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Development of RISHA Precast Concrete System for School Buildings Function in Indonesia's Severe Earthquake Regions

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Abstract **-** This paper aims to convey the results of the development of dry joint modular precast concrete system for school buildings function in the severe earthquake areas of Indonesia with values of $Ss \geq 0.911$ and $S1 \geq 0.391$. The numerical model of the structure was developed by utilizing the partial experimental test results from various type of structural member joints of the system. From the results of these tests, the nonlinear behavior of each type of structural joints in the form of a moment vs rotation curve is obtained to be implemented in the structural model using nonlinear link elements in order to represent the nonlinear behavior of the structure. The contribution of strength and stiffness of the infilled masonry walls using lightweight Autoclaved Aerated Concrete (AAC) brick is modelled through nonlinear strut elements whose behavior has been calibrated with experimental test results. A pushover analysis in the numerical model was carried out to obtain the system capacity curve of the proposed building structure with the typology of the school building. The results of performance point evaluation of the structural capacity curve using methods of ATC-40 and FEMA 440 in various loactions of severe earthquake areas in Indonesia show that the performance of Damage Control (DC) was achieved by providing a horizontal steel frame at the topmost elevation of the building structure in order to obtain the diaphragm behavior in each structural module.

Keywords**:** dry joint, modular, precast concrete, RISHA, school building, pushover analysis

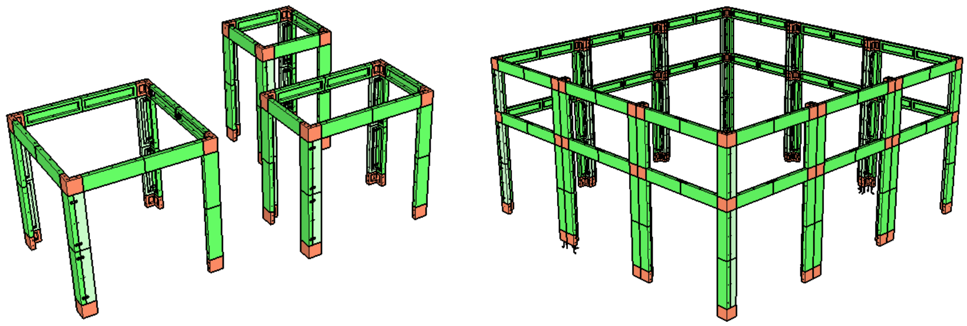
1. Introduction

RISHA (abbreviation for *Rumah Instan Sederhana Sehat* / Instant home, simple, 'healthy') is a precast concrete system with bolted connection that was developed by Research Institute of Human Settlements, Department of Public Works of Republic Indonesia in 2004. The precast concrete structure comes with three reinforced concrete structural components namely P1, P2, and P3. The beam components in RISHA structure are assembled by the P1 components while the column components are assembled by the combination of P1 and P2 that can be formed to the L, T, and Plus column section shape [1]. Initially, the precast concrete system was designed as modular volumetric structure with the standard modular dimension of 3x3x3 m^3 for residential function. The standard modular structures then are replicated in order to achieve the targeted dimension of the overall structure.

One of the advantages of a precast system like RISHA is the good economic value obtained when it is mass-produced [2]. A study prove that houses with RISHA technology have advantages in terms of cost and duration of construction [3]. In addition, the application of this precast system also has advantages in terms of environmental friendliness which is achieved through the efficient use of concrete volume compared to conventional buildings [4].

Currently, there are some needs of the RISHA precast concrete system to be implemented for school buildings that obviously need larger structure. This lead to the extension in the dimension of the precast concrete system standard volumetric modular structure up to 9x9x4.5m^3 from the initial design of 3x3x3 m^3 as illustrated in Figure 1 to meet the minimum size requirement of a classroom in Indonesia.

In year 2021, Directorate of Housing and Settlement Engineering Development, Directorate General of Human Settlements, Ministry of Public Works and Housing, conducted a research about development of the precast concrete system in order to be implemented for school building. The research involved the combination of several experimental laboratory tests and analytical works. As the result, this paper proposes the configuration and design for the modular precast concrete structure for school building applications in the severe earthquake areas in Indonesia with values of $Ss \geq 0.911$ and $S1 \geq 0.391$.



(a) Standard volumetric module (b) Extended volumetric module

Fig. 1: Standard and extended volumetric module of the proposed precast concrete structure.

2. Literature Review

2.1. Precast Structure Modelling Strategy Literature Review

Modeling the joint behavior of precast reinforced concrete elements is a complex mathematical problem. When calculating conventional reinforced concrete structures, the structural elements are assumed to be an integral monolith system. Meanwhile, this assumption does not apply to precast concrete systems where the connection behavior model can only be known through experimental studies. Each type of precast joint system has a different behavior from one another. The actual computational model for the analysis of precast joints has resulted in the introduction of semi-rigid joint systems.

One method that can be applied to model semi-rigid connections in structural models is to use nonlinear link elements at the connection points. With these link elements, the stiffness matrix and the equivalent load matrix can be modified so that the effect of the connections on the behavior of the structure can be modeled [5]. Various studies [6-11] show that the dry bolted precast concrete connection has the benefits of better ductility and energy dissipation ability, and more convenience in assembling. Equivalent stiffness formulation methods to simplify the analysis of precast concrete frame structures with semi-rigid joints can also be used [12].

2.2. Experimental Study

The experimental laboratory tests are mainly purposed to understand the nonlinear behavior of joint connections that exist in the proposed system. A number of cyclic loading tests were performed partially to obtain the nonlinear behavior of each joint type such as column to foundation joint and beam to column joint [18]. Further the results of those partial joint tests have been processed and analyzed to develop a well calibrated numerical model of the proposed system.

3. Materials and Methods

3.1. Specimen Description

The experimental tests were conducted using the precast concrete components that were produced without a mix design process in order to get the minimum strength data that represents the poor material condition which is frequently found in the fields. Here the concrete for the specimens were produced using 2 cements : 3 sands : 4 screen agregate composition that have the average compressive strength of 17 MPa. This specimen is used in order to get the minimum strength for research use only, while the actual authorized guideline still specifies fc' of 25 MPa for new construction.

The mechanical connection components such as bolt and strip plate were using the standard connection dimension where the diameter of bolts are 12 mm (M12 bolt) while the strip plates for bolted connection pad were 2 mm thicks and

30 mm wide. The material of M12 bolts are ASTM A307 while the strip plates are using galvanized steel materials with yield stress (fy) of 250 MPa. The examples of some structural components photopgraphs is presented in Figure 2.

Fig. 2: Examples of some structural components photographs.

3.2. Precast Structure Joint Connection Nonlinear Behavior

As mentioned above, the precast joint connections nonlinear behaviors in this research were obtained by a number of partial joint connections cyclic loading tests. In this research, partial joint connections tests were performed for an exterior beam-column joint connection and several column joint connections. The results of each capacity curves then are converted into momen-rotation curves that represent the semirigid behavior of each connections.

The typical setup of the column joint connections for all column section types is illustrated in Figure 3. The rotation of the column joints are calculated by processing the the displacement datas that are recorded by some transducers located around the lower part of the column as depicted in Figure 3. The momen-rotation curves of several column types of the proposed system such as column L, T, and Plus' in both orthogonal directions of the column sections are presented in Figure 4, while the momen-rotation curve of the exterior beam-column joint connection followed by it's cyclic loading test configuration is presented in Figure 5. It is necessary to note that all of the column joint tests are conducted in form on fully anchored condition as specified in the official RISHA guideline (no bolt holes in the column-to-foundation connection are left empty).

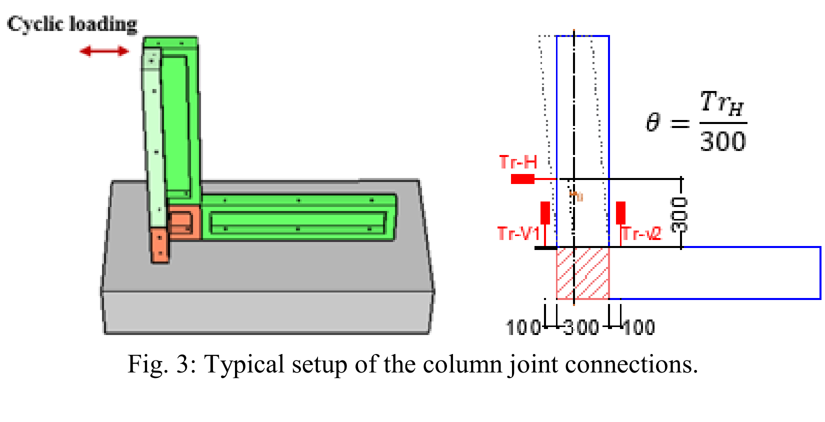


Fig. 3: Typical setup of the column joint connections.

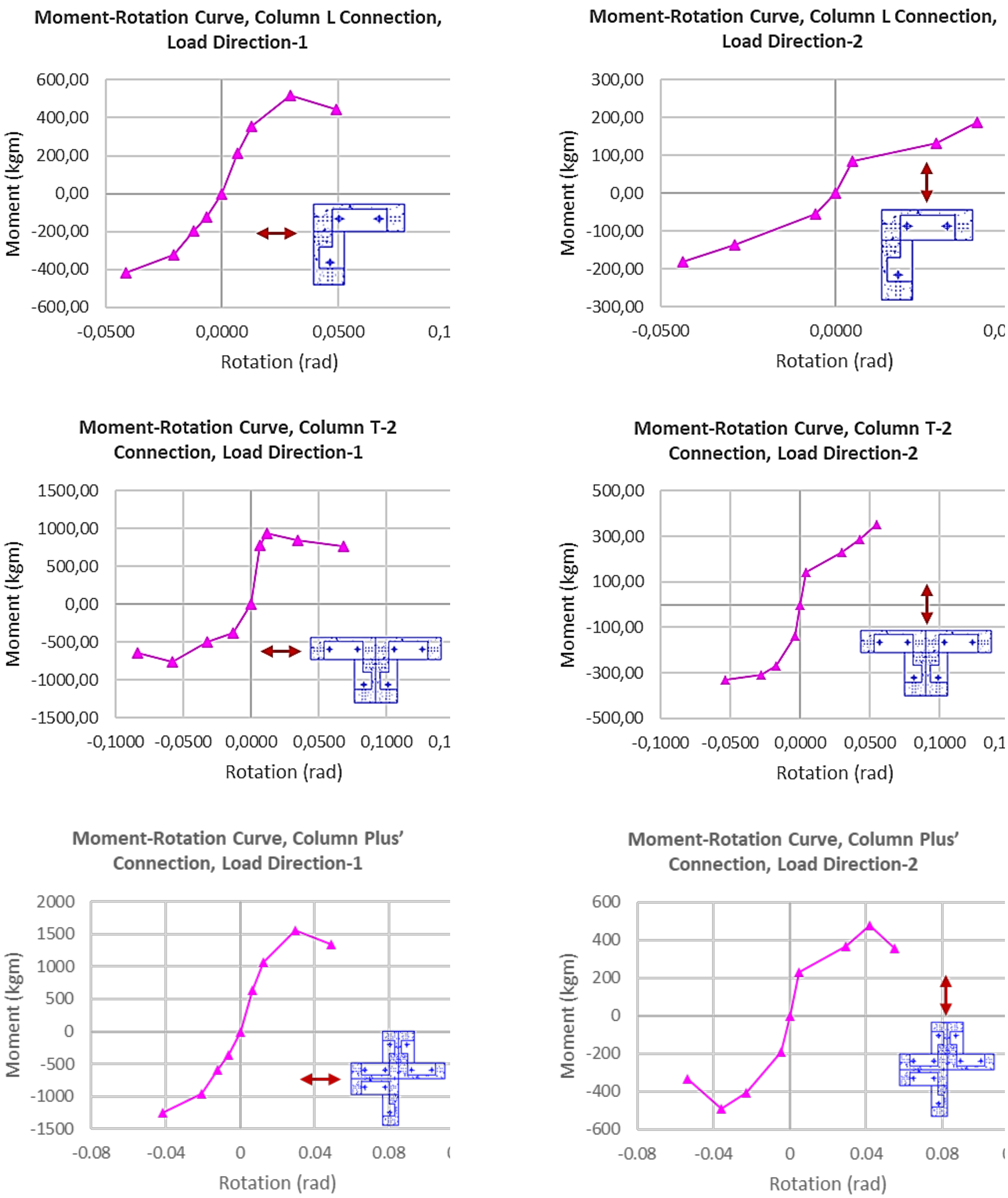


Fig. 4: The momen-rotation curves of several column types of the proposed system.

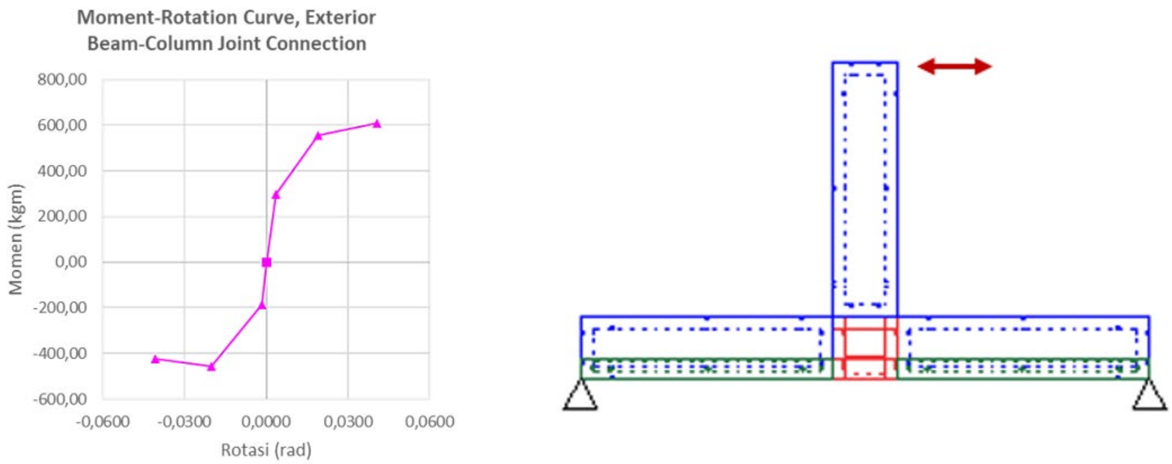


Fig. 5: The momen-rotation curves of exterior beam-column joint of the proposed system.

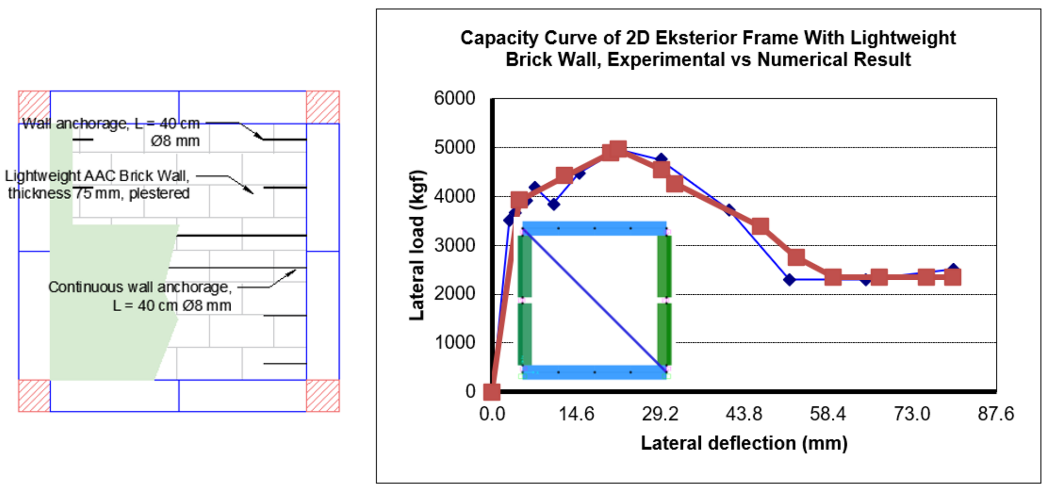


Fig. 6: Compression strut element calibration for lightweight brick wall modeling.

3.3. Lightweight Brick Wall Strength Contribution

The strength and stiffness of the infilled wall contribution on the overall building structure is considered by modelling a diagonal compression strut element inside a frame structure. The strut element behavior is calibrated by a cyclic loading test of 2D frame specimen with 75mm thick plastered lightweight AAC brick wall. The lightweight bricks wall are anchored to the frame structure as illustrated in Figure 6 (left). Figure 6 (right) shows the calibration result of the strut element where the numerical model capacity curve meet the experimental result. The opening of the wall is considered by applying a reduction factor of the strut element strength of 0.5 considering the ratio of opening in each structural frame in the school prototype to be proposed.

3.4. The Proposed Structural Configuration

Some structural configuration adjustments are proposed for the prototype of the school building. The strip plate dimension of the mechanical connection is recommended to be increased using a thickness of a minimum of 2.5 mm and approximately 40 mm wide. The minimum compressive strength of the concrete materials should be 25 MPa as stated in the official RISHA componet fabrication guideline. All of the techncial rules of RISHA connection system including column anchorage to foundation and wall anchorages should be followed.

In order to get better dynamic behavior for the overall building structure, the steel stiffener frames that are located on the building top elevation at the extended volumetric module of the structure are introduced. The stiffener frames consist of UNP 100x50x5x7.5 beams that are supported by longitudinal UNP 150x75x6.5x10 girders. The diagonal members using Pipe 3.5 inch on each corner of the modules are also installed. Figure 7 illustrates the configuration of the steel brace frames on the proposed school building with four class rooms. Additionally, some 5 mm thick steel plates installation at the joints of the perimeter interior columns at an elevation of +3.00 meters of the structure is also proposed. This led to the improvement of the joint rigidity of the column-to-column connections. The location and 3D visualization of the steel plate placements are presented in Figure 8.

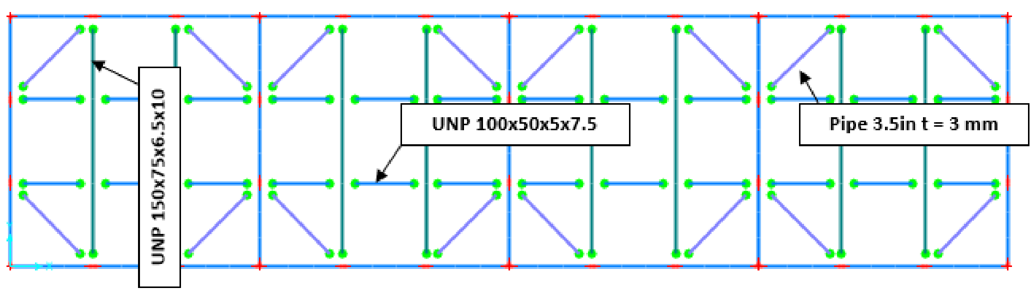


Fig. 7: Steel brace frames on the building top elevation for extended module of the proposed structure.

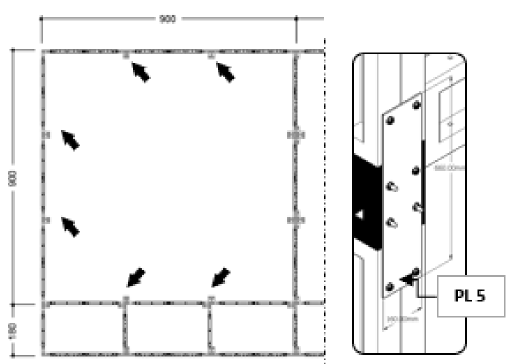


Fig. 8: Steel plates at the joints of the perimeter column at an elevation of 3 meters of the structure.

4. Results and Discussions

4.1. Structure Numerical Modelling

The structural performance of the proposed system is evaluated using the developed numerical modeling methodology for this precast structural system where the semirigid nonlinear behavior of each structural joint connections are modeled using nonlinear link elements that consists of the momen-rotation definitions related to each joint types as given in Figure 4 and Figure 5. The idealization method for the numerical model and the calibration result of the numerical model according to the capacity curve from the 2D frame cyclic test is depicted in Figure 9a.

In this research, the structural performance evaluation of protoypte building is conducted on the school building with four classrooms of $9x9x4.5\ m^3$ size with a corridor in front of the building as shown in Figure 9b. For a conservative reason, this prototype model is taken because it is expected to be the largest extended volumetric model for each classroom. The numerical modeling of the structure is conducted using SAP2000.

4.2. Pushover Analysis and Performance Evaluation

The result of pushover analysis of the prototype structure in both X and Y directions are presented in Figure 10. There is a difference in the stiffness between Push-X and Push-Y due to the asymmetric orientation of the structural columns and the difference in the masonry-infilled walls density in each direction. The structural performance evaluation will be conducted using ATC-40 and FEMA 440 Equivalent Linearization methods. The monitored joint displacement in each loading direction is selected to the joint with the largest deformation of the top elevation for conservative reason. However, the installed steel brace frame induce a diaphragm behaviour at the top elevation of the structure so the deformation of any joint nodes in the highest elevation of the structure are approximately the same.

There are six locations that are selected for structural performance evaluations. All of them are locations with severe earthquake hazard in Indonesia with values of $Ss \geq 0.911$ and $S1 \geq 0.391$ except Manokwari city. The interstory drift limit of 0.015 is taken for the Damage Control performance according to ATC-40 as shown in Table 1. The result of structural performance evaluation according to ATC 40 and FEMA 440 Equivalent Linearization methods are presented in Table 2 and Table 3 respectively, while the sample of performance point results calculated from the computer program are depicted in Figure 11 to Figure 14. It is seen that the prototype structure reaches the Damage Control performance (DC) which means that the structural components are expected not to experience damages after the design earthquake and only some minor repairs for nonstructural elements are necessary.

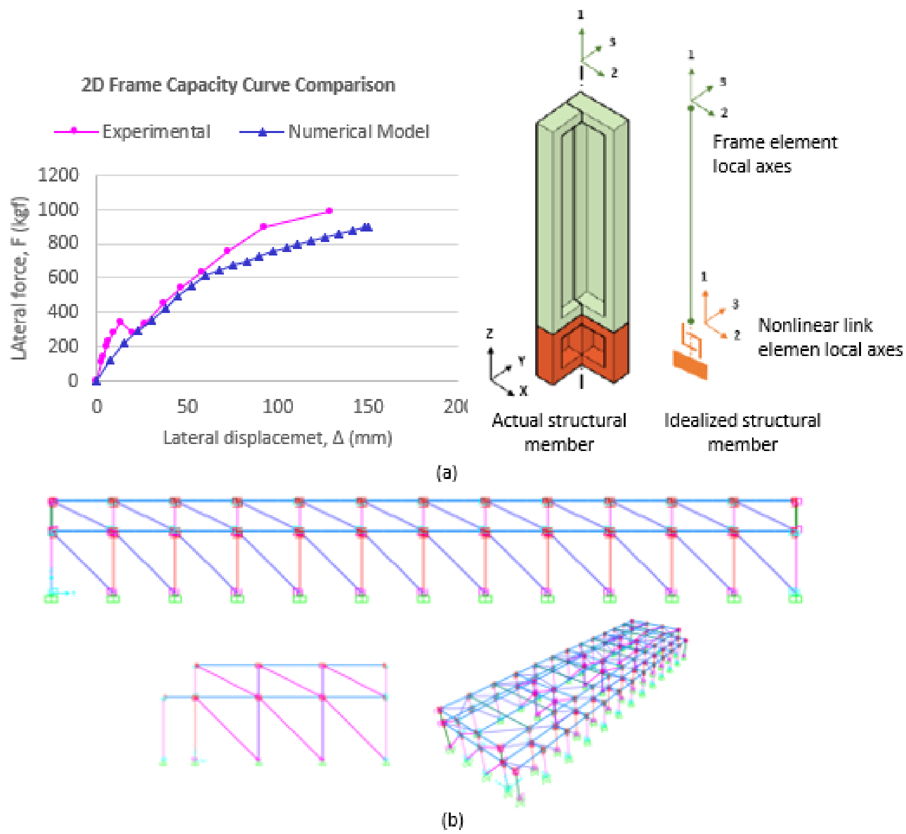


Fig. 9: (a) Nonlinear behaviour modelling and calibration for the numerical model using nonlinear link element for each joint; (b) Numerical model of the school building prototype with four 9x9x4.5 m^3 size for each classrooms.

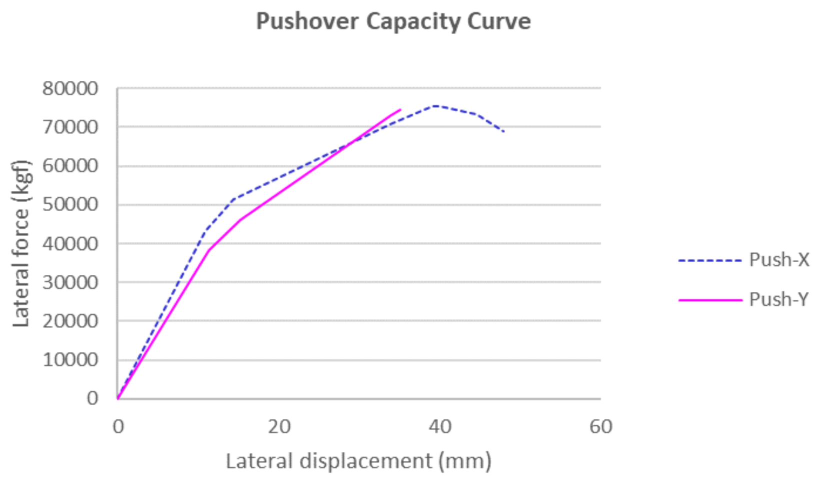


Fig. 10: Pushover capacity curve of the prototype model.

Table 1: Performance level limit according to ATC-40.

Interstory drift limit	Performance level			
	Immediate Occupancy	Damage Control	Life Safety	Structural Stability
Maximum total drift	0.01	0.01 - 0.02	0.02	0.33 Vi/Pi
Maximum inelastic drift	0.005	0.005 - 0.015	No limit	No limit

Table 2: ATC 40 performance evaluations.

No.	Lokasi	Coeff. ATC-40		Performance Point		d/h	Performance
		Ca	Cv	V (kgf)	d (m)		
1	Banda Aceh SD	0.570	0.998	72645.84	0.0340	0.0113	DC
	Banda Aceh SE	0.474	1.170	64516.04	0.0280	0.0093	DC
2	Bengkulu SD	0.600	1.020	74341.02	0.0350	0.0117	DC
	Bengkulu SE	0.480	1.200	65005.66	0.0280	0.0093	DC
3	Gorontalo SD	0.600	1.020	74341.02	0.0350	0.0117	DC
	Gorontalo SE	0.480	1.200	65005.66	0.0280	0.0093	DC
4	Jayapura SD	0.600	1.050	74341.02	0.0350	0.0117	DC
	Jayapura SE	0.540	0.495	70035.78	0.0320	0.0107	DC
5	Palu SD	0.600	1.020	74341.02	0.035	0.0117	DC
	Palu SE	0.480	1.200	65005.66	0.028	0.0093	DC
6	Mamuju SD	0.642	1.080	71949.43	0.034	0.0113	DC
	Mamuju SE	0.510	1.275	67489.83	0.03	0.0100	DC

Table 3: FEMA 440 equivalent linearization performance evaluations.

No.	Lokasi	Coeff. ATC-40		Performance Point		d/h	Performance
		Ca	Cv	V (kgf)	d (m)		
1	Banda Aceh SD	0.570	0.998	70425.59	0.0340	0.0113	DC
	Banda Aceh SE	0.474	1.170	74410.63	0.0350	0.0117	DC
2	Bengkulu SD	0.600	1.020	69094.13	0.0340	0.0113	DC
	Bengkulu SE	0.480	1.200	74227.86	0.0350	0.0117	DC
3	Gorontalo SD	0.600	1.020	69094.13	0.0340	0.0113	DC
	Gorontalo SE	0.480	1.200	74227.86	0.0350	0.0117	DC
4	Jayapura SD	0.600	1.050	69094.13	0.0340	0.0113	DC
	Jayapura SE	0.540	0.495	72099.53	0.0340	0.0113	DC
5	Palu SD	0.600	1.020	69094.13	0.034	0.0113	DC
	Palu SE	0.480	1.200	74227.86	0.035	0.0117	DC
6	Mamuju SD	0.642	1.080	67172.58	0.033	0.0110	DC
	Mamuju SE	0.510	1.275	72991.28	0.035	0.0117	DC

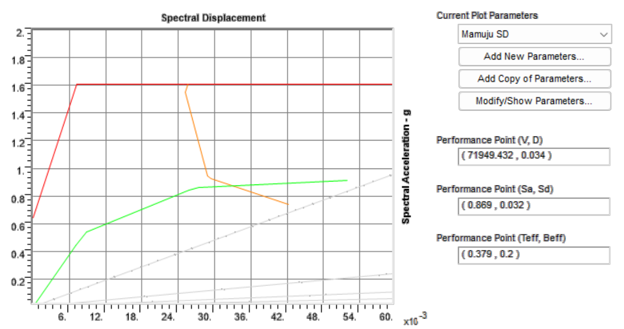


Fig. 11: ATC 40 performance point sample on Mamuju SD.

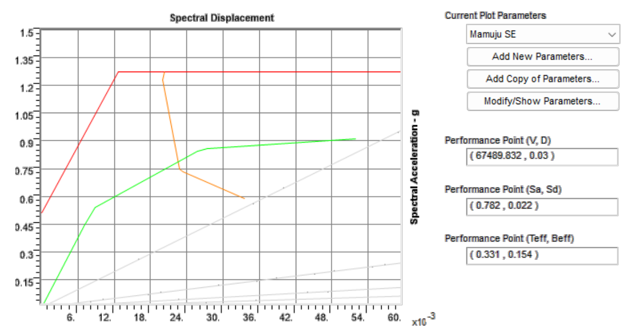


Fig. 12: ATC 40 performance point sample on Mamuju SE.

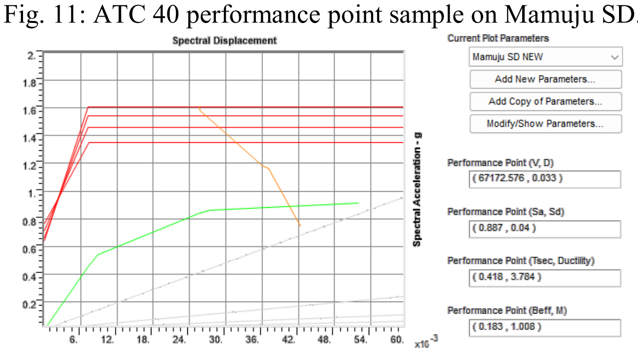


Fig. 13: FEMA 440 equivalent linearization performance point sample on Mamuju SD.

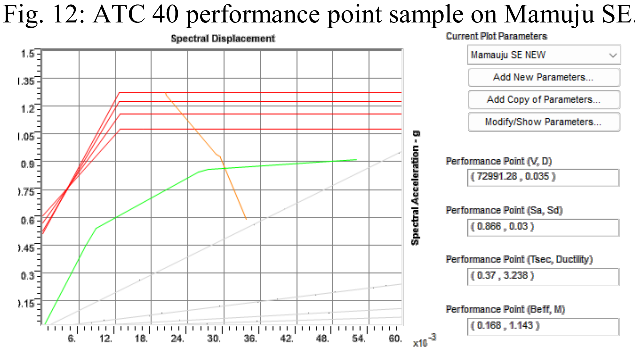


Fig. 14: FEMA 440 equivalent linearization performance point sample on Mamuju SE.

5. Conclusion

The performance point evaluations of the proposed precast concrete school building prototype using ATC-40 and FEMA 440 Equivalent Linearization methods in some Severe Earthquake hazard areas in Indonesia show that the performance of Damage Control (DC) was achieved. It means that the structural components are expected not to experience damages after the design earthquake and only some minor repairs for nonstructural elements are necessary.

However, the capacity curve of the numerical model that is obtained in this research is still considering the poor quality of concrete materials. By updating the moment-rotation and other material properties definitions such as the concrete compressive strength of a minimum 25 MPa as specified in the proposed technical guideline, would lead to higher structural capacity curve.

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