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# Seismic retrofitting analysis using concrete jacketing and shear wall on dental hospital building of Andalas University

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**Abstract.** Due to some damage to the structural elements of Andalas University Dental Hospital building, an evaluation of the building structure was carried out. Based on the structural evaluation using Indonesian standard code, SNI 03-1726-2012, it was found that the building is not strong enough to resist the combination loads acting on the structure, especially seismic load. Therefore, the structure should be seismically improved. In this study, there are two retrofitting methods are proposed and analysed: adding concrete jacketing and shear wall. The concrete jacketing method was conducted by enlarging the cross-sectional dimensions and adding reinforcement bar to the structural elements (beam and column) that are unable to resist the working loads. By using the concrete jacketing method, there are many structural components must be strengthened. Therefore, another retrofitting method is proposed, by adding the concrete shear wall. Shear wall is specially designed structural walls include in the buildings to resist the horizontal forces that induced in the plane of the wall due to earthquake forces. From the analysis, it is concluded that the two retrofitting methods are effective enough to reduce the internal forces and displacement of the building. Considering the more effective and efficient work, retrofitting method using shear wall was recommended for strengthening the Dental Hospital building.

## 1. Introduction

The reinforced concrete (RC) structure should be designed according to recent standard code, which has enough capacity to carry the working combined loads acting on the structure [1]. The lack of structural performance in existing building was caused by many factors such as change of seismic code. The seismic adequacy of existing buildings should be evaluated to prevent the enormous loss of life and property due to the earthquakes [2].

The Dental Hospital building is one of the new building in Andalas University. During construction, it was found deflection at some beams and damage of slabs which was experience considerable thickening, as seen in Figure 1. Based on the design documents, it is identified that the building was designed using the old standard code in determining seismic load. In order to prevent further damages, the building should be retrofitted. Retrofitting is a set of operations done on a part or all structure so that it can bear more loads and overheads than the initial condition and shows better behavior characteristics. Three major purposes are considered in retrofitting buildings: increasing resistance against lateral loads, increasing ductility and increasing resistance with ductility [3].

There are many retrofitting methods to improve the performance of the building structure such as steel bracing, shear walls, and concrete jacketing. A comparative analysis of building seismic retrofitting for a Dental Hospital building was carried out. In order to find the best retrofitting solution,



several variants were proposed for the comparative analysis: reinforced concrete jacketing method; and shear wall method. A comparison has been made for the retrofitted building using concrete jacketing and shear wall in terms of cross-section capacity, lateral displacement, bending moments, shear forces, story drifts, and natural vibration period.



**Figure 1.** The damages of (a) the beam and (b) the slab on the existing building

## 2. Structural Analysis of Existing Structure

All data related to structural analysis was taken from the field investigation and the design document from the consultant. Data taken directly is the dimension of each structural element and reinforcement bar installed. The other data such as  $f'_c$  and  $f_y$ , are taken from the design document. The building structure was modeled and analyzed by using ETABS program [4]. The structure analysis results data from the ETABS program such as internal forces and displacements can be used to evaluate the structural performance due to gravity and earthquake loading. Seismic loading method for a medium-rise building or irregular building can be done using spectrum response dynamic analysis [5].

### 2.1. Data of Existing Structure

A three-story Dental Hospital building with plan and front views, as shown respectively in Figures 2 and 3, is considered for the study. The building is composed of moment resisting RC frame with solid slab, 120 mm thickness, located in Padang city. The structure members are made of the reinforced concrete structure. The overall plan of the building is rectangular with dimensions 40.8 x 14.2 m<sup>2</sup>. The height of the building is 13.2 m. The columns size are K1 (350 x 350 mm); K2 (350 x 600 mm); K3 (350 x 400) mm, and beams size are B1 (300 x 400 mm); B2 (350 x 600 mm); B3 (200 x 300 mm); B3-a (200 x 300 mm); BM1 (250 x 400 mm); BM2 (250 x 500 mm). The 3D model of the building structure is developed in ETABS program, as shown in Figure 4. Beam and column elements were modeled as frame elements while the in-plane rigidity of the slab is simulated using rigid diaphragm action.

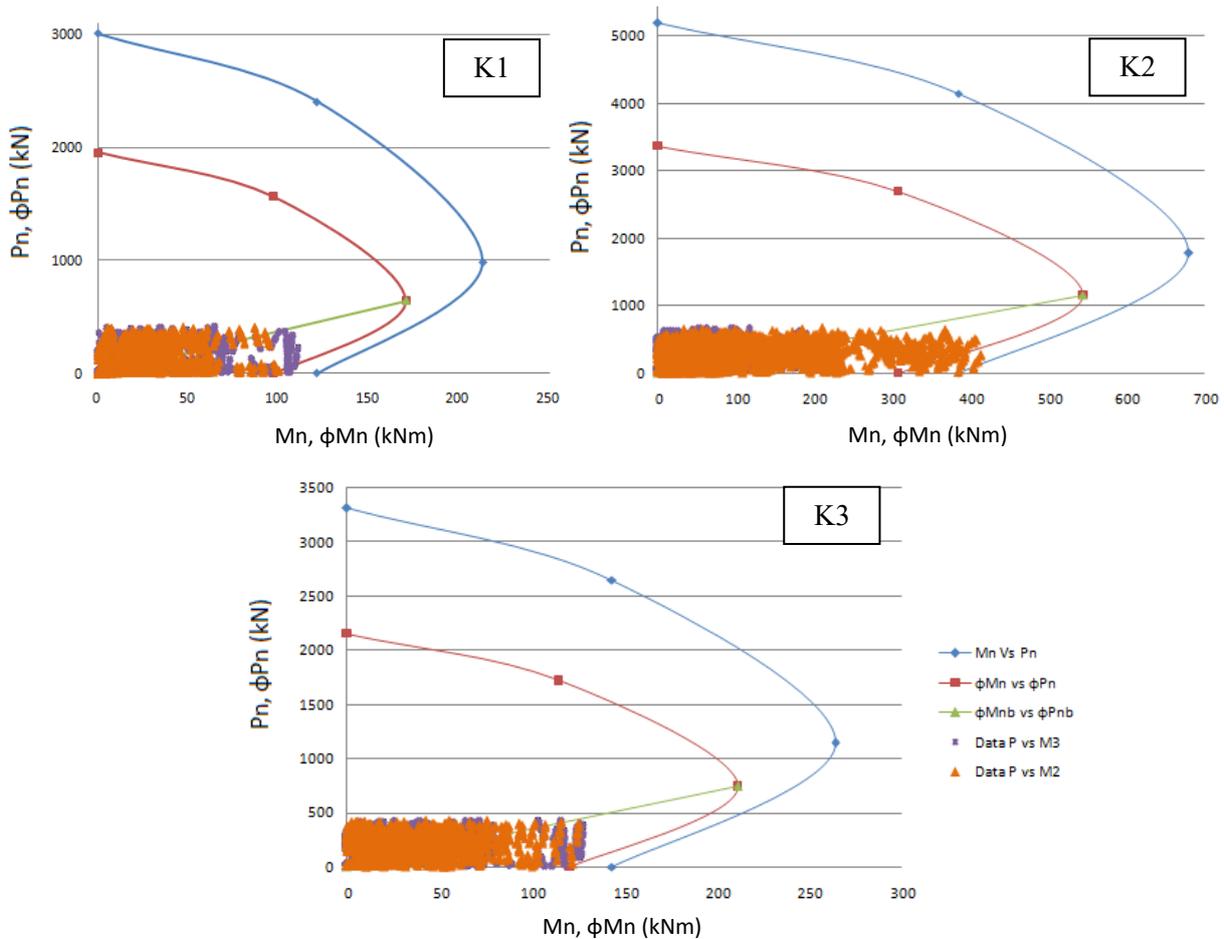
The building is analyzed for the combined effect of gravity and earthquake loads, considering all the design load combinations specified in the SNI 1725:2012 standard code [6]. The RC frame structure was analysed according to SNI 03-2847-2013 standard code [7]. The compressive strength of concrete is taken as 20.75 MPa; the yield strength of steel reinforcement bars is 390 MPa and 240 MPa for longitudinal and transverse reinforcement, respectively.

In designing a multi-story building, the vertical gravitational forces (dead and live loads) and the horizontal forces of earthquakes should be calculated properly [8]. The required parameters to determine earthquake load are as follows: based on SNI 03-1726-2012 the earthquake reduction factor (R) is 8, and the building importance factor (I) is 1.5.  $S_{DS}$  and  $S_{D1}$  values obtained by using the response spectra of earthquake Padang City on the medium soil conditions. The value of  $S_{DS}$  is 0.932 and  $S_{D1}$  is 0.60. Values of R, I,  $S_{DS}$  and  $S_{D1}$  are then used as input data to the ETABS software to determine earthquake load.



**Table 1.** The beam capacities in the existing building

| Story | Beam             | Reinf. Bar |      | Bend. Moment     |             | $\phi M_n < \mu$ | Shear Force     |            | $\phi V_n < V_u$ |
|-------|------------------|------------|------|------------------|-------------|------------------|-----------------|------------|------------------|
|       |                  | T          | C    | $\phi M_n$ (kNm) | $\mu$ (kNm) |                  | $\phi V_n$ (kN) | $V_u$ (kN) |                  |
| 1     | B1 (30.40) Ext.  | 3D19       | 3D19 | 92.1             | 116.4       | NOT              | 213.8           | 142.1      | OK               |
|       | B1 (30.40) Int.  | 3D19       | 3D19 | 92.1             | 131.3       | NOT              | 213.8           | 111.9      | OK               |
|       | B2 (35.60) Ext.  | 7D19       | 5D19 | 335.2            | 212.0       | OK               | 345.7           | 160.3      | OK               |
|       | B2 (35.60) Int.  | 7D19       | 5D19 | 335.2            | 316.5       | OK               | 345.7           | 216.3      | OK               |
|       | B3 (20.30) Ext.  | 2D19       | 2D19 | 21.1             | 56.0        | NOT              | 107.1           | 63.0       | OK               |
|       | B3a (20.30) Int. | 5D19       | 3D19 | 50.7             | 61.2        | NOT              | 107.1           | 90.3       | OK               |
| 2     | B1 (30.40) Ext.  | 3D19       | 3D19 | 92.1             | 89.3        | OK               | 213.8           | 73.0       | OK               |
|       | B1 (30.40) Int.  | 3D19       | 3D19 | 92.1             | 78.7        | OK               | 213.8           | 81.3       | OK               |
|       | B2 (35.60) Ext.  | 7D19       | 5D19 | 335.2            | 109.9       | OK               | 345.7           | 77.7       | OK               |
|       | B2 (35.60) Int.  | 7D19       | 5D19 | 335.2            | 126.9       | OK               | 345.7           | 90.0       | OK               |
|       | B3 (20.30) Ext.  | 2D19       | 2D19 | 21.1             | 20.9        | OK               | 107.1           | 38.9       | OK               |
| 3     | B1 (30.40)       | 3D19       | 3D19 | 92.1             | 26.0        | OK               | 213.8           | 19.6       | OK               |
|       | B-M2 (25.50)     | 5D19       | 3D19 | 194.7            | 39.0        | OK               | 345.7           | 25.8       | OK               |



**Figure 5.** The P-M interaction diagram of existing columns (K1, K2, and K3)

2.2.2. *Column capacities.* P-M interaction diagrams illustrate the ability or capacity of a column to carry the axial and bending moment due to the working loads, as shown in Figure 5. The points illustrate the combination of axial force and bending that acting in the column. In the column K3, the internal forces acting on the column are still within the nominal moment and axial reduction limit so that the columns are still able to resist the internal forces due to the loads. While in the columns K1 and K2, the axial and moment of the columns exit the nominal axial and moment limit, it means that the column is unable to withstand the working load. Based on shear force capacity, all columns are able to withstand the shear force acting on the structure.

2.2.3. *Inter-story drift.* Drift is generally defined as the lateral displacement of one story relative to the story below. Drift control is necessary to limit damage to interior partitions, elevator and stair enclosures, glass, and cladding systems. Drift,  $\Delta x = \delta_x - \delta_{x-1}$ , calculated by using SNI 03-1726-2012 for x and y directions, as shown in Table 2. It can be seen that the inter-story drift in x direction at the top story of the building has not reached the required limit.

**Table 2.** The inter-story drift in the existing building

| Story           | Height (mm) | $\Delta a$ X,Y (mm) | Disp. X (mm) | Drift X (mm) | $\Delta S$ X (mm) | $\Delta a < \Delta S$ X | Disp. Y (mm) | Drift Y (mm) | $\Delta S$ Y (mm) | $\Delta a < \Delta S$ Y |
|-----------------|-------------|---------------------|--------------|--------------|-------------------|-------------------------|--------------|--------------|-------------------|-------------------------|
| 1 <sup>st</sup> | 0           | 0                   | 0            | 0            | 0                 | OK                      | 0            | 0            | 0                 | OK                      |
| 2 <sup>nd</sup> | 4200        | 48.46               | 6.44         | 6.44         | 23.61             | OK                      | 5.75         | 5.75         | 21.08             | OK                      |
| 3 <sup>rd</sup> | 4000        | 46.15               | 13.84        | 7.40         | 27.13             | OK                      | 10.28        | 4.53         | 16.62             | OK                      |
| Roof            | 5000        | 57.69               | 30.43        | 16.59        | 60.83             | NOT                     | 13.72        | 3.44         | 12.61             | OK                      |

$$\Delta a = 0,015 H / \rho$$

$$\Delta s = D_x \cdot C_d / I_e$$

2.2.4. *Natural vibration period.* The vibration period of the Mode 1 ( $T_1$ ) structure in the X direction is 0.732 second; it means that the structure of the building is likely to experience movement every 0.732 second. Then, the approximate fundamental period ( $T_a$ ), in seconds, is determined from the equations (1) and (2):

$$T_a = C_t \cdot h_n^x \quad (1)$$

$$\text{Natural vibration period of the structure from ETABS } (T_1) < T_a \quad (2)$$

$$0.732 < 0.0466 \times 200.9$$

$$0.732 > 0.6907 \dots \text{NOT OK}$$

The natural vibration period of the structure does not meet the required limits. It concludes that the structure has not sufficient rigidity.

Based on the results of the strength and performance evaluation of the existing building structure, it can be concluded that the building structure is not able to withstand the combination of loads so that the structure needs to be strengthened. Retrofitting methods using concrete jacketing and shear wall to improve the seismic performance of the building was proposed in this study. Seismic performance of the existing building and the retrofitted building is compared to quantify the improvement of performance due to the adding of concrete jacketing and shear wall.

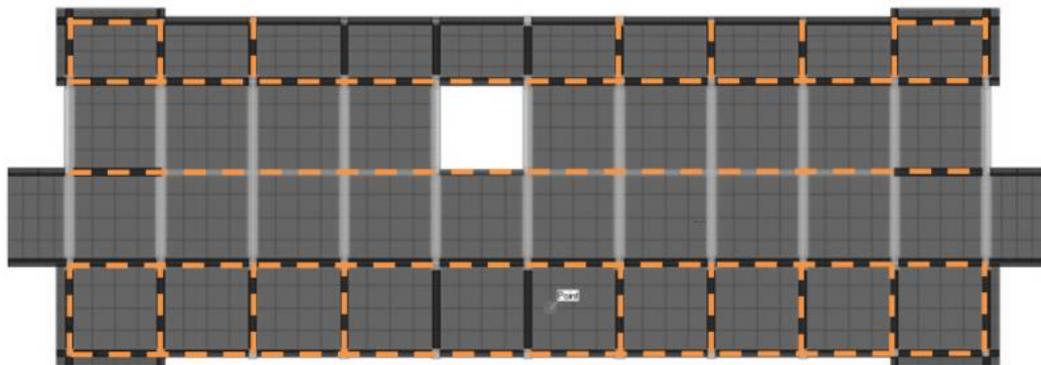
### 3. Analysis of Retrofitting Structure

#### 3.1. Strengthening by concrete jacketing

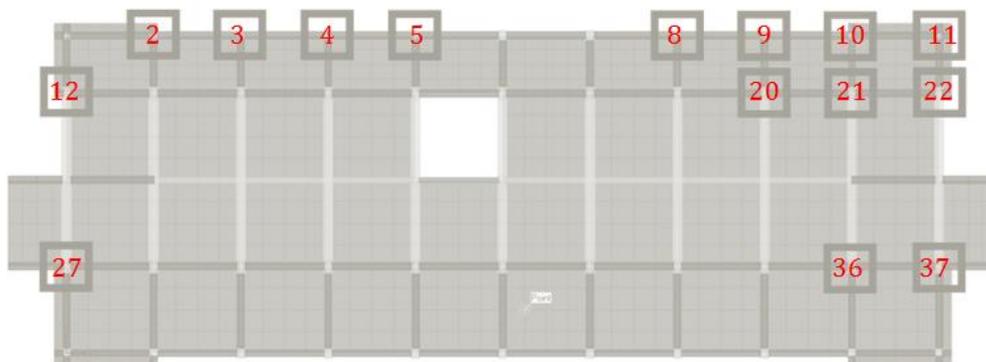
Concrete jacketing is one of retrofitting structures used to the columns and beams of the building. The demand for using concrete jackets to strengthen or repair reinforced concrete has been increasing in the past few decades. Reinforced concrete jacketing is a common method for retrofitting existing

columns with poor structural performance. Jacketing implemented by enlarging the column and beam section by increase the amount of reinforcement [9, 10].

Concrete jacketing is a popular method of retrofit as it follows the same design and construction procedures of RC columns. The jacket can provide protection from both environmental effects and fire. The jacket can increase the axial and flexural strength by increasing confinement and providing additional steel reinforcement. Modeling columns and beams that jacketed on ETABS is done by enlarging the cross-sectional dimensions and adding reinforcement according to the amount planned for columns and reinforcing beams. In this study, the beams and columns selected to concrete jacketing is shown in Figures 6 and 7, respectively. Tables 3 dan 4 show the detailing and dimension of concrete jacketing on the columns and beams, respectively.

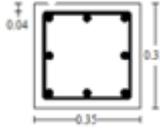
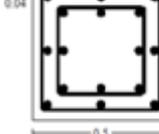
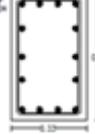
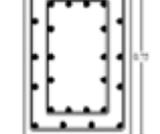


**Figure 6.** The position of beams needed to concrete jacketing

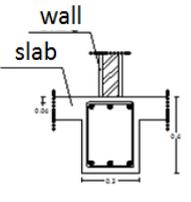
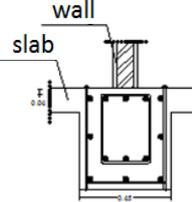
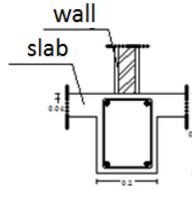
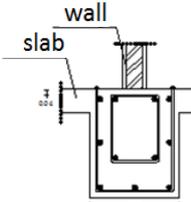


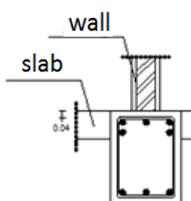
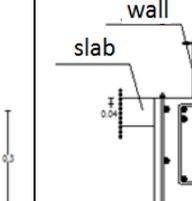
**Figure 7.** The position of columns needed to concrete jacketing

**Table 3.** The section of concrete jacketing on the columns

|                  |   |   |  |   |   |
|------------------|---|---|--|---|---|
| Column 1 (K1)    |  |  | Column 12, 20, 21, 22, 27, 36, 37 (K2) |  |  |
| Condition        | Existing  | Conc. Jacketing   | Condition                              | Existing  | Conc. Jacketing   |
| Section          | 350.350   | 500.500   | Section                                | 350.600   | 500.750   |
| Flex. Reinf. Bar | 8D19  | 16D19   | Flex. Reinf. Bar                       | 14D19   | 28D19   |
| Shear Reinf. Bar | 2Ø12-100  | 4Ø12-100  | Shear Reinf. Bar                       | 2Ø12-100  | 4Ø12-100  |

**Table 4.** The section of concrete jacking on the beams

|                  |   |   |                  |  |   |
|------------------|---|---|------------------|--|---|
| Beam B1          |  |  | Beam B3          |  |  |
| Condition        | Existing  | Conc. Jacketing   | Condition        | Existing   | Conc. Jacketing   |
| Section          | 300.400   | 450.550   | Section          | 200.300  | 350.450   |
| Flex. Reinf. Bar | 6D19  | 13D19   | Flex. Reinf. Bar | 4D13   | 11D13   |
| Shear Reinf. Bar | 2Ø12-100  | 4Ø12-100  | Shear Reinf. Bar | 2Ø10-100   | 4Ø10-100  |

|                  |  |   |
|------------------|--|---|
| Beam B3A         |  |  |
| Condition        | Existing   | Concrete Jacketing  |
| Section          | 200.300  | 350.450   |
| Flex. Reinf. Bar | 8D13   | 15D13   |
| Shear Reinf. Bar | 2Ø10-100   | 4Ø10-100  |

3.1.1. *Load-bearing capacity.* The capacity of the columns and beams can be increased by the concrete jacking method so that the structure is capable of carrying axial and bending moments. Likewise, the columns without jacking are still able to carry the working loads that work significantly in column capacity, especially the increase in moment capacity. The moment capacity increase reached 198% for the column K1 and 212% for the column K2 compared to the existing structure. Based on the analysis, the increase of capacity is enough to overcome the problems that occurred on the 1<sup>st</sup> floor columns that have not to carry the working bending moment.

**Table 5.** The percentage of increase in the beam capacity

| Beam | Zone    | Moment   |              | Increasing (%) | Shear    |              | Increasing (%) |
|------|---------|----------|--------------|----------------|----------|--------------|----------------|
|      |         | Existing | Retrofitting |                | Existing | Retrofitting |                |
| B1   | Support | 92.1     | 219.6        | 138.5          | 213.8    | 344.9        | 61.3           |
|      | Midspan | 92.1     | 347.7        | 277.6          | 213.8    | 344.9        | 61.3           |
| B3   | Support | 21.1     | 66.6         | 215.3          | 107.1    | 202.4        | 89.1           |
|      | Midspan | 21.1     | 82.9         | 292.4          | 107.1    | 202.4        | 89.1           |
| B3a  | Support | 47.6     | 111.2        | 133.4          | 107.1    | 202.4        | 89.1           |
|      | Midspan | 47.6     | 126.3        | 165.2          | 107.1    | 202.4        | 89.1           |

Table 5 shows the percentage of increase in load-bearing capacities (bending and shear) in the beams. After retrofitting by concrete jacking, there is a significant increase in beam capacities, especially the

bending moment. Bending moment and shear capacity increase reached 292% and 89% compared to the existing structure, respectively. Based on the analysis, the increase of capacity is enough to carry the working loads that occurred in the 1<sup>st</sup> floor beams.

3.1.2. *Displacement.* The use of concrete jacketing was indirect effect to the building displacement. The lateral displacement has been calculated for both X and Y-directions, for effects of the earthquake in both directions, as seen in Figure 8. The inter-story drift of jacketing structure for all stories has been less than the required permit limit. The displacement on x-direction reduced by around 15% and y-direction is a maximum of 26%.

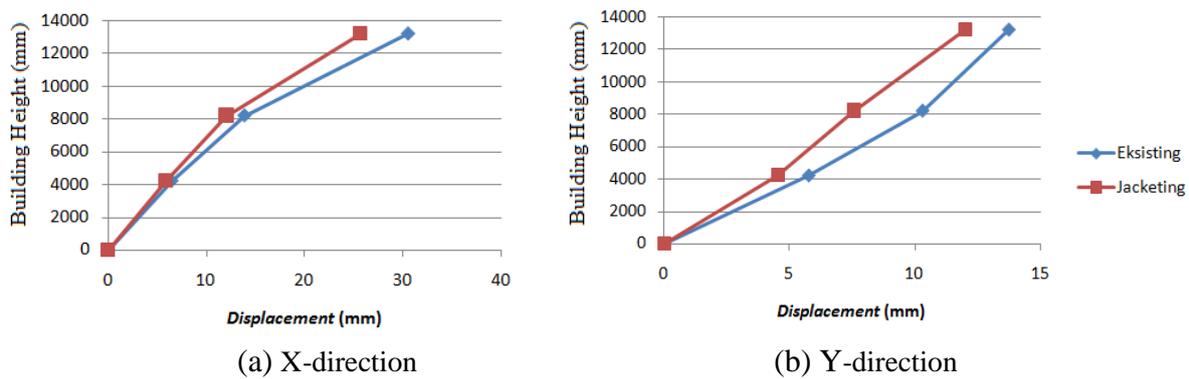


Figure 8. The comparison of displacement between existing and jacketing structures

3.1.3. *Natural vibration period.* The vibration period of Mode 1 ( $T_1$ ) structure in the X direction is 0.6075 seconds. By using the parameter values of the  $C_t$  and  $x$  approach periods, then  $T_a$  is 0.6907. The natural vibration period of the structure has met the required limits. Therefore, it concludes that the addition of jacketing to certain beams and columns can reduce the structure's natural vibration period, which means that the structure has sufficient rigidity.

### 3.2. Strengthening by the shear wall

RC shear walls have been used as the most effective method to enhance the seismic resistance of the existing building. The optimal location of new structural elements should be considered when it was placed, which may align to the full height of the building to minimize torsion. In this paper, the results compare for frame structure coupled with a shear wall at the boundary and exterior faces of building [11]. In this study, the shear wall is placed at the corners of the building with variations on the only 1<sup>st</sup> and both 1<sup>st</sup> and 2<sup>nd</sup> floors, as shown in Figures 9 and 10. Shear wall detail used are as follows:

Thickness: 20 cm

The compressive strength of concrete ( $f'_c$ ): 24.9 MPa

The yield strength of reinforcement bar ( $f_y$ ): 390 MPa

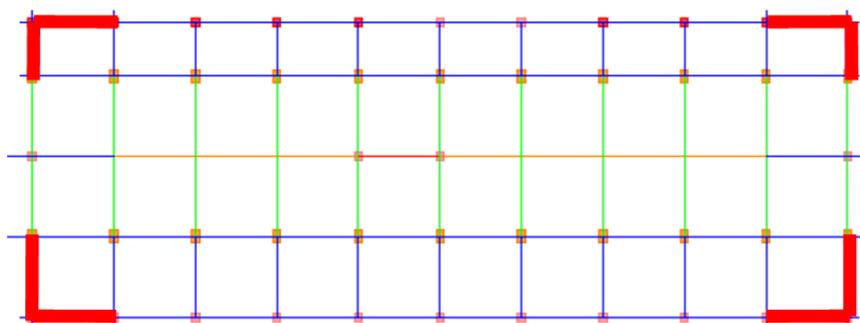
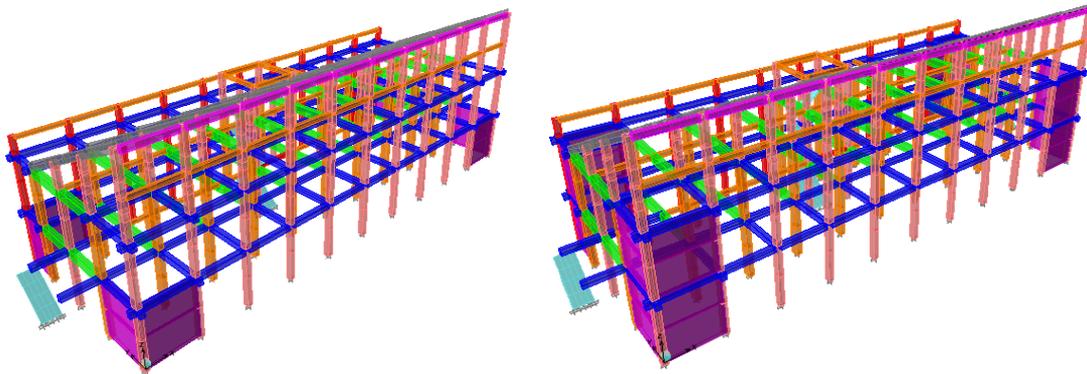


Figure 9. The placement location of the shear wall system



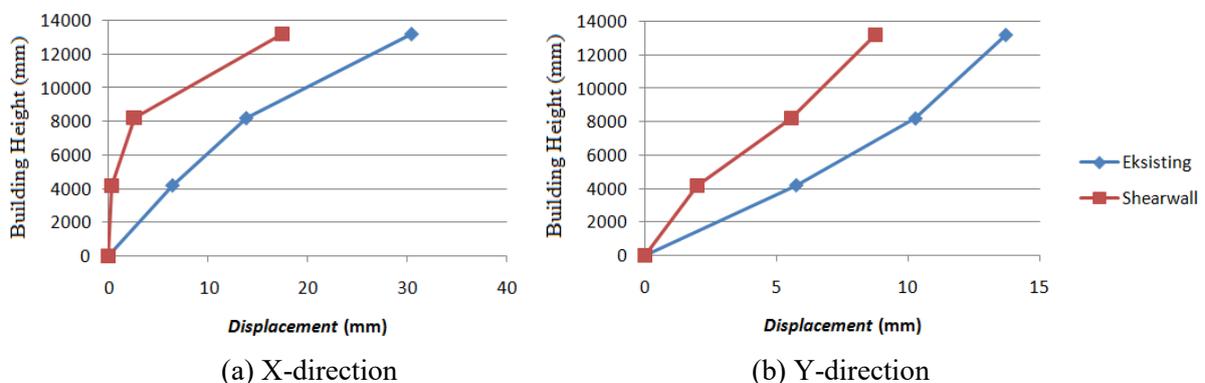
**Figure 10.** The retrofitting structure by adding the shear wall (a) Model 1; (b) Model 2

**3.2.1. Load-bearing capacity.** After retrofitting by adding the shear wall, the cross-sectional capacity of the columns does not change, but the internal force due to the combination of loads reduce. It was found that all the points on the graph had entered in the P-M interaction diagram, both for Models 1 and 2. Also, these models can carry the shear forces due to the given load. There is a beam that unable to resist the bending moment in Model 1 (beam B1 on the 2nd floor), whereas all beam is able to carry the internal forces in Model 2.

By using this retrofitting method, the increase in axial force occurs in the columns. It caused by the shear wall's weight-load (dead load). The maximum percentage of axial force increase is 34% and 4% on the 1<sup>st</sup> and 2<sup>nd</sup> floors, respectively. While the shear force and bending moment decrease, by the maximum reduction for the shear is about 90%. On the top floor that not fitted with a shear wall, the shear force increases by around 14%. The bending moment decreases by 94%, and on the top floor, it has increased by 59%.

After retrofitting using the shear wall, there is some increase in axial force and a decrease in shear force and bending moment in the structure. Maximum axial force increases up to 396% because it must bear the weight of the shear wall itself. The shear force is reduced by 25%, while on the 2<sup>nd</sup> and top floors; there is a beam that has increased shear force. The bending moment decreased by around 66%.

**3.2.2. Displacement.** Inter-story drift on X-direction in Model 2 is lower than the required limits, but the top floor is still higher than the required limits in Model 1. The difference of structural response occurs in the structure after the attached shear wall. A significant reduction occurred by around 94% and 65% for X and Y-directions, respectively. A comparison graph of displacement between the existing structure and retrofitting by the shear wall is shown in Figure 11.



**Figure 11.** The comparison of displacement between existing and shear wall structures

*3.2.3. Natural vibration period.* The vibration period of the Mode 1 (T1) in the X-direction for the structure with the shear wall only on the 1st floor (Model 1) is 0.494s, whereas for a structure with the shear wall on the 1<sup>st</sup> and 2<sup>nd</sup> floors (Model 2) is 0.358s. By using the Ct and x approaches, then the Ta is 0.6907. Both the Models 1 and 2 have a small natural vibration period from the required limit, which means that the structure has sufficient rigidity.

Based on the analysis of the proposed models, it was concluded that the concrete shear wall could be the best alternative retrofitting (Model 2). The Model 1 has some internal forces that still higher than the required.

### 3.3. Comparison of the bill of quantity (BoQ) for concrete jacketing and shear wall methods

The amount of cost (BOQ) needed for retrofitting a structure should be considered to determine the best retrofitting method. The calculation results of retrofitting cost show that the shear wall method Model 2 requires a cost IDR 359,845,700.-, while the concrete jacketing method requires a cost IDR 550,687,600. From these two proposed retrofitting methods, adding the shear wall becomes a more economical alternative method than the concrete jacketing.

## 4. Conclusion

RC buildings which were designed without following the current seismic code may undergo severe damage when the earthquake occurs. The need for retrofitting of the earthquake-vulnerable buildings in Indonesia has been tremendously increased after the devastating earthquake in the past few years. Retrofitting by concrete jacketing and shear wall is equally effective in building structures that are unable to withstand working loads. The choice of the best retrofitting alternative used must consider several other aspects, such as ease of implementation, risk in construction work, and other factors that can make the work more effective and efficient. The results of the two alternatives retrofitting methods, Andalas University Dental Hospital building is recommended to be strengthened by providing a structural wall (shear wall) attached in 1<sup>st</sup> and 2<sup>nd</sup> floors of the building corner. The reason for choosing the concrete shear wall method is effective to reduce the internal forces and displacement of the building. Finally it is more economical in terms of cost.

## References

- [1] H. A. Safarizkia, S. A. Kristiawan, and A. Basuki, Evaluation of the Use of Steel Bracing to Improve Seismic Performance of Reinforced Concrete Building, The 2<sup>nd</sup> International Conference on Rehabilitation and Maintenance in Civil Engineering, Procedia Engineering 54, 2013, pp. 447 – 456.
- [2] S. S. Ramteke, S. P. Raut, Pushover Analysis and Seismic Retrofitting Using Shear walls and Bracing System of Frame Structure, International Journal of Modern Trends in Engineering and Research (IJMTER) Volume 02, Issue 04, April – 2015, pp. 50-59.
- [3] M. Ghaderi, H. Ghafarzade, and J. Jamali, Evaluation of Seismic Performance of the Reinforced Concrete Buildings with Steel Bracing, Shear Wall and Column Jacketing, Technical Journal of Engineering and Applied Sciences 3 (23) pp. 3313-3324, 2013.
- [4] Computers and Structures, Inc. 2005. Manual ETABS (Integrated Building Design Software). California, USA.
- [5] U. L. Salve and R. S. Londhe, Effect of Curtailed Shear Wall on Storey Drift of High Rise Buildings Subjected To Seismic Loads, IOSR Journal of Mechanical and Civil Engineering (IOSR-JMCE) Volume 11, Issue 4 (Jul-Aug. 2014), pp. 45-49.
- [6] Badan Standardisasi Nasional (BSN). SNI 03-1726-2012 Standar Perencanaan Ketahanan Gempa Untuk Struktur Bangunan Gedung dan Non Gedung, BSN, ID, 2012.
- [7] Badan Standardisasi Nasional (BSN). SNI 03-2847-2013 Persyaratan Beton Struktural untuk Bangunan Gedung, BSN, ID, 2013.
- [8] S. A. Fauzan, Erizal, and A. Sapei, Strengthening of Tapered Beam using CFRP, Asian Journal

of Applied Sciences Volume 03 – Issue 06, December 2015, pp. 886- 895.

- [9] Fauzan et. al, Structural Failure Analysis and Retrofitting Mesjid Raya Andalas Padang Earthquake Post 30 Sept 2009, Journal of Civil Engineering, Volume 7, ISSN 1558-2133, 2011.
- [10] Fauzan, F. A. Ismail, S. Apriwelni, I. Faradiza, and Z. A. Jauhari, Strengthening of Nurul Ilmi Mosque with Concrete Jacketing, INSIST Vol. 1 No. 1, October 2016, pp. 25 – 29.
- [11] Y. Haryanto, B. S. Gan, N. G. Wariyatno, and E. W. Indriyati, The Performance of A Ten-Story Irregular Apartment Building Model under Seismic Load in Purbalingga Regency Indonesia, ARPN Journal of Engineering and Applied Sciences Vol. 12, No. 17, September 2017, pp. 4858- 4866.