the dominant motion pattern. The results of the vibration range for the concrete structure are listed in Table 1.

Table 1 Participation of Concrete Structure Mass

Moode	T (second)	Mass Participation (%)								
		ix	UX	Shape	iy	UY	Shape	iz	UZ	Shape
1	0.935855	51.0620	51.062		0.042	0.042		67.209	67.209	
2	0.875618	0.1990	51.261		17.046	17.089		5.18	72.389	
3	0.771157	0.0800	51.341		0.344	17.432		1.119	73.508	
4	0.754685	0.0009	51.342		33.462	50.894		1.515	75.023	
5	0.676395	0.0051	51.347		24.614	75.509		0.16	75.183	
6	0.63096	26.1480	77.495		0.00098	75.51		3.623	78.806	
7	0.614702	0.1110	77.606		0.157	75.667		1.731	80.537	
8	0.58907	1.5840	79.19		0.025	75.692		1.648	82.185	
9	0.561302	0.0820	79.272		0.021	75.713		0.031	82.216	
10	0.507839	1.5660	80.839		0.001	75.714		0.328	82.544	
11	0.448267	2.0180	82.857		0.011	75.725		2.201	84.745	
12	0.438511	0.0150	82.872		1.828	77.554		0.716	85.461	
13	0.364047	0.6500	83.522		1.559	79.112		0.091	85.552	
14	0.357696	3.2970	86.819		0.377	79.49		1.849	87.401	
15	0.30916	2.7480	89.568		0.016	79.506		2.334	**89.735**	**Rotation-Z**
16	0.255101	0.0089	**89.576**	**Translation-X**	6.441	85.947		0.41	**90.145**	**Rotation-Z**
17	0.206031	4.7060	**94.283**	**Translation-X**	0.156	**86.103**	**Translation-Y**	2.059	92.204	
18	0.193337	0.1020	94.384		6.055	**92.158**	**Translation-Y**	0.448	92.652	

From the data above it can be seen that the concrete structure mode achieves 90% mass participation in 3 motion patterns. In the translational-x motion pattern which has reached 90% is in mode 17. In the direction of translational-y motion pattern mass participation reaches 90% occurs in mode 18. And the rotazi-z motion pattern reaches 90% in mode 16.

4.3.2. Plastic Design Check

Checking the plastic design is done by increasing Aog (0.234g) to reach the conditions above CP (Collapse Prevention). Before reaching the collapsing condition, the plastic joint will pass through several stages of constraints as in the curve in Figure 11. By using SAP2000, the output of these boundaries will be seen in the form of a color indicator as will be explained next.

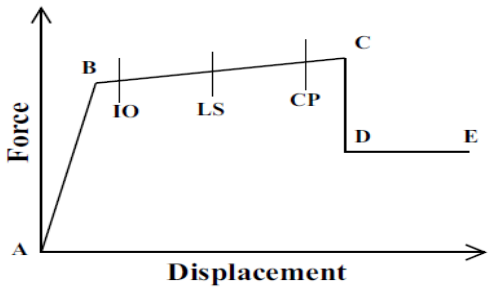

Figure 11 Plastic Joints

B: elastic boundary, the first plastic joint is formed in pink.
IO: immediate occupancy, plastic joints are formed in dark blue.
LS: life safety, plastic joints are formed in light blue.

CP: collapse prevention, plastic joints are formed in green.

Plastic joints are formed at the ends of members, be they beams or columns. The process of forming plastic joints together with the increased earthquake load received by structures (in this case the earthquake time history). The earthquake load received by the member structure will cause each member to increase the rotation value and moment. When a member of a structure has experienced a rotation value and a certain moment, the plastic joint will form.

In reinforced concrete structures, gedung e ITERA, the condition when plastic joints formed had already exceeded collapse prevention (CP) began to be achieved when the structure received an earthquake with an initial acceleration of 2 times the initial acceleration of the earthquake (0.468g) as in Figure 11.

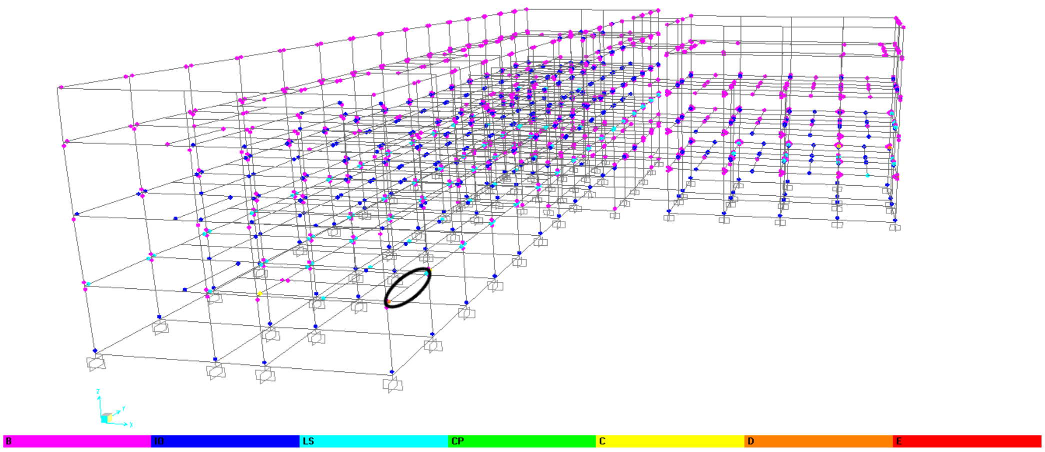

Figure 11 Plastic Joint Indicators 2 x Aog Concrete Structure

When the concrete structure has not received an earthquake load, the plastic joint is still not formed, and when given an earthquake load with the initial earthquake acceleration, the plastic joint indicator that is formed is almost all in purple (elastic boundary) and concrete structure with 208 members (circled in color black in Figure 11) is the member who first reaches the immediate occupancy condition (blue in the indicator). The same thing happens when the earthquake load is increased by the initial acceleration of the earthquake, the plastic joint indicator changes occur gradually and those who always reach the plastic boundary conditions are the 208 members.

The member used as a reference in plastic design checking is member number 208 (marked in Figure 11) because the first member experienced a collapse which is marked by changes in the plastic joint color indicator. Plastic joint indicators will appear on both ends of the member. The member is a y direction beam with a cross section size of 250x500 mm (bxh) in the model of the concrete structure of the E ITERA building.

Plastic joints are formed when a condition has been reached. In this case the SAP2000 plastic joint application is formed and can change according to the color of the indicator in the image often with an increase in the rotation value and moment in the structure.

4.3.3. Displacement

Checking the horizontal direction displacement from the top of the structural system is seen from the SAP output compared to the displacement limits set out in SNI 1726-2012 and FEMA 356.

Based on SNI 1726-2012 interstory displacement of 1.5% from the level below the level reviewed. In the concrete structure of the ITERA E building there are 2 joint points taken as a point of review of the intersection as seen in Figure 12, namely joint 1128 (J1128) and joint 709 (J709).

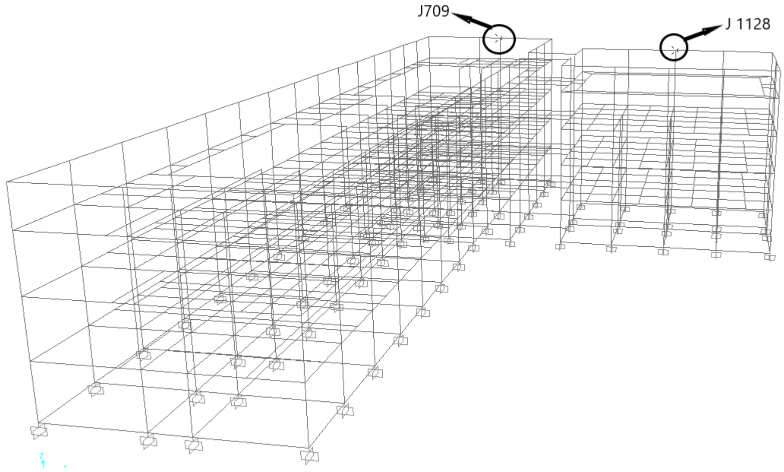

Figure 12 Concrete Joint Structures Reviewed

The concrete displacement between the floors on the joint is also checked along with the increase in aog as shown in Table 2.

Table 2 Concrete Structure Displacement (SNI 1726-2012)

PGA (g)	*Displacement, S* (m)		S limit (m)
	J1128	J709	
0.234	0.060381	0.018507	0.045
0.351	0.089083	0.028693	0.045
0.3744	0.094937	0.030719	0.045
0.3978	0.100812	0.032754	0.045
0.4212	0.106752	0.034807	0.045
0.4446	0.112583	0.036816	0.045
0.468	0.118506	0.038874	0.045

From the data in Table 2, it can be seen that the concrete structure of the ITERA E building has passed the permit displacement value (SNI 1726-2012) in the acceleration case of 0.234g at joint 1128 but has not passed the displacement permit on joint 709. The same thing happened when an earthquake acceleration was increased (as shown in Table 2).

By using the standard in FEMA 356 with the limitations of 1% immediate occupancy, LS (life safety) 2%, and CP prevention (collapse prevention) of 4% of the level below the level reviewed, the FEMA 356 displacement data on the concrete structure can be seen in Table 3.

Table 3 Concrete Structure Displacement (FEMA 356)

PGA (g)	*Displacement, S*(m)		IO = 1% (m)	LS = 2% (m)	CP = 4% (m)
	J1128	J709			
0.234	0.060381	0.018507	**0.03**	**0.06**	**0.12**
0.351	0.089083	0.028693	**0.03**	**0.06**	**0.12**

0.3744	0.094937	0.030719	**0.03**	**0.06**	**0.12**
0.3978	0.100812	0.032754	**0.03**	**0.06**	**0.12**
0.4212	0.106752	0.034807	**0.03**	**0.06**	**0.12**
0.4446	0.112583	0.036816	**0.03**	**0.06**	**0.12**
0.468	0.118506	0.038874	**0.03**	**0.06**	**0.12**

From Table 3 it can be seen using the FEMA 356 standard, the concrete structure of the E ITERA building has reached life safety conditions in the 0.234g earthquake for joint 1128. And for the joint 709 it is still in the immediate occupancy condition at 0.234g earthquake. The same thing happened in the two joints which was reviewed when an earthquake acceleration was increased (as shown in Table 3).

It can be seen that both using the SNI standard, increasing the earthquake up to 2 times the initial earthquake did not deliver the entire structure through the permit displacement. And using FEMA356 the concrete structure of the E ITERA building had not yet reached a collapse prevention condition when an earthquake acceleration was increased to 0.468g (2 times the initial earthquake acceleration).

4.3.4. Rotation

Rotation checking is carried out every increase in Aog (initial earthquake acceleration) from initial earthquake conditions (original) until the rotation of the structure reaches a collapsing condition.For the condition of rotation value on the concrete structure of the E ITERA building can be seen its representation in Table 4. And the member used to review the value of rotation on the structure is member number 208 according to the member which is used as a reference for checking the plastic design of E ITERA building concrete structures.

Table 4 Rotation of Concrete Structures

PGA (g)	0.05 θmax (rad)	0.05 θmin (rad)	0.95 θmax (rad)	0.95 θmin (rad)	IO (+)	IO (-)	LS (+)	LS (-)	CP (+)	CP (-)
0.234	0.01243	-0.01243	0.01459	-0.01459	0.01	-0.01	0.02	-0.02	0.025	-0.025
0.351	0.01907	-0.01907	0.02259	-0.02259	0.01	-0.01	0.02	-0.02	0.025	-0.025
0.3744	0.02041	-0.02041	0.0242	-0.0242	0.01	-0.01	0.02	-0.02	0.025	-0.025
0.3978	0.02174	-0.02174	0.02581	-0.02581	0.01	-0.01	0.02	-0.02	0.025	-0.025
0.4212	0.02307	-0.02307	0.02743	-0.02743	0.01	-0.01	0.02	-0.02	0.025	-0.025
0.4446	0.02441	-0.02441	0.02901	-0.02901	0.01	-0.01	0.02	-0.02	0.025	-0.025
0.468	0.0257	-0.0257	0.03059	-0.03059	0.01	-0.01	0.02	-0.02	0.025	-0.025

The increase in the initial earthquake acceleration in the concrete structure was carried out up to 0.468g (2 x Aog) because the plastic joint indicator had exceeded CP at the SAP output (the indicator was orange). From Table 4 it can be seen that in the earthquake 0.3978g the condition of CP (collapse prevention) had begun to form and in the 0.468g earthquake (2 x Aog) the CP condition had been reached. This condition can occur because the rotation value at the edges of the frame has exceeded the rotation value limit in FEMA 356 (according to the standard used in SAP).

4.3.5. Base Shear

This base shear is the result of dynamic shear force analysis with the most dominant loading combination of time history combination.For reinforced concrete structures, the value of base shear for non-linear dynamic seismic load time history is shown in Table 5.

Table 5 Base Shear of Concrete Structure

PGA (g)	Nilai *Base Shear* (kN)
0.234	43935.725
0.351	65555.331
0.468	87152.566

From the data in Table 5 above, it can be seen the value of base shear concrete structure for dynamic nonlinear time history earthquake load with initial earthquake acceleration of 0.234g is 43935.725 kN and increases in accordance with the increase in earthquake acceleration.

This basic shear force is the result of dynamic shear force analysis with the most dominant loading combination of time history combination. And from data above, it can be concluded that the dynamic base shear structure is affected by an increase in the initial acceleration of the earthquake (Aog). In addition, the base shear values larger will cause the cost of the lower structures to be more expensive because of the need for a stronger lower structure (for the same soil conditions).

5 Conclusions

This study describes the method of time history leveling in analyzing the behavior of structures in Building E ITERA. This method by increasing the seismic load of time history has been matched with the response of the spectrum of Building E ITERA (South Lampung), so that the response of the structure can be seen gradually. In this case the structural behavior reviewed is mass participation, plastic design, deviation, rotation and base shear. In addition this method can be used to predict which structures are vulnerable to failure as mitigation measures.

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