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Performance Assessment of Linear Dynamic Procedures for Hospital Buildings with Concrete Moment Frame Systems in Indonesia

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Abstract: The infrastructure damage caused by the 7,5M_w earthquake in Palu on September 28, 2018, underscores the critical importance of evaluating the seismic resilience of public facility buildings, including the Tadulako University Hospital. To assess the feasibility and seismic vulnerability level of the structure, an evaluation was conducted based on prevailing standards, specifically ASCE/SEI 41-17. This study discusses the evaluation of the existing hospital building, focusing on the performance of its structural components under seismic loading. Referring to ASCE/SEI 41-17, two levels of seismic hazard were utilized: BSE-1E (225-year return period) and BSE-2E (975-year return period), with their respective target performance levels set as Immediate Occupancy (IO) and Life Safety (LS). The evaluation procedure employed the Linear Dynamic Procedure (LDP) via modal response spectrum (MRS) analysis. The research findings identified failures in several column and beam elements to satisfy the acceptance criteria, indicating potential deficiencies in structural performance. Consequently, the existing building is declared non-compliant with both the IO (BSE-1E) and LS (BSE-2E) target performance levels.

Keywords: *Hospital Building; Seismic Hazard; Performance Levels; Linear Dynamic Procedure.*

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I. INTRODUCTION

The vulnerability of Palu City to natural disasters, particularly earthquakes, is attributed to its location along the Palu-Koro strike-slip fault zone. Striking evidence of this is the 7.5M_w earthquake on September 28, 2018 [1]. This event triggered a tsunami and soil liquefaction that resulted in massive casualties [2] and paralyzed various public infrastructure, ranging from hospitals, airports, and ports to roads and bridges. Given the high vulnerability of this region to seismic activity, implementing earthquake-resistant design methods in infrastructure is an urgent necessity to mitigate the risk of structural failure. However, the seismic reality on the ground often triggers massive structural damage or even total collapse. This phenomenon not only threatens human lives but also implies a staggering surge in estimated rehabilitation and reconstruction costs, reaching billions of rupiah [3].

Current building infrastructure is required to meet seismic-resistant design criteria, particularly for high-risk category buildings such as hospitals, which must have a conditional probability of failure of less than 2.5% under Risk-Targeted Maximum Considered Earthquake (MCE_R) ground motions [4]. One of the affected structures is the Tadulako University Hospital, located in Palu, Indonesia. The building was designed as a reinforced concrete structure using legacy seismic codes, which consequently suffered damage leading to a complete loss of operational functionality during the 2018 Palu Earthquake (Figure 1). This inevitably raises a critical question regarding how well existing structures designed under older regulations would perform when subjected to current seismic loads, specifically referencing SNI 1726:2019. Therefore, a structural evaluation of the existing building is essential to mitigate the risk of structural failure induced by updated seismic demands, thereby ensuring compliance with the performance-based design targets appropriate for its risk category and specific seismic hazard level [5].

This paper describes a study on structural assessment procedures to identify the seismic vulnerability of an existing hospital building by applying the Linear Dynamic Procedure (LDP) as regulated in ASCE/SEI 41-17 [6]. This standard serves as the reference for numerical modeling and determining the acceptance criteria in linear analysis to identify underperforming structural components and evaluate the overall building performance. Consequently, in accordance with the building code, the evaluation stages are classified into four categories: evaluation of deformation-controlled components for column axial-biaxial moment (PMM) combinations, evaluation of force-controlled components for

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column shear, evaluation of deformation-controlled components for beam moments, and evaluation of force-controlled components for beam shear.

Fig 1 Severe Failure of The Structural Columns at The *Tadulako* University Hospital

II. RESEARCH METHODS

➢ *Structural Performance Criteria for Existing Buildings*

ASCE/SEI 41-17 requires the application of performance-based design concepts for existing buildings by selecting one or more target Building Performance Objectives, which consist of a Target Building Performance Level and a Seismic Hazard Level. The building function of the research object is a hospital, which falls under Risk Category IV [7]. In accordance with the provisions of ASCE/SEI 41-17, the seismic performance evaluation of this structure encompasses two limit states: Immediate Occupancy (IO) performance for the BSE-1E hazard level (20% probability of exceedance in 50 years) and Life Safety (LS) performance for the BSE-2E hazard level (5% probability of exceedance in 50 years).

➢ *General Overview of the Existing Building*

The Tadulako University Hospital building (Figure 2), which serves as the object of this study, features a structural configuration consisting of two building wings separated by an expansion joint. The structure stands at a height of 30.20 meters, with horizontal plan dimensions of 69.0 meters along the X-axis and 24.0 meters along the Y-axis. The lateral force-resisting system utilized is concrete moment frames (C1). Gravity loads comprising dead loads, live loads, and superimposed dead loads corresponding to the building's occupancy were calculated in accordance with SNI 1727:2020 [8], while seismic loads were analyzed using SNI 1726:2019 [7] and ASCE/SEI 41-17 [6]. The structural model is based on secondary field data, which includes as-built drawings, geotechnical investigation results (Standard Penetration Test), and destructive concrete core testing.

The structural component parameters evaluated under the maximum seismic demand consist of two column types on the 1st floor, namely 550 x 700 mm and 500 x 500 mm cross-sections, as well as two beam types, measuring 400 x 600 mm on the 2nd Floor and 400 x 800 mm on the 4th floor, respectively.

➢ *Material Properties*

The capacity evaluation of components for force-controlled actions is based on the lower-bound material properties derived from construction documents, destructive testing results, or reference standards [9]. The lower-bound compressive strength of concrete (f'_{cL}) was determined to be 19.06 MPa through core drill testing. Meanwhile, the lower-bound yield strengths of the longitudinal (f_{ylL}) and transverse (f_{ytL}) reinforcing steel are 420 MPa and 280 MPa, respectively. Conversely, for deformation-controlled actions where ductile behavior and plastic hinge formation are expected, the expected material strengths are utilized. This parameter is obtained by multiplying the lower-bound value by a modification factor in accordance with the provisions of ASCE/SEI 41-17 [6], which are 1.50 for concrete (f'_{cE} = 28.59 MPa) and 1.25 for reinforcing steel (f_{ylE} = 525 MPa and f_{ytE} = 350 MPa).

Fig 2 Existing Building of Tadulako University Hospital

➢ *Ground Motion Acceleration*

A field average Standard Penetration Test (SPT) resistance value of 48.06 was obtained, indicating that the soil characteristics correspond to Site Class D (stiff soil) [7]. The Ministry of Pekerjaan Umum dan Perumahan Rakyat (PUPR) provides an official portal (https://rsa.ciptakarya.pu.go.id/2021/) to facilitate the calculation and determination of coordinate-based design spectral response acceleration parameters. Based on the geographic coordinates (latitude -0.8428, longitude 119.8972), the parameters for the Design Basis Earthquake (DBE) are as follows: Short-period (0.2 s) spectral response acceleration (S_s) with a value of 1.512g; and a long period (1.0 s) spectral response acceleration (S_1) with a value of 0.60g. Considering that the Indonesian seismic hazard map [3] adopts a 2475-year return period to represent the Maximum Considered Earthquake (MCE_R), adjustments to the spectral response parameters are absolutely necessary. This adjustment is performed by converting the S_s and S_1 values to a 225-year return period (BSE-1E) and a 975-year return period (BSE-2E), which empirically refers to the provisions of Eurocode 8 Annex A.2 [5].

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Ground motion acceleration can be computed using a modified equation in accordance with ASCE/SEI 41-17 provisions. The spectral acceleration response (S_a) as a function of the structural period (T) is determined by utilizing the parameters S_{XS}/B_1 and S_{X1}/B_1 as substitutes for S_{DS} and S_{D1}, respectively. Nevertheless, for short-period conditions where $T < T_0$, the value of is determined via linear interpolation between $0.4S_{XS}$ and S_{XS}/B_1. The damping coefficient (B_1) is determined in accordance with ASCE/SEI 41-17 Section 2.4.1.7.1.

The determination of the summary spectral response acceleration parameters at this research site is presented in Table 1. Subsequently, the response spectra for each return period were developed following the ASCE/SEI 41-17 procedure, resulting in the 5% damped elastic response spectrum curves used for the modal response spectrum (MRS) analysis, as shown in Figure 3.

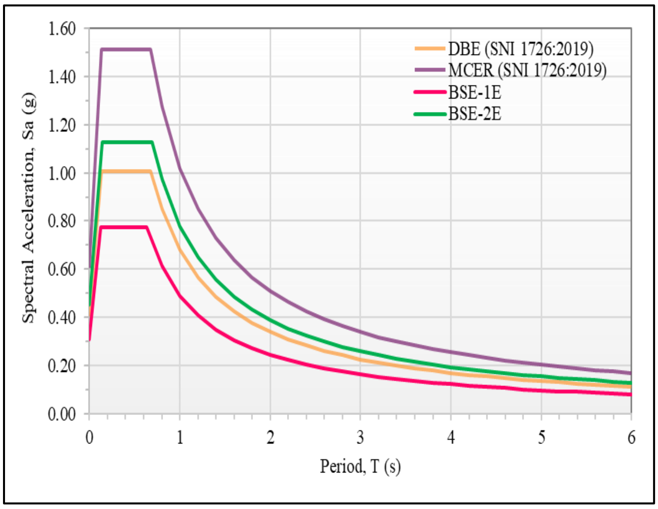

Fig 3 Response Spectrum Used For the MRS Analysis

➢ *Modeling Approaches*

Three-dimensional (3D) mathematical modeling was performed using ETABS software (Figure 4), which generally simulates the behavior of the building in assessing its structural performance. Beams and columns were modeled as frame elements, while slabs were modeled as shell-thin slab elements composed of concrete material with a thickness of 120 mm. For analysis purposes, the slabs were discretized (meshing) into smaller elements with a mesh size of 0.25 m. Finally, the modeling of beam-column joints in ETABS is implicitly permitted using the centerline model, provided that $\sum M_{ColE}/\sum M_{BE} > 1.20$ [6], with the rigid zone factor set to 1.00. The effective stiffness of the degraded structural components is calculated in accordance with the provisions of ASCE/SEI 41-17, taking into account shear, flexural, and axial behaviors.

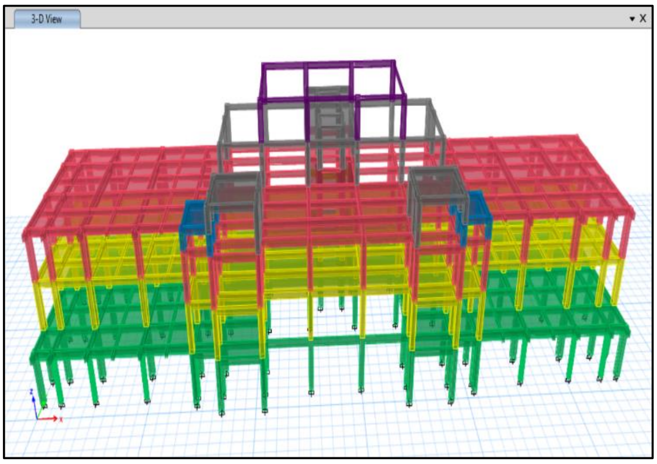

Fig 4 3D Views of the 7-Story Model in ETABS

➢ *Linear Dynamic Procedure*

The structural performance evaluation in this study uses the Linear Dynamic Procedure (LDP) through modal response spectrum (MRS) analysis. The use of the LDP method predicts displacements that closely approximate the maximum displacements expected to occur during the selected seismic hazard level; however, it will yield internal forces that exceed the forces obtained in buildings experiencing inelastic behavior (yielding) [11]. The MRS analysis requires a model capable of calculating the peak modal response of sufficient modes to capture at least 90% mass participation in both principal horizontal directions of the building. In dynamic analysis, determining the extreme responses of structural components including displacements, story shear forces, base shear forces, and other internal actions must combine the contribution of each mode shape using the SRSS or CQC approach. The acceptance criteria in ASCE/SEI 41-17 Section 7.5.2.2 are used as a reference to evaluate the internal force and displacement responses from the LDP analysis, for both deformation-controlled and force-controlled categories. The magnitude of contribution and the distribution of the structure's inelastic performance are represented by the Demand-Capacity Ratios (DCR) based on Eq. (1).

$$DCR = \frac{Q_{UD}}{Q_{CE}} \quad (1)$$

In Eq. (1), the force resulting from gravity and earthquake load effects is denoted as Q_{UD}. Meanwhile, Q_{CE} represents the expected strength of a deformation-controlled element at the deformation level being evaluated. The use of linear procedures is not applicable if the lateral force-resisting system has irregularities and the DCR value of its components exceeds the lower limit between 3.0 and the *m*-factor.

For both primary and secondary elements, the evaluation criteria for deformation-controlled behaviors and force-controlled behaviors are calculated in Eq. (2) and Eq. (3), respectively.

$$\textit{Deformation-controlled:} \quad m\kappa Q_{CE} \geq Q_{UD} \quad (2)$$

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$$Force\text{-}controlled\text{:}\quad \kappa Q_{CL} \geq Q_{UF} \tag{3}$$

where m is a factor applied to modify component capacities to account for the expected ductility associated with the target structural performance level. Meanwhile, κ is accommodated through the knowledge factor. For force-controlled actions, the capacity value used refers to the lower-bound strength at the corresponding deformation condition, symbolized by Q_{CL}, and Q_{UF} represents the force-controlled structural response demand induced by the combination of gravity loads and earthquake lateral forces.

The results of the linear assessment procedure in this study are expressed through the *Normalized* Demand-Capacity Ratio (DCR_N). Through this parameter, verification of the acceptance criteria can be directly evaluated as a unity check, where a DCR_N value greater than 1.0 indicates that the structural component fails to meet the targeted structural performance level criteria [12]. Furthermore, m and κ factors are applied to reduce the elastic demand to the expected demand, considering the Earthquake Hazard Level. This approach provides a more consistent presentation method for comparing the various assessment procedures used in this study.

$$\text{Deformation-controlled:}\quad DCR_N = \frac{Q_{UD}}{m\kappa Q_{CE}} \tag{4}$$

$$\text{Force-controlled:}\quad DCR_N = \frac{Q_{UF}}{\kappa Q_{CL}} \tag{5}$$

The internal forces and component deformations resulting from the LDP analysis are represented as design actions. In their determination, actions for components classified as deformation-controlled are calculated using Eq. (6), whereas force-controlled components are calculated based on Eq. (7).

$$\text{Deformation-controlled:}\quad Q_{UD} = Q_G + Q_E \tag{6}$$

$$\text{Force-controlled:}\quad Q_{UF} = Q_G \pm \frac{\chi Q_E}{C_1 C_2 J} \tag{7}$$

with Q_Q and Q_E representing the gravity load and seismic action load respectively, the internal force adjustment coefficient that accounts for structural behavior and performance levels is denoted by χ. The displacement under linear elastic conditions is related to the peak inelastic displacement through the use of the modification factor C_1, while the maximum displacement response is adjusted using the modification factor C_2 to accommodate the effects of pinched hysteresis characteristics, stiffness degradation, and structural strength reduction. Finally, the force transfer is reduced using the factor J.

All load combinations in Eq. (6) and (7) consistently integrate gravity load parameters, which include dead load (Q_D) and live load (Q_L) as follows:

$$Q_G = 1.1(Q_D + Q_L) \tag{8}$$

$$Q_G = 0.9Q_D \tag{9}$$

III. RESULTS AND DISCUSSION

➢ *Modal Properties*

The dynamic characteristics of the structure are evaluated through eigenvalue analysis (modal analysis), which includes the periods of vibration (T), modal direction factors (MDF), and modal participating mass ratios (MPMR) of each mode shape along the global X, Y, and Z axes. The analysis results indicate that Mode 1 (Figure 5) is a translational mode in the Y-direction, with a MDF of 99.90% and a natural period of 0.579 seconds. Subsequently, Mode 2 is the dominant translational mode in the X-direction, featuring an MDF of 99.30% and a natural period of 0.559 seconds. Lastly, Mode 3 is identified as a torsional mode, with an MDF reaching 98.70% and a natural period of 0.502 seconds. The final characteristic reviewed is the modal participating mass ratios, which indicates that only 6 modes are required to capture at least 90% of the MPMR in each orthogonal direction of the building. Using 5 modes yields an MPMR of 97.08% in the X-direction, while 4 modes provide 93.20% in the Y-direction, and 6 modes achieve 97.06% in the Z-direction, thereby satisfying the established requirements. When the number of modes is increased to 15, 100% of the MPMR is captured in each earthquake loading direction considered. Based on these observations, it was decided to use 15 modes in the analysis to ensure the validity of the results.

➢ *Base Shear*

This study applies modal response spectrum analysis as a representation of the Linear Dynamic Procedure (LDP) in accordance with the provisions of ASCE/SEI 41-17. Unlike other dynamic methods (e.g., SNI 1726:2019), this procedure does not require scaling of the base shear calculations. The application of the PDL in building performance evaluation aims to estimate structural displacement through the multiplication of the C_1C_2 factors and the unreduced base shear (Table 2), adjusted to the design seismic load level and the structural performance targets.

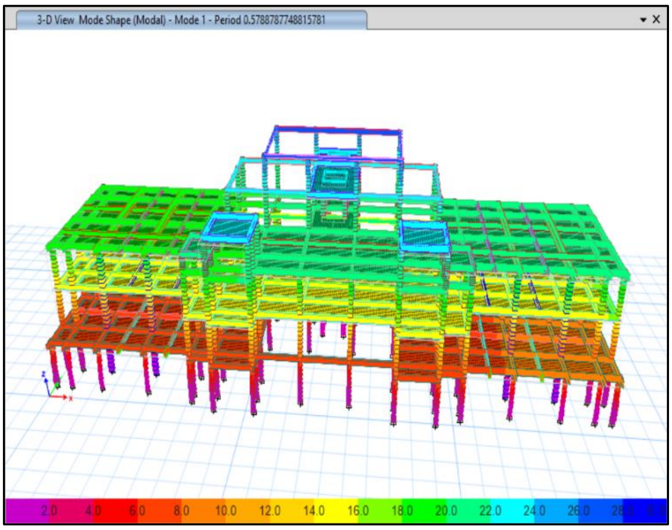

Fig 5 Mode 1 (Translational), Building Period of 0.579 s

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➢ *Column Component Evaluation*

- DCA Evaluation of Column

The evaluation results of the existing column component capacities based on the DCA acceptance criteria are presented in Table 3. The expected strength value is calculated by integrating all load effects from multi-directional seismic actions occurring simultaneously for each load combination. Compliance with the acceptance criteria refers to Eq. (4). In this case, the *m*-factors values are obtained through linear interpolation based on ASCE/SEI 41-17 [6], accounting for the effects of axial force, transverse reinforcement ratio, and the column shear strength effect. Meanwhile, κ is established as 0.90. The maximum DCR for the 55/70 column on the 1st floor, resulting from the interaction between axial force and moment (PMM), is 2.24 under the BSE-1E scenario and 3.39 under BSE-2E. Meanwhile, for the 50/50 column, the maximum DCR values are 1.50 (BSE-1E) and 2.35 (BSE-2E). These columns fail to meet the requirements as their DCRs exceed 1. Furthermore, even when multiplied by the *m*κ coefficient, the 1st-floor column components still do not satisfy the acceptance criteria for the Immediate Occupancy (IO) performance level under the BSE-1E seismic hazard, nor the Life Safety (LS) level under BSE-2E.

- FCA Evaluation of Column

The evaluation results of column components 55/70 and 50/50, based on the acceptance criteria of the FCA linear dynamic procedure under BSE-1E and BSE-2E response spectrum demands, are presented in Tables 4. The shear strength evaluation of the 55/70 column indicates that the maximum DCR_N are 0.56 (BSE-1E) and 0.83 (BSE-2E). Meanwhile, for the column axial strength evaluation, the DCR_N values obtained are 0.38 (BSE-1E) and 0.48 (BSE-2E). At the BSE-1E and BSE-2E seismic hazard levels, the shear strength evaluation results for the 50/50 column show that the maximum DCR_N values are 0.44 and 0.65, respectively. As for the column axial strength evaluation, the largest ratios are 0.29 (BSE-1E) and 0.36 (BSE-2E). These columns satisfy the requirements, with all DCR_N values being less than 1.

➢ *Beam Component Evaluation*

- Deformation-Controlled Actions (DCA)

The results of the comprehensive performance evaluation of the beam components against the Immediate Occupancy (IO) performance level under the BSE-1E seismic hazard and the Life Safety (LS) performance level under the BSE-2E seismic hazard are presented in Table 5. In determining the *m*-factors for flexure-controlled elements, the longitudinal reinforcement ratio, detailing configuration, shear capacity, and element shear demand are evaluated in accordance with the provisions of ASCE/SEI 41-17 [6]. The 30/60 beam elements on the 2nd floor still satisfy the acceptance criteria for negative moment, with DCR_N values of 0.94 (BSE-1E) and 0.92 (BSE-2E). However, a significant increase occurs in the positive moment, where the DCR_N values reach 2.73 (BSE-1E) and 2.69 (BSE-2E). The structural performance of the 40/80 beam elements on the 4th floor shows negative moment DCR_N values of 0.34 (BSE-1E) and 0.25 (BSE-2E), while the positive moment DCR_N values are 0.20 (BSE-1E) and 0.25 (BSE-2E), respectively. Overall, beam components with a $DCR_N > 1.0$ indicate a failure to meet the acceptance criteria. The concentration of failures occurs primarily at the positive moment sections, indicating the structure's inability to achieve the target IO and LS performance levels at those locations.

- Force-Controlled Actions (FCA)

The evaluation of the shear capacity of the moment-resisting frame beams using the force-controlled actions linear dynamic procedure indicates a variation in the values due to the seismic demand of BSE-1E and BSE-2E, as outlined in Table 6. For the 30/60 beam elements, the values are 1.69 (BSE-1E) and 2.60 (BSE-2E), respectively. Meanwhile, for the 40/80 beams, the obtained values are 0.56 (BSE-1E) and 0.65 (BSE-2E). Given that the permissible acceptance criteria limit is $DCR_N < 1$, components exceeding this value are determined to have failed to meet the required performance targets, namely IO (BSE-1E) and LS (BSE-2E).

IV. CONCLUSION

Performance evaluation of the existing reinforced concrete hospital building structure using modal response spectrum analysis (linear dynamic procedure) indicates that the existing columns experience axial-flexural interaction failure under IO (BSE-1E) and LS (BSE-2E) performance objectives, likely due to lower axial forces caused by high lateral demands. Again, the shear capacity of the columns remains within safe limits for all required performance objectives. The beams experience flexural and shear failures at several locations under the IO and LS performance objectives. The linear dynamic analysis steps in this study can be implemented as a reference for the seismic performance-based rehabilitation of reinforced concrete structures.

Table 1 Seismic Response Spectra Acceleration Parameters

Seismic Hazard	S_s	S_1	F_a	F_v	S_{XS}	S_{X1}	T_0	T_s
BSE-1E	0.579g	0.230g	1.337	2.140	0.774g	0.492g	0.127s	0.636s
BSE-2E	1.041g	0.413g	1.083	1.887	1.128g	0.780g	0.138s	0.691s

Table 2 Base Shear Calculation Results Through MRS Analysis

Seismic Hazard	Return Period	Base Shear in X-Direction	Base Shear in Y-Direction
BSE-1E	225-year	32745.09 kN	32309.37 kN
BSE-2E	975-year	47759.99 kN	47124.03 kN

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Table 3 DCA Evaluation of Column Axial Force and Biaxial Moment 1st Floor

Column Type	Seismic Hazard	Demand			DCR	κ	*m-factors*	*m*κ	DCR$_N$
		P_u (kN)	M_{ux} (kN.m)	M_{uy} (kN.m)					
C55/70	BSE-1E	2396.92	-379.41	-1763.67	2.24	0.90	1.441	1.297	**1.73**
	BSE-2E	3340.90	-591.25	-2801.93	3.39	0.90	2.187	1.968	**1.72**
C50/50	BSE-1E	1833.03	-182.25	-726.96	1.50	0.90	1.401	1.261	**1.19**
	BSE-2E	2798.80	-287.86	-1145.75	2.35	0.90	2.024	1.822	**1.29**

Table 4 FCA Evaluation of Column Shear and Axial Force 1st Floor

Column Type	Seismic Hazard	V_{CL} (kN)	V_{UF} (kN)	DCR$_N$ Shear	P_{CL} (kN)	P_{UF} (kN)	DCR$_N$ Axial
C55/70	BSE-1E	728.62	367.15	0.56	6238.20	2125.36	0.38
	BSE-2E	755.26	565.97	0.83	6238.20	2683.17	0.48
C50/50	BSE-1E	423.26	167.33	0.44	6238.20	1624.49	0.29
	BSE-2E	441.84	258.68	0.65	6238.20	2031.53	0.36

Table 5 DCA Evaluation of Beam Moment

Beam Type	Seismic Hazard	Demand		*m-factors*	*m*κ	DCR		DCR$_N$	
		M_u^- (kN.m)	M_u^+ (kN.m)			Negative Moment	Positive Moments	Negative Moment	Positive Moments
B30/60	BSE-1E	847.02	803.81	1.250	1.125	1.05	3.07	0.94	**2.73**
	BSE-2E	1324.27	1281.07	2.000	1.800	1.66	4.85	0.92	**2.69**
B40/80	BSE-1E	689.37	192.82	2.486	2.238	0.76	0.45	0.34	0.20
	BSE-2E	919.27	422.73	4.459	4.013	1.01	1.00	0.25	0.25

Table 6 FCA Evaluation of Beam Shear

Beam Type	Seismic Hazard	Nominal Shear Capacity			V_{UF} (kN)	DCR$_N$
		V_{cL}	V_{sL}	V_{CL}		
B30/60	BSE-1E	117.03	231.16	348.19	530.19	**1.69**
	BSE-2E	117.03	231.16	348.19	815.86	**2.60**
B40/80	BSE-1E	214.17	317.28	531.45	267.18	0.56
	BSE-2E	214.17	317.28	531.45	312.44	0.65

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