Evaluation of Notional Loads Magnitude to Three Calibration Moment Resisting Frames Subjected to Indonesian Seismic Load

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***Abstract*—AISC 2010 has been adopted on Indonesian steel design code since 2015. AISC 2010 uses Direct Analysis Method (DAM) for steel structural stability where it substitutes Effective Length Method (ELM). DAM is a second order elastic analysis, allows a more accurate determination of the load effects in the structure through the inclusion of the effects of geometric imperfections and stiffness reductions directly within the structural analysis. Notional load as 2 per mill of gravity load should be applied horizontally to represent geometric imperfection of 1/500L. It is allowed to adjust the notional load coefficient proportionally based on a nominal initial story out-of-plumbness ratio. Results from three different calibration frames from previous research which are considered as advanced analysis were used as references. Through numerical simulation by using SAP software, the advanced analysis considered as a second order inelastic method enabling to accommodate the real collapse mechanism of structure will be validated through three calibration frames. Evaluation studies were first conducted to compare ELM and DAM effectiveness, and later to find out the appropriate magnitude of notional load on steel moment resisting frame subjected to Indonesian Seismic Load. The calibration frames consisted of one story, 3-stories and 6-stories were reanalyzed with four different methods: ELM first order analysis, ELM second order analysis, DAM with different notional loads coefficient as 0.002 and 0.003; and Response Spectrum taking into account the two different notional loads coefficients. Indonesian seismic load in three seismic zones with three different soil conditions were considered. The results were compared to advanced analysis. It is found that DAM has the closest result to advanced analysis and notional load coefficient of 0.003 reveals as the most appropriate value considered from its base shear-drift curve, P-M interaction and drift.**

***Index Terms*—effective length method, direct analysis method, response spectrum, moment resisting frames, calibration frame**

I. INTRODUCTION

The development of steel design method has become more rapid affected by development of computer technology. Long time ago analysis of steel structure was conducted by hand calculation, hence it needs some simplification in calculation. However, the simplification was far from ideal condition where sometimes it does not meet real condition. Several corrections have been made to the assumption taken for steel structural analysis during the time.

AISC has been update several times until it launched the latest version, AISC 2010 as a correction of AISC 2005. Indonesian design code for steel building of SNI 03-1729-2015 is a translation version of AISC 2010. One of significant correction from AISC 2005 to 2010 is steel structure stability analysis. AISC 2005 has effective length method (ELM) as the main method as mentioned in Chapter C: Design for Stability [1] and AISC 2010 has direct analysis method (DAM) as the main method as mention in Chapter C: Design for Stability [2].

Both ELM and DAM use column interaction equations to estimate the capacity of individual steel columns [3]. In addition, both use second order elastic analysis where ELM approximates the P-delta effect by using amplification factor and the value of effective length factor (K) due to buckling of compression member is estimated based on relative rigidity between girder and column at both ends. With the development of computer, DAM takes into account the P-delta effect directly in the analysis and hence the K value is set as 1.

In addition, DAM accommodates geometric imperfection and strength reduction during analysis. Use of notional loads to represent geometric imperfection is described on section C2.2b AISC 2010. The loads shall be applied as lateral loads at all structural level. The notional load coefficient of 0.002 in Equation C2-1 is based on a nominal initial story out-of-plumbness ratio of 1/500 of column length. According to AISC, it is permissible to adjust the notional load coefficient proportionally.

Several researches have been done to see the effect of ELM and DAM to structural steel design [3,4]. The results found that ELM has higher stress ratio on 1-storey with 1 bay steel structure [3]. DAM has its advantages which are simpler to applied and more accurate for case which have significant second order effect. Another test results of 1 story and 3 bays steel building confirmed that finding [4]. None of the research has been done to simulate the magnitude of notional loads.

Since Indonesia is located in ring of fire zone, hence evaluation study to evaluate the appropriate magnitude of

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notional load on steel frame subjected to Indonesian Seismic Load was carried out. The Indonesian archipelago which is located of three major tectonic plates (The Indo-Australian, Pacific and Eurasian plates) makes the amount of earthquake hazards in this region is high based on its high subduction related seismic [5]. Analysis based on ELM and DAM were conducted to see the significant changing of structural response to both methods in term of axial force and bending moment (PM) interaction, and drift.

The analysis was compared to advanced analysis obtained from calibration moment resisting frames published by other researchers [6,7]. The frames consisted of one story, 3-stories and 6 stories were reanalyzed with four different methods: ELM first order elastic analysis, ELM second order elastic analysis, DAM with two different coefficient of notional loads as 0.002 and 0.003; and Response Spectrum taking into account the two different notional loads coefficients.

The Indonesian seismic load in three zones (Samarinda, Jakarta and Padang) with three different soil conditions (soft, medium and hard) were considered.

II. VALIDATION OF FE MODEL

The numerical study was conducted using SAP software [8]. The use of Finite Element (FE) software is common in understanding the frame response to a specific load case [9].

To validate the numerical model, calibrations were performed against calibration frames. The frames consisted of 3 different stories, 1, 3 and 6 published by different researchers. Fig. 1, 2 and 3 show configurations of those frames.

The 1 and 3 stories frame taken from a set of calibration frames in North America which was selected for second order inelastic analysis [6,7] as a benchmark to verify FE model or computer programs, and more importantly to have interaction values from a column subject to bending and axial compressive load. The 1 and 3 stories are named as El-Zanaty and Yarimci, respectively according to name of researcher conducted the experimental test.

According to the experimental test, the material of steel like stress-strain curve and hinge property have been stated, in order to represent the non-linier material of analysis.

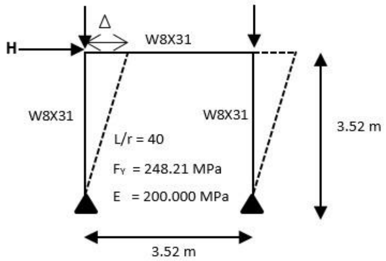

Figure 1. Calibration frame 1 [6]

The six-story frame shown in Fig. 3 was proposed by Vogel as one of three frames for verifying the reliability and accuracy of second-order inelastic analysis programs [10]. The three calibration frames with different stories were modeled and reanalyzed. Non-linearity in geometry and material were taken into account to represent the non-linier inelastic analysis. The one, three and six stories frames hereafter are called as calibration frame 1, 3 and 6 respectively and the numerical model represent those frames are called as advanced analysis.

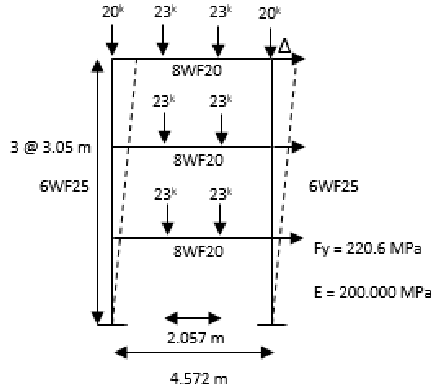

Figure 2. Calibration Frame 3 [6]

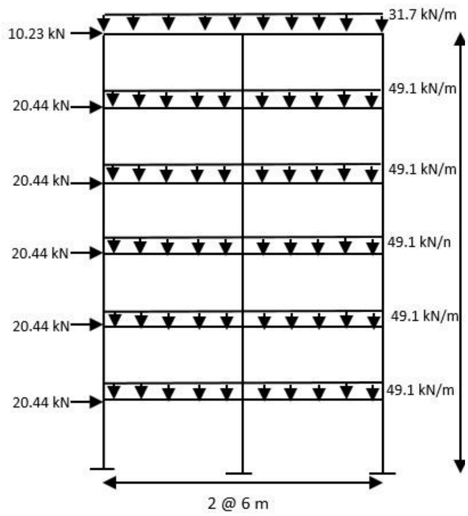

Figure 3. Calibration Frame 6 [7]

Calibration frame 1 and 3 was loaded vertically to present the gravity load. Horizontal force was increased until the frames reached the maximum capacity. The vertical load is varied as a ratio to its yield force (P_y), which were P/Py 0,2; and 0.6.

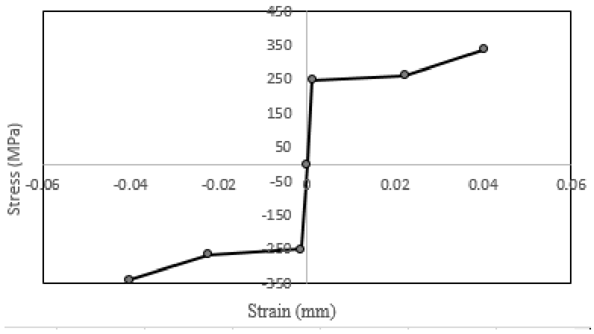

Figure 4. Stress-Strain Relationship of A36 Steel

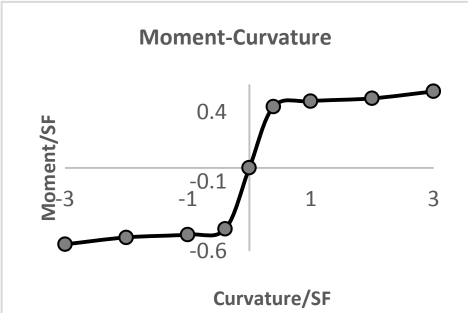

Figure 5. Moment-Curvature of Beam Column Joint

All experimental data gathered from calibration frame 1 were input on FE model which were: the tri-linier stress strain relationship of A36 steel material, as shown in Fig. 4, and moment-curvature of beam-column joint as presented in Fig. 5.

Residual stress was not considered since it is not supported by the software. Beam plastic hinges at beam edges were assigned and P-delta plus large displacement analysis were carried out.

Figs. 6 and 7 show comparison of load deflection curve between SAP numerical model and experiment of calibration frame 1.

As can be seen, the result of numerical model is close to experimental result. Hence, the model can be used for further study.

Validation results of calibration frame 3 are shown on Fig. 8. Similar with 1 story, non-linier inelastic analysis was conducted by taking into account non-linier material and P-delta effect plus large deformation. Push over analysis was carried out and plastic hinges on beam end and column end were assigned.

Due to equipment problem at the initial stage of experimental test, only half of the curve was obtained. The dashed curve in Fig. 8 is an approximation.

As shown in the figure, the results between numerical model and test is slightly different. In general, numerical simulation using SAP can represent the experimental test.

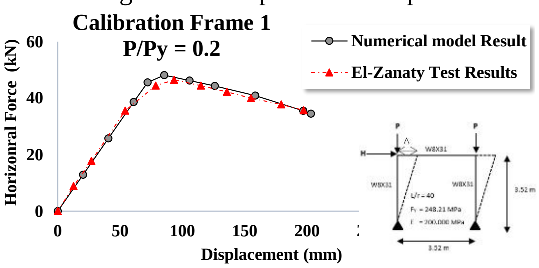

Figure 6. Validation of Numerical Model against Calibration Frame 1 (P/Py=0.2)

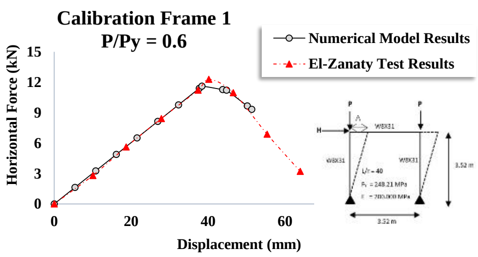

Figure 7. Validation of Numerical Model against Calibration Frame 1 (P/Py=0.6)

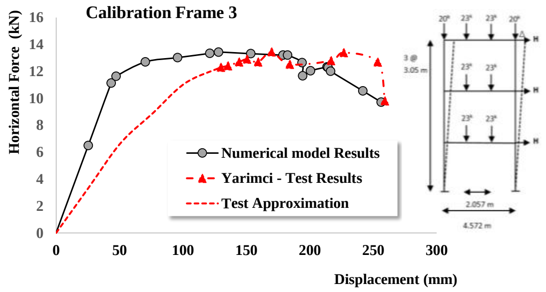

Figure 8. Validation of Numerical Model against Calibration Frame 3

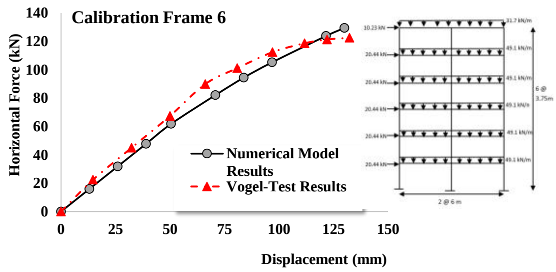

Figure 9. Validation of Numerical Model against Calibration Frame 6

Vogel's six stories calibration frame [8] and [9] was intended to verify second-order non-linier inelastic analysis and hence the numerical model should consider non-linier material and P-delta effect plus large deformation. Imperfection of 1/450L at each level was also take into account. The difference between FE model and Vogel's-frame is initial residual stress that is not supported by SAP software. Push-over analysis result of numerical analysis is plotted on force-deflection curve as shown in the Fig. 9. Similar with two preceding calibration frames, close results between numerical analysis and calibration frame is shown and hence the model is valid and can be used for further analysis. Since the model can represent the second-order inelastic analysis where the non-linearity on geometric and material are considered, hereafter the validated models are named as advanced analysis.

III. Comparison Study of DAM and ELM

After the validation stage has been done, the calibration frames were analyzed by ELM and DAM. The objective is to evaluate the significant changing of structural response to both methods. Two variations of notional loads of 0.002 and 0.003 from gravity load were included in the analyses**.** Based on Appendix 8 AISC 2010 [2], ELM can be referred as a first order elastic analysis where moment due to P-delta effect is calculated based on amplification factor. ELM can also be analyzed as second order analysis if the P-delta effect is included in the analysis. In order to see the different effect of first order and second order analysis of ELM, the calibration framed were analyzed by both methods. The development of computer and structural analysis software assist the structural analysis process and hence, the P-delta effect can be directly calculated.

The method is named as direct analysis method (DAM). It includes geometric imperfection as notional load and structural stiffness reduction by 20% than initial stiffness. As mentioned earlier, AISC allows to adjust the notional load coefficient. A study of the effect of two different notional loads value was performed. The advanced analysis based on validation models is chosen as a benchmark. Push-over analysis is carried out to three calibration frames. The push over force is calculated based on limitation of ELM where the ratio of second-order to first-order analysis is less or equal to 1.5. Hence the maximum push over forces for each calibration frame are as follow: 106.04 kN and 97.54 kN for calibration frame 1 with P/Py equal to 0.2 and 0.6 respectively, whereas calibration frame 3 and 6 are 36.38 kN and 246 kN. P is an external load given by the P/Py ratio. Results of push over analysis are plotted on base-shear versus drift at top story as presented on Fig. 10 to Fig. 13.

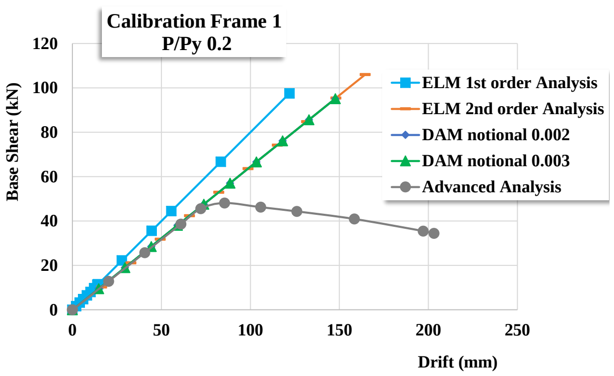

Figure 10. Comparison results of calibration frame 1 (P/Py=0.2) analyzed by ELM, DAM and Advanced Analysis

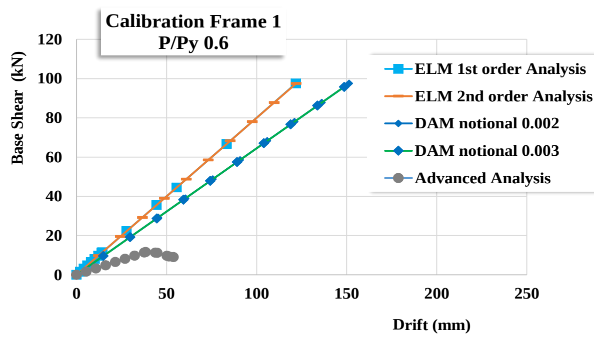

Figure 11. Comparison results of calibration frame 1 (P/Py=0.6) analyzed by ELM, DAM and Advanced Analysis

As shown in Fig. 10 and 11, ELM first order and second order analysis has the same result. DAM with different notional load also has the same fact. However, DAM has closer results to advanced analysis than ELM. Similar trend is also found on calibration frame 3 as shown in Fig. 12. It is found that there is a slightly effect of notional load magnitude as can be seen from curve of DAM with notional load coefficient as 0.003 is closer to advanced analysis. Figs. 10 and Fig 11 also show that gravity load affect the maximum base shear value. As the P/Py increases from 0.2 to 0.6, then the maximum base shear of advanced analysis reduced by 73.53% from 46.51kN to 12.29 kN.

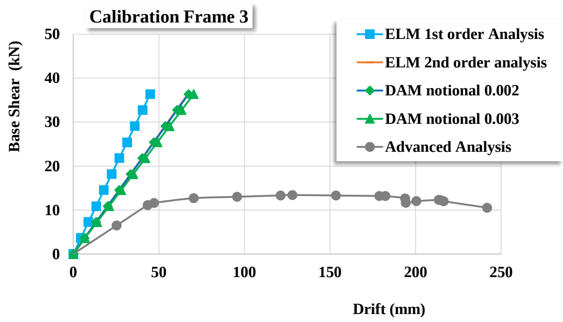

Figure 12. Comparison results of calibration frame 3 analyzed by ELM, DAM and Advanced Analysis

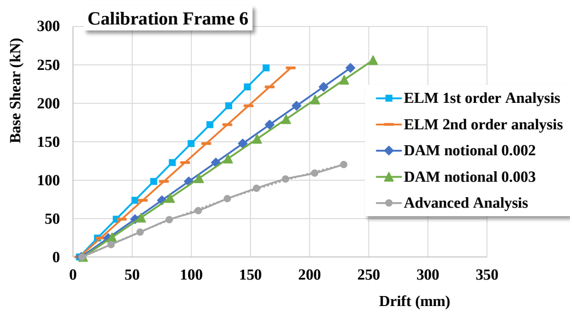

Figure 13. Comparison results of calibration frame 6 analyzed by ELM, DAM and Advanced Analysis

Calibration frame 6 confirms the facts found on calibration 1 and 3 where DAM can simulate real structure better than ELM. As shown on Fig. 13, ELM second order analysis is more accurate revealed from its curve that is more close to advanced analysis than ELM first order. Among all, the closest result is found on DAM with notional coefficient as 0.003.

It can be concluded that DAM is more precise in presenting the real structure than ELM. Furthermore, notional load coefficient as 0.003 is suggested to be used than 0.002.

IV. Calibration Frames Subjected to Indonesian Seismic Zone Loads

As concluded earlier that DAM with 0.003 as notional load coefficient is more precise in describing the real structure than ELM. However, the study of 3 calibration frame was based on push over analysis.

Further study to investigate the effect of notional load magnitude was carried out against Indonesian seismic load.

The frames were simulated in 3 different seismic zones in Indonesia from the lightest, medium to the strongest seismic load which are represented by Samarinda, Jakarta and Padang, respectively.

Referring to experimental and numerical analysis of calibration frames as shown in Figs. 1 - 3, only dead load and horizontal loads applied on structure. In this study, horizontal forces consist of notional load and earthquake load as in (1), (2) and (3).

Therefore, load combinations for dead load (D), notional dead load (ND), and earthquake (E) according to ASCE [11] are as follow:

C1 : 1.4D + 1.4ND (1)
C2 : 1.2D + 1.0E (2)
C3 : 0.9D + 1.0E (3)

In this case, the load values of each load patterns are 802.41 kN for dead load including self-weight and external force load of the structure, 2.4 kN for notional dead load based on 0.003 times of dead load and 36.43 kN for earthquake load based on base shear of structure.

TABLE I. ANALYSIS RESULTS OF CALIBRATION FRAME 3

Calibration Frame 3 DAM_0.003	Load Combination		
	C1	C2	C3
Pu (kN)	566.50	524.95	401.30
Mu (up) kNm	51.80	67.79	55.50
Mu (bottom) kNm	30.65	67.50	58.87
Py (kN)	1044.65		
PM Interaction	1.50	1.67	1.38
Pu/Py	0.54	0.50	0.38

where:
Pu : Factored axial force
Mu : Factored bending moment
Py : Axial compressive yield strength
PM : Axial force and bending moment

All frames are pushed until failure. Horizontal forces presenting earthquake load is determined based on limitation of ELM where the ratio between second order analyses to first order is less or equal to 1.5. The earthquake load is calculated based on SNI 1729-2012 [11] and ASCE 7-16 [12] using equivalent static seismic analysis as follow:

$$F_x = C_{vx} V \tag{4}$$

$$V = C_s . W \tag{5}$$

where:
C_{vx} : Distribution factor for each floor
W : Effective weight of the structure,
C_s : Seismic response factor
W : Total effective weight of structure
F_x : Seismic load for x-story
V : Base shear of structure

Among the three load combinations, C2 produces the highest PM interaction. This situation is also occurred to all calibration frames, and hence only PM interaction of calibration frame 3 is presented as can be seen from Table I. Hereafter, only load combination 2 is used for seismic simulation for three seismic zones. The results are presented on PM interaction and drift (lateral displacement of top story). PM interaction formula is determined in AISC 360-10 Chapter H [2], based on factored axial force Pu and bending moment Mu compared to its each nominal strength Pn and Mn multiplied by its resistance factors, compressive and bending, ϕ_c and ϕ b as follow:

For ratio of Pu /ϕ_cPn ≥ 0.2

Interaction Formula:

$$Pu/ (/\phi_c Pn) + 8Mu/(9\phi_b Mn) \leq 1 \tag{6}$$

For ratio of Pu / ϕ_c Pn < 0.2

Interaction Formula:

$$Pu/(2\phi_c Pn) + Mu /(\phi_b Mn) \leq 1 \tag{7}$$

Analysis results of all calibrated frames in three seismic zones and three different soil conditions are discussed in the next section. According to Indonesian Seismic Design Code, the symbol for hard soil, medium soil and soft soil are SC, SD and SE, respectively.

V. PM-INTERACTION

Table II to V present PM-interaction for all calibration frames. Magnitude of earthquake loads applied on each frame according to equation 4 is also presented as equivalent static loading and it is also dynamically analyzed using response spectrum (RS) analysis according to each spectrum of the corresponding seismic zones.

Based on the advanced analysis, all calibration frames do not meet the seismic loads in Padang zone since it exceeds the maximum base shear of all calibration frames and it is noted as "n/a".

The maximum base shear values for all frames refer to the curves shown on Fig. 8 – 11. None of the earthquake loads of calibration frame 1_P/Py=0.6 below the maximum base shear of advanced analysis due to higher gravity load.

TABLE II. PM INTERACTION OF CALIBRATION FRAME 1 (P/PY=0.2)

Seismic Zone	Samarinda			Jakarta			Padang		
Soil Type	SC	SD	SE	SC	SD	SE	SC	SD	SE
Earthquake Loads (kN)	6.79	9.06	14.06	39.4	44.83	50.95	78.91	78.91	71.04
Adv Analysis	0.48	0.52	0.61	1.1	1.16	n/a	n/a	n/a	n/a
DAM _0.003	0.46	0.5	0.58	0.99	1.08	1.18	1.64	1.64	1.51
DAM _0.002	0.45	0.48	0.56	0.94	1.02	1.11	1.53	1.53	1.41
ELM 2nd Order	0.45	0.48	0.56	0.94	1.02	1.11	1.53	1.53	1.41
ELM 1st Order	0.45	0.48	0.56	0.94	1.02	1.11	1.53	1.53	1.42
RS_0.003	0.37	0.37	0.37	0.4	0.4	0.41	0.45	0.44	0.43
RS_0.002	0.36	0.36	0.36	0.39	0.39	0.4	0.44	0.43	0.42

TABLE III. PM INTERACTION OF CALIBRATION FRAME 1 (P/PY=0.6)

Seismic Zone	Samarinda			Jakarta			Padang		
Soil Type	SC	SD	SE	SC	SD	SE	SC	SD	SE
Earthquake Loads (kN)	20.16	26.88	42	116.9	133.1	151.2	234.2	234.2	210.6
Adv Analysis	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
DAM _0.003	1.37	1.48	1.74	2.99	3.26	3.57	4.96	4.96	4.56
DAM _0.002	1.33	1.43	1.66	2.78	3.02	3.3	4.54	4.54	4.18
ELM 2nd Order	1.19	1.25	1.37	1.98	2.11	2.25	4.85	4.85	4.46
ELM 1st Order	1.19	1.25	1.37	1.97	2.09	2.25	4.56	4.56	4.2
RS_0.003	1.08	1.08	1.09	1.14	1.14	1.15	1.21	1.21	1.17
RS_0.002	1.05	1.05	1.06	1.11	1.12	1.13	1.19	1.19	1.16

TABLE IV. PM INTERACTION OF CALIBRATION FRAME 3

Seismic Zone	Samarinda			Jakarta			Padang		
Soil Type	SC	SD	SE	SC	SD	SE	SC	SD	SE
Earthquake Loads (kN)	81.59	108.8	170	473.2	538.4	611.9	947.7	947.7	853.2
Adv Analysis	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
DAM _0.003	2.56	3.1	4.3	10.26	11.54	12.98	19.58	19.58	17.73
DAM _0.002	2.56	3.1	4.3	10.25	11.53	12.97	19.56	19.56	17.71
ELM 2nd Order	2.41	2.9	3.99	9.43	10.61	11.92	17.95	17.95	16.25
ELM 1st Order	1.89	2.11	2.61	5.08	5.61	6.21	8.94	8.94	8.17
RS_0.003	1.2	1.41	1.87	4.07	4.58	5.25	7.61	7.68	7.18
RS_0.002	1.17	1.38	1.83	3.99	4.5	5.19	7.51	7.59	7.13

Only calibration frame 1_P/Py=0.2 and calibration frame 6 which have advanced analysis results that can be used to see which methods has the accurate PM-interaction.

As presented on Table V, DAM_0.003 is slightly closer to advanced analysis than DAM_0.002. Similar PM–interaction is found on DAM_0.002, second order ELM and first order ELM.

However, other calibration frames show different results, where first order ELM has the smallest PM-interaction whereas DAM_0.003 has the highest. Besides, response spectrum linier analysis produces the smallest PM-interaction among all methods. The SRSS modal response combination is taken as the first 3 dominant frequencies that occur on the structure are well separated.

Among all frames, only calibration frame 1 in Samarinda zone meets design criteria which its PM-interaction is less than 1 as presented on Table II. For seismic zone Jakarta with hard soil (SC), it is found that the frame faintly exceeds its maximum capacity based on advanced analysis, but DAM and ELM perceive this frame is still adequate.

TABLE V. PM INTERACTION OF CALIBRATION FRAME 6

Seismic Zone	Samarinda			Jakarta			Padang		
Soil Type	SC	SD	SE	SC	SD	SE	SC	SD	SE
Earthquake Loads (kN)	77.77	103.7	162	451	513.2	583.2	903.3	903.3	813.3
Adv Analysis	1.13	1.24	n/a	n/a	n/a	n/a	n/a	n/a	n/a
DAM _0.003	1.13	1.24	1.49	2.73	3	3.3	4.68	4.68	4.29
DAM _0.002	1.11	1.22	1.32	2.72	2.98	3.28	4.66	4.66	4.27
ELM 2nd Order	1.16	1.27	1.3	2.69	2.95	3.24	4.55	4.55	4.18
ELM 1st Order	1.16	1.23	1.28	2.17	2.34	2.52	3.39	3.39	3.15
RS_0.003	1.1	1.1	1.13	1.55	1.55	1.58	2.85	2.85	2.8
RS_0.002	1.09	1.09	1.12	1.53	1.54	1.56	2.38	2.38	2.3

n/a means PM value is not available, as the maximum base shear resulted from the advanced analysis is far below the earthquake loads..

As predicted, higher seismic load leads to higher PM-interaction. In addition, except Padang, hard soil (SC) gives results of lower seismic load leads to lower PM-interaction compared to medium and soft soil, as can be seen on Table II to V.

VI. DRIFT

Comparison of allowed horizontal displacement at top story of all frames is displayed on Table VI. Refer to results of PM interaction showed on Table II to Table V, only drift of calibration frame 1 and 6 is presented here without Padang zone and the results are limited to seismic zone where its drift below or slightly above the allowable value. The value of allowable drift that takes into account based on SNI 1729-2012[11] and ASCE 7-16, section 12.12 [12].

TABLE VI. ALLOWABLE DRIFT FORMULA

Allowable Story Drift			
Structure	Risk Category		
	I or II	III	IV
Structures, other than masonry shear wall structures, four stories or less above the base as defined in Section 11.2, with interior walls, partitions, ceilings, and exterior walls system that have been designed to accommodate the story drifts	$0.025h_x$	$0.02h_x$	$0.015h_x$
Masonry cantilever shear wall structures	$0.01h_x$	$0.01h_x$	$0.01h_x$
Other masonry shear wall structures	$0.007h_x$	$0.007h_x$	$0.007h_x$
All other structures	$0.02h_x$	$0.015h_x$	$0.01h_x$

Where h_x : the story height below level x

Based on Table VI, the allowable drift taken is $0.02h_x$ in regard of the type of structure, which is taken as all other structures, as classified in risk category type II. As shown in Table VII, confirming the preceding result, drift predicted based on DAM_0.003 analysis have the closest value to advanced analysis.

TABLE VII. DRIFT OF CALIBRATION FRAME 1

Seismic Zone	Samarinda			Jakarta		
Soil Type	SC	SD	SE	SC	SD	SE
Calibration Frame 1 : P/Py= 0.2 : Allowable Drift = 70.51 mm						
Adv Analysis	12.12	15.75	23.14	61.84	*73.87*	*90.5*
DAM _0.003	11.43	13.97	22.1	61.21	*71.12*	*83.82*
DAM _0.002	10.52	12.7	20.32	60.96	68.58	*78.74*
ELM 2nd Order	10.67	14.22	21.84	60.96	69.6	*78.99*
ELM 1st Order	8.38	11.18	17.53	49.28	55.88	63.5
RS_0.003	3.15	3.28	3.72	6.28	7.26	8.87
RS_0.002	2.56	2.4	2.85	6.12	6.43	7.67

TABLE VIII. DRIFT OF CALIBRATION FRAME 1 AND 6

Seismic Zone	Samarinda			Samarinda		
Soil Type	SC	SD	SE	SC	SD	SE
	Calibration Frame 1 : P/Py = 0.6 Allowable Drift = 70.51 mm			**Calibration Frame 6 Allowable Drift = 75 mm**		
Adv Analysis	n/a	n/a	n/a	*133.45*	*187.57*	*n/a*
DAM _0.003	31.28	41.71	65.15	*82.91*	*107.72*	*163.53*
DAM _0.002	31.27	41.7	65.15	61.92	*80.7*	*123.04*
ELM 2nd Order	25.2	33.61	52.5	61.87	*80.7*	*123.04*
ELM 1st Order	25.19	33.6	52.49	54.97	71.64	*109.13*
RS_0.003	8.76	8.96	9.64	50.98	56.48	*106.84*
RS_0.002	6.1	6.3	6.99	49.87	53.48	*105.01*

Table VII and VIII only show the seismic zone that suits most of the allowable drift of each structure. Therefore frame 3 will not be shown regarding there is none of drift that suits the allowable drift of the structure.

VII. Conclusion

Based on base shear vs drift curve, the closest graph with the advanced analysis belongs to DAM with notional load coefficient as 0.003. Slightly different result is found between DAM_0.003 and DAM_0.002 in term of its PM interaction. DAM_0.003 can also predict drift of top story better than ELM. It can be concluded that DAM could represent actual structure better than ELM.

None of the frames has enough strength against seismic loads in Padang which is consider as one of the strongest seismic zone in Indonesia. It can be explained that all frames are ordinary moment resisting frame with low ductility. Only calibration frame 1_P/Py=0.2 in Samarinda can meet the design criteria due to seismic load in Indonesia revealed from PM interaction and drift value.

Conflict of Interest

The authors declare no conflict of interest.

Author Contributions

Heru Purnomo was the coordinator of the research group; Reza Agus Kurniawan conducted the research; Mulia Orientilize, Heru Purnomo and Sjahril A. Rahim analyzed the data; Heru Purnomo, Mulia Orientilize and Reza Agus Kurniawan wrote the paper; all authors had approved the final version.

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