A Proposal of Seismic Index for Existing Buildings in Indonesia using Pushover Analysis

Alex Kurniawandy[1, 2,*] & Shoji Nakazawa[2]

[1]Department of Civil Engineering, Universitas Riau,
Jalan HR Subrantas Km 12.5, Kampus Bina Widya, Pekanbaru, 28293, Indonesia
[2]Department of Architecture and Civil Engineering, Toyohashi University of Technology, 1-1 Tempaku-cho, Toyohashi, 441-8580, Japan
*E-mail: alexkurniawandy@eng.unri.ac.id

Highlights:

- A building's seismic performance can be expressed in an index called the seismic index.
- The factors of strength and ductility are the determinants in the formulation of a building's seismic performance.
- Pushover analysis is more accurate since the investigation is carried out under nonlinear conditions.
- A seismic demand index was determined based on the capacity spectrum method by considering seismic hazard in Indonesia.

Abstract. Indonesia has often suffered major earthquake damage over the past 50 years. There are thousands of buildings in earthquake-prone regions that still need seismic evaluation and rehabilitation. One method of evaluating the seismic performance of an existing building is by assessing it using the Japanese seismic index for structures. A basic seismic index can be calculated based on the strength and ductility criteria. The strength and ductility performance of a structure can be obtained by pushing a building until it reaches its maximum deformation capacity. This paper describes a proposal to obtain a basic seismic index using pushover analysis. Its adjustment to determine a seismic demand index by considering seismic hazard in Indonesia was carried out using the capacity spectrum method. Two existing buildings in Indonesia were evaluated. The evaluation result indicated that both buildings were in safe condition. The proposal of the seismic index method can be useful in determining the performance index of existing structures. The ductility index can also be used to estimate the response modification factor of a structure.

Keywords: *ductility; existing building; pushover analysis seismic index; strength.*

1 Introduction

Over the past 50 years Indonesia has often suffered major earthquake damage. There are thousands of buildings in earthquake-prone regions that still need seismic evaluation and rehabilitation [1]. It is difficult to precisely estimate the

Received February 19th, 2019, 1st Revision July 25th, 2019, 2nd Revision August 8th, 2019, 3rd Revision January 20th, 2020, Accepted for publication May 14th, 2020.
 ISSN: 2337-5779,
DOI: 10.5614/j.eng.technol.sci.2020.52.3.2

magnitude and number of earthquakes that will occur during the lifetime of a building.

The Japanese seismic code for the assessment of existing buildings was used as the main starting point in this study. The Japan Building Disaster Prevention Association (JBDPA) published a code for the seismic evaluation of existing buildings and guidelines for the retrofitting of existing reinforced concrete buildings in 1977 [2]. In Japan, the seismic screening procedure mentioned in this guideline is used as a practical tool to identify vulnerable buildings. This tool became essential after a new set of rules and laws for the seismic design of buildings was issued in 1981 and severe earthquake damage was recorded in buildings constructed before 1981. The Japanese standard for the seismic evaluation of buildings has since been widely used to evaluate the capacity of existing buildings up to now. In 2001 an English version was published to encourage engineers in other countries to take up the issue of seismic evaluation. However, to enable the application of this standard in other countries requires some justification and adjustment. The estimation of structural capacity and demand load has a different approach through this code.

In this study, the determination of a seismic index for was obtained based on static non-linear analysis, also known as pushover analysis. This analysis is a widely used method for the seismic performance evaluation of buildings, where the capacity of the structure is analyzed by considering its post-elastic behavior [3]. The purpose of this study was to obtain a seismic index for existing structures based on pushover analysis and to determine a seismic demand index by considering seismic hazard in Indonesia.

2 Methodology

A seismic index is an index that shows the performance of a building to see whether it is in a safe condition if an earthquake occurs. The index is obtained from the product of the strength index and the ductility index, called the basic seismic index. Furthermore, the index is compared to the demand index, which is a limit of safety. When the seismic index is greater than the demand index, the structure is considered to be within the safety limit.

In this study, the first analysis was based on carrying out for the determination of a seismic index in accordance with the Japanese seismic code [2]. A demand index was also obtained based on the index of Japanese seismic code. The second analysis consisted of calculating the seismic index by using a method derived from pushover analysis. The demand index was estimated based on the seismic hazard in Indonesia as indicated by the performance point of the design response spectrum and the capacity curve.

The methodology of this study has some limitations that need to be considered carefully. The pushover analysis was carried out based on an equivalent lateral force distribution assuming that the first mode response is predominant.

The cases selected in this study did not consider the irregularity of the structure. However, higher mode effects should be considered for unsymmetrical structures. A large displacement was not considered in the analysis and the nonlinear behavior was modeled as a bilinear elastoplastic model.

2.1 Seismic index

The seismic performance of an existing building is represented by the seismic index, denoted as I_S in Japan [2,4,5]. The seismic index (I_s) is calculated with Eq. (1) for each story and each principal orthogonal direction of the structure. Eq. (1) is defined as follows:

$$I_s = E_o \cdot S_D \cdot T \tag{1}$$

where E_o is the basic seismic index, T is the time index, and S_D is the irregularity index.

The basic seismic index (E_o) is given as the product of strength index *C* and ductility index *F*, which is calculated differently in the first, second, and third level of the procedure. This study adopted the first-level procedure to calculate the seismic index. Eqs. (2) and (3), which are the formulas for determining the basic seismic index of a structure with shear walls and short columns, are defined as follows:

$$E_o = \frac{n+1}{n+i}(C_w + \alpha_1 C_c)F_w \tag{2}$$

$$E_o = \frac{n+1}{n+i}(C_{sc} + \alpha_2 C_w + \alpha_3 C_c)F_{sc} \tag{3}$$

where n is the number of stories, C_w is the strength index of walls, C_c is the strength index of columns, and C_{sc} is the strength index of short columns. α_1, α_2, and α_3 are the effective strength factors for columns, walls, and short columns respectively.

F_w Is the ductility index of the walls and F_{sc} is the ductility index of short columns. C_c And C_{sc} is expressed by Eqs. (4) and (5) as:

$$C_c = \frac{\tau_c \cdot A_c}{\sum W} \cdot \beta_c \tag{4}$$

$$C_{sc} = \frac{\tau_{sc} \cdot A_{sc}}{\sum W} \cdot \beta_c \tag{5}$$

In the above equation, τ_c is the average shear stress at the ultimate state of the columns, which can be taken as 1.0 N/mm^2 or 0.7 N/mm^2 in the case h_o / D is larger than 6.0.

h_o is the clear height of the columns and D is the depth of the columns. τ_{sc} is the average shear stress at the ultimate state of short columns, which can be taken as 1.5 N/mm^2.

2.2 Irregularity

An unsymmetrical structural configuration of the building in either the horizontal or the vertical direction is one of the main causes of structural failure during past earthquake experiences.

The irregularity index S_D was introduced by JBDPA [2] in order to modify the basic seismic index of structure E_0 by quantifying the effects of the shape complexity and the stiffness unbalance distribution and the like on the seismic performance of a structure.

Methods of calculating the irregularity index consider the effect of horizontal balance, elevation balance, eccentricity, and stiffness. The eccentricity of the floor plan is related to the distance between the centroid of gravity and the center of lateral stiffness.

The irregularity index is calculated as the geometric product of the degree of incidence *qi* as calculated using Eq. (6), which is derived from grade index G_i and the range adjustment factor R_i.

$$S_D = q_a \times q_b \times ... \times q_i \quad \begin{cases} q_i = \left[1-(1-G)\times R_{1i}\right] & i = a, b, ..., j \\ q_i = \left[1.2-(1-G)\times R_{1i}\right] & i = h \\ q_i = \left[1-(1-G)\times R_{2i}\right] & i = l, n \end{cases} \tag{6}$$

The factors R_{1i} or R_{2i} should be used according to the classification of the adjustment factors given in Table 1.

Table 1 Classification of grade index (G) and adjustment factor (R).

		Gi (Grade index)			**R**	
		1.0	**0.9**	**0.8**	**R1i**	**R2i**
Horizontal balance	Regularity	Regular a1	Nearly regular a2	Irregular a3	1.0	0.5
	Aspect ratio	b ≦ 5	5 < b ≦ 8	8 < b	0.5	0.25
	Narrow part	0.8 ≦ c	0.5 ≦ c < 0.8	c < 0.5	0.5	0.25
	Expansion joint	1/100 ≦ d	1/200 ≦ d < 1/100	D < 1/200	0.5	0.25
	Well-style area	e ≦ 0.1	5 < e ≦ 8	0.3 < e	0.5	0.25
	Eccentric well-style area	f ≦0.4 &1 f 2 ≦0.1	f ≦0.4 &1 0.1<f 2 ≦0.3	0.4<f or1 0.3<f 2	0.25	0
Elevation balance	Underground floor	1.0 ≦ h	0.5 ≦ h < 1.0	h < 0.5	0.5	0.5
	Story height uniformity	0.8 ≦ I	0.7 ≦ I < 0.8	I < 0.7	0.5	0.25
	Soft story	No soft-story	Soft story	Ecc. soft story	1.0	1.0
Eccentricity	Eccentricity	1 ≦ 0.1	0.1 < l ≦ 0.15	0.15 < l		1.0
Stiffness	(Stiffness /mass) ratio	n ≦ 1.3	1.3 <n ≦ 1.7	1.7 < n		1.0

2.3 Pushover Analysis

The pushover analysis method has been developed over the past twenty years and has become the preferred analysis for designing and evaluating building structures. This method is most reliable for estimating the capacity of a structure beyond the elastic limit [3]. Pushover analysis is a method where a structure is subjected to gravity loading and a monotonic displacement-controlled lateral load pattern is continuously increased through elastic and inelastic behavior until the ultimate condition or collapse of the structure is reached. The lateral load can be used to represent the range of base shear induced by earthquake loading and its configuration may be proportional to the distribution of mass along with building height or equivalent lateral force, mode shape, or other practical means. The output is a capacity curve that plots a strength-based parameter against deflection. For example, performance may relate the strength level achieved in certain members to the lateral displacement at the top of the structure, or a bending moment may be plotted against plastic rotation. The results will provide insight into the ductile capacity of the structural system and indicate the mechanism, load level, and deflection at which failure occurs.

When analyzing frame objects, material nonlinearity is assigned to discrete hinge locations where plastic rotation occurs [6,7]. Beam and column components are modeled as nonlinear frame elements by defining plastic hinge at both ends of the elements. As shown in Figure 1, the plastic hinge properties have five points (labeled A, B, C, D, and E) that define the force-deformation behavior. The same type of curve is also used for the moment and rotation relationship. The values assigned to each of these points vary depending on the type element, material properties, and section size.

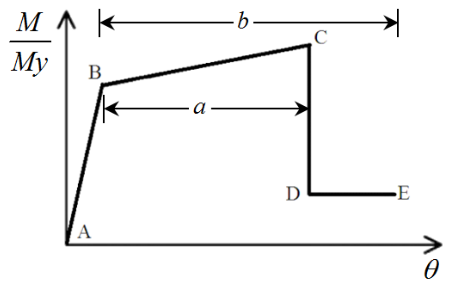


Figure 1 Moment-rotation relationship of typical plastic hinge.

The linear response is related to a line between a point A and an effective yield point B. The slope from point B to point C is typically a small percentage (0% to 10%) of the elastic slope and is included to represent phenomena such as strain hardening. Point C has an ordinate that represents the strength of the element and an abscissa value equal to the deformation at which significant strength degradation begins (line CD). Beyond point D, the element responds with substantially reduced strength until point E. At deformations greater than point E, the seismic element strength is essentially zero [6].

The application of the conventional pushover procedure explained in ATC-40 [3] has some limitations for unsymmetrical structures. This procedure produces a capacity curve representing the first mode response of the structure based on the assumption that the fundamental mode of vibration is the predominant response. Furthermore, it is essential to take into account the higher mode effects to accommodate the irregularity. Several researchers have proposed a pushover method that considers structure irregularity and higher mode effects [8-12].

Chopra studied the effect of irregularity to estimate the floor displacement by introducing a modal pushover analysis [12]. In general, the procedure of the modal pushover analysis (MPA) for linear elastic systems is identical to the response spectrum analysis (RSA) but for inelastic systems; a non-linear static analysis is conducted separately for each modal mode. First, a modal analysis for linearly elastic vibration is conducted to determine the period, mode shape, mass ratio and participation factor for each mode. The independent pushover analysis for each mode is conducted separately by using the lateral force distribution calculated by Eq. (7).

$$s_n^* = m\,\phi_n \tag{7}$$

where m is the mass and ϕ_n is the mode shape in n-mode. The independent pushover analysis for each mode is conducted separately. The result of the capacity curve will be produced for each mode as many as the n-th number of analyses. The sum of the absolute value (SAV) or the root sum square (SRSS) or the complete quadratic combination (CQC) can be used to combine the response of each analysis.

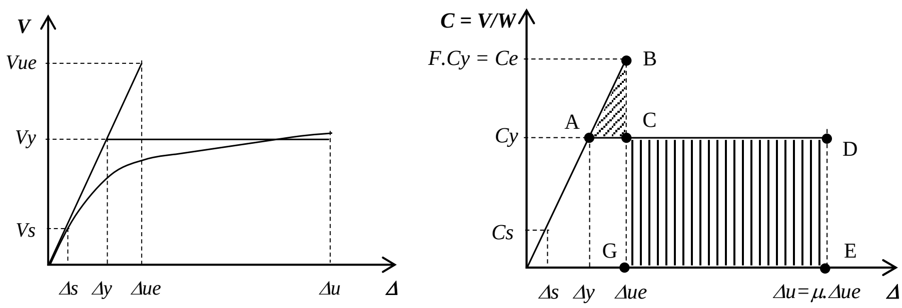


Figure 2 Converting the capacity curve into an elastoplastic curve by the constant energy principle.

A procedure for seismic index calculation by using the capacity curve of a pushover analysis is proposed in this paper. The left side of Figure 2 shows a capacity curve and an idealized bilinear elastoplastic curve. The right side of Figure 2 shows a curve that describes the constant energy principle to get the relationship between the ductility factor (F) and the ductility index (μ). By the constant energy principle, the area of triangle ACB is the same as the area of rectangle CDEG. The ductility index is derived as follows,

$$\begin{aligned}
&\frac{1}{2}(\Delta_{ue} - \Delta_y).(C_e - C_y) = C_y.(\mu.\Delta_y - \Delta_{ue}) \\
&\frac{1}{2}(F.\Delta_y - \Delta_y).(F.C_y - C_y) = C_y.(\mu.\Delta_y - F.\Delta_y) \\
&\frac{1}{2}\Delta_y.C_y(F-1)^2 = C_y.\Delta_y(\mu - F) \\
&F = \sqrt{2\mu - 1}
\end{aligned} \tag{8}$$

The step-by-step procedure of the calculation can be described as follows:

1. Convert the capacity curve into an elastoplastic curve based on the concept of equal energy:
   a. Calculate elastic stiffness (K_e) based on the ratio between shear strength and displacement at elastic conditions $K_e = V_s / \Delta_s$.
   b. Calculate the displacement of Δ_y at the intersection of the linear curve with a gradient of K_e , where $\Delta_y = V_y / K_e$. In the bilinear elastoplastic idealization, the elastic shear strength (V_y) is considered to be equal to the ultimate shear strength (V_u).
2. Determine the ductility factor (μ) at the displacement of maximum shear strength (Δ_u) over (Δ_y), $\mu = \Delta_u / \Delta_y$.
3. Calculate ductility index (F) with the concept of constant energy by Eq. (6), $F = \sqrt{2\mu - 1}$.
4. Estimate the ultimate elastic shear strength *(Vue)* by using ductility index *(F)* with equation $V_{ue} = F.V_y$ and calculate the displacement on the ultimate shear strength elastic (V_{ue}) with the equation $\Delta_{ue} = V_{ue} / K_e$.
5. Determine the total weight of the structure and convert the shear strength to a shear ratio, where V_y , V_{ue} are C_y , C_{ue} respectively.
6. Calculate the basic seismic index (E_o) with the equation $E_o = C_y.F$.
7. Determine the seismic index of a structure by using Eq. (1).

3 Case Study

Calculation of the seismic index based on the JBPDA method and pushover analysis will be described in the following case study. Two buildings were evaluated in this study. Both buildings were located in Indonesia and were designed using the Indonesian seismic design code.

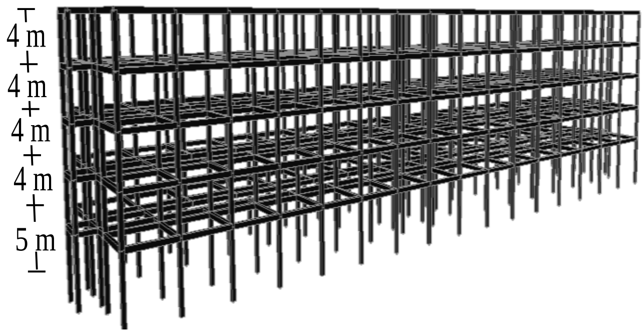
These buildings were selected for evaluation because they were designed with the old seismic design code [13,14]. Currently, the standard has been updated [15]. The updated standard changes the demand capacity of existing buildings. In a particular location the demand capacity may decrease while in other locations it may increase. The earthquake-resistant standard design for buildings can be updated again due to the development of new seismic hazard maps of Indonesia [16-19]. Problems will arise when the demand capacity increases. Therefore, a re-evaluation of existing buildings against the new earthquake load regulations is needed.

The first building is a five-story apartment building, referred to as building A. The second building, referred to as building B, is a four-story office building.

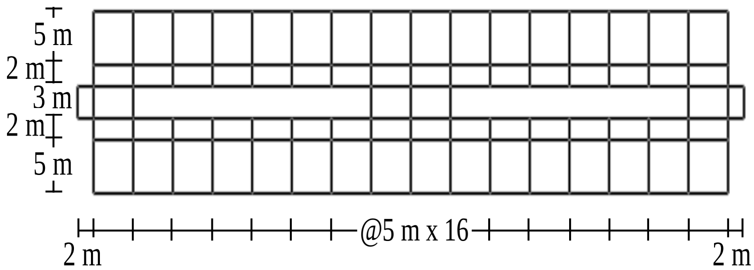
Table 2 Material properties of building A and B.

Concrete		Longitudinal Rebar			Transversal Rebar		
***fc'* (MPa)**	**Ec (MPa)**	***fy* (MPa)**	***fu* (MPa)**	**Es (MPa)**	***fy* (MPa)**	***fu* (MPa)**	**Es (MPa)**
250	2.0E4	390	560	2.0E5	235	382	2.0E5

Building A has an area of 1360 m^2 per floor. The structure is a space frame that has compressive strength concrete (*fc'*) of 25 MPa as shown in Table 2.



(a) Perspective view of building A



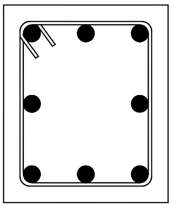
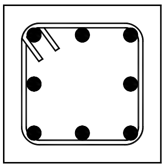
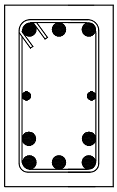
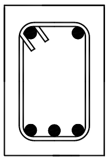
(b) Plan view of building A

Figure 3 Perspective and plan view of building A.

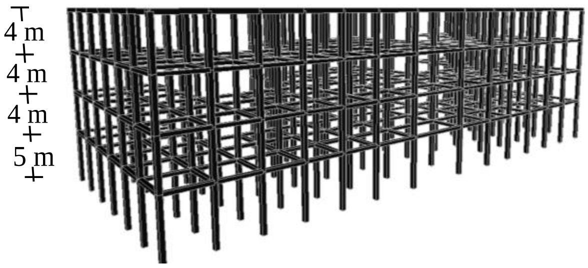
There are two types of columns in this building. Type 1 has a rectangular section of 500 x 600 mm^2 (C1) and type 2 has a square section of 350 x 350 mm^2 (C2). The layout of the building has a typical floor plan for every story.

The floor height is identical on the 1st until the 5th floor as shown in Figure 3. The beams also have two types, namely B1 and B2. Beam B1 has a size of 300 x 500 mm^2 and B2 beam has a size of 200 x 300 mm^2. The detailed beam and column dimensions can be seen in Table 3.

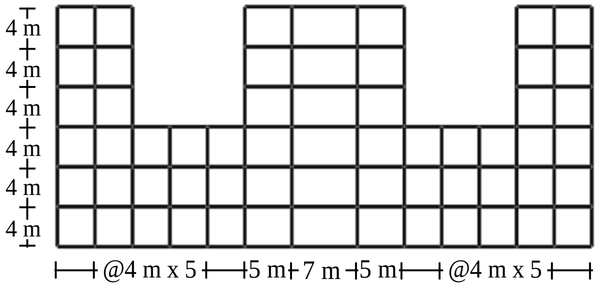
Table 3 Beam and column dimensions of building A.

Column (C1) 500 x 600 mm	**Column (C2) 350 x 350 mm**	**Beam (B1) 300 x 500**	**Beam (B2) 200 x 300**
			
Longitudinal rebar: 8.D19	Longitudinal rebar: 8.D16	Longitudinal rebar: Top 3.D19 Bottom 5.D19	Longitudinal rebar: Top 2.D16 Bottom 3.D16
Transversal rebar: D10 – 100 mm	Transversal rebar: D10 – 100 mm	Transversal rebar: D10 – 100/150 mm	Transversal rebar: D10 – 100/150 mm

The floor plan area of building B is 1008 m^2 for each floor. The structure is made of reinforced concrete using a space frame system with *fc'* of 25 MPa.



(a) Perspective view of building B



(b) Plan view of building B

Figure 4 Perspective and plan view of building B.

The columns of building B have a typical rectangular section of 400 x 500 mm^2. The detailed beam and column dimensions can be seen in Table 4. Building B has an irregular floor plan as shown in Figure 4. The planar irregularities were investigated by the effect of the non-coincidence between the center of mass (CM) and the stiffness center, also known as the center of rigidity (CR). According to ASCE 41-13 [6], the torsional effect is not very significant since the distance between the center of mass and the stiffness center is less than 20% of the building width in either plan dimension. As shown in Table 5, the distance of center mass and rigidity was 0.29, 0.55, 0.68 and 0.77 m respectively for story 1 to story 4.

The average of eccentricity was around 2%. An eigenvalue analysis was calculated for structure B to confirm the response to the higher mode effects. In Table 6, the result shows that the first mode is the predominant response since the equivalent mass ratio (Rho) is 90% (ASCE 7 [20]).

Table 4 Beam and column dimensions of building B.

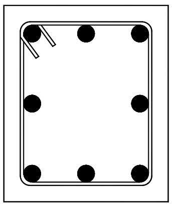
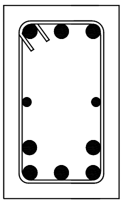
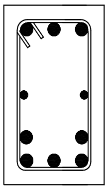
Column (C1) 400 x 500 mm	**Beam (B1) 300 x 500**	**Beam (B2) 250 x 400**
		
Longitudinal rebar: 8.D19	Longitudinal rebar: Top 3.D19 Bottom 5.D19	Longitudinal rebar: Top 3.D16 Bottom 5.D16
Transversal rebar: D10 – 100 mm	Transversal rebar: D10 – 100/150 mm	Transversal rebar: D10 – 100/150 mm

Table 5 Floor plan eccentricity of building B.

Story	**XCM (m)**	**YCM (m)**	**XCR (m)**	**YCR (m)**	**Distance CM-CR (m)**	**Eccentricity (%)**
1	28.5	10.88	28.5	10.60	0.29	1.2
2	28.5	10.88	28.5	10.34	0.55	2.3
3	28.5	10.88	28.5	10.21	0.68	2.8
4	28.5	10.88	28.5	10.11	0.77	3.2

Table 6 Floor plan eccentricity of building B.

Mode	**X-direction**		**Y-direction**	
	Period (s)	**Rho**	**Period (s)**	**Rho**
1	0.76	0.89	0.62	0.89
2	0.26	0.08	0.21	0.08
3	0.17	0.01	0.14	0.02

4 Results of the Seismic Evaluation

4.1 Seismic Index

The calculation of the seismic index for buildings A and B was determined based on the Japanese seismic code. The seismic index was calculated for each story and multiplied by a story modification factor. As an example, the modification story factor for the 5th floor (C_5) was calculated as follows:

$$C_5 = \frac{n+1}{n+i} = \frac{5+1}{5+5} = 0.6 \tag{9}$$

where n is the number of stories in the building and i is the observed story. A similar method was applied to the other floors. Secondly, the strength index of the vertical elements was determined. In the case of building A, the only vertical elements for resisting lateral load were the columns.

Since there were two types of columns, the total strength index can be calculated as follows:

$$C_c = \frac{1.0 \times 2.0E07}{1.6E07} \times 1.12 + \frac{0.7 \times 4.2E07}{1.6E07} \times 1.12 = 1.61 \tag{10}$$

The basic seismic index of structure E_o for the 5th floor determined with Eq. (2), was calculated as follows:

$$E_{05} = 0.6(0 + 1.0 \times 1.61) \times 1.0 = 0.97 \tag{11}$$

The time index T was taken as 1.0 since no reduction factor was considered. The irregularity index S_D was taken as 1.0 since no reduction factor of the irregularity of the floor and sectional plan was considered. The seismic index for the 5th floor of building A following Eq. (1) is described with Eq. (12):

$$I_{s5} = 0.97 \times 1.0 \times 1.0 = 0.97 \tag{12}$$

The same procedure as shown in Eqs. (7)-(10) was applied to the other floors of building A. Table 7 show the seismic index of building A for each story. As shown in Table 7, the maximum seismic index for both directions occurred for story 5.

The result of the seismic index in the transversal direction was higher than in the longitudinal direction. This was because the ratio h_o/D in the transversal directions was smaller than 6, while in the longitudinal direction it was larger than 6. Therefore, the average shear stress in the columns in the transversal direction was 1 N/mm^2, while it was 0.7 N/mm^2 in the longitudinal direction.

Building B (Table 8) had irregularity index (S_D) equal to 1.0, the same as building A.

Table 7 Seismic index of building A.

Direction	Story	E_o	S_D	T	I_s
Transverse	5	0.97	1.00	1.00	0.97
	4	0.54	1.00	1.00	0.54
	3	0.40	1.00	1.00	0.40
	2	0.35	1.00	1.00	0.35
	1	0.32	1.00	1.00	0.32
Longitudinal	5	0.89	1.00	1.00	0.89
	4	0.49	1.00	1.00	0.49
	3	0.37	1.00	1.00	0.37
	2	0.32	1.00	1.00	0.32
	1	0.30	1.00	1.00	0.30

Table 8 Seismic index of building B.

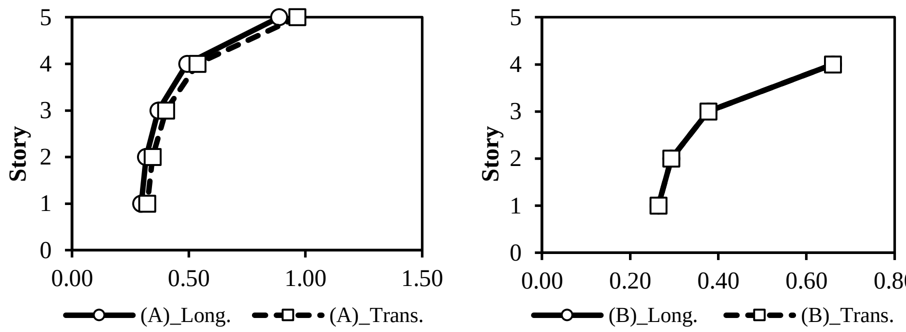
Direction	Story	E_o	S_D	T	I_s
Transverse	4	0.66	0.1	1.00	0.53
	3	0.38	0.1	1.00	0.30
	2	0.29	0.1	1.00	0.23
	1	0.26	0.1	1.00	0.21
Longitu-dinal	4	0.66	0.1	1.00	0.53
	3	0.38	0.1	1.00	0.30
	2	0.29	0.1	1.00	0.23
	1	0.26	0.1	1.00	0.21

The effect of the unsymmetrical floor plan was calculated with Eq. (6) as shown in Eq. (13). The eccentricity of 2% has a grade index (G) of 1.0 and produced no significant irregularity index.

$$S_{D,l} = \left[1-(1-G)\times R_{2i}\right] = \left[1-(1-1.0)\times 1.0\right] = 1.0 \tag{13}$$

As shown in Table 8, Building B has the same basic seismic index in both directions. The average shear stress of the column was equivalent in the transversal direction and in the longitudinal direction.

Figure 5(a) and (b) show a graph of the seismic index for each story. The strength index was the same for both directions in building B. Therefore the graph for the longitudinal direction coincides with transversal direction, as shown in Figure 5(b).



(a) Seismic index of building A (b) Seismic index of building B

Figure 5 Seismic index of buildings A and B.

4.2 Seismic Index Base on Pushover Analysis

The pushover analysis of a structure produces a capacity curve. The capacity curve plots the displacement of the top floor on the x-axis and the base shear on the y-axis. The modeling parameters for nonlinear hinge properties use a generalized force-deformation relation of concrete elements, where the flexural type of deformation controlled behavior is considered for both beams and columns. The plastic rotation angle at maximum plastic capacity (see Figure 1) was taken as 0.025 radians for the beams and 0.035 radians for the columns [6]. The target of displacement magnitude of both buildings was equal to the allowable story drift multiplied by the total height of the structure.

Table 9 Lateral forces in buildings A and B.

Story	Building A		Building B	
	Height (m)	Lateral Force (kN)	Height (m)	Lateral Force (kN)
5	21	4,308.75		
4	17	3,369.58	17	2975.95
3	13	2,466.16	13	2134.47
2	9	1,607.71	9	1353.45
1	5	811.33	5	653.42

The allowable story drift was taken as 0.025 by considering risk category I based on ASCE 7-10. An equivalent seismic load in accordance with SNI 1726-2002 and ASCE 7-10 [14, 20] was used as the lateral force in both buildings. The equivalent static acceleration was modified by a seismic coefficient depending on the seismicity of the location, the soil properties and the natural period multiplied by the total mass of the structure to get the base shear force [21,22]. The detailed lateral load can be seen in Table 9. In this analysis, the lateral load in both directions was assumed to be the same.

The conventional pushover analysis has some limitations in estimating the response of irregular structures like building B. The modal pushover analysis (MPA) is an advanced pushover method for irregular structures, as described in the previous section. In this study, because of the dominant response of building B in the first mode, the planar irregularity of this building was not considered, by introducing the assumption that the horizontal displacement of each floor is the same. In this case, the effect of irregularity is not very large.

In order to determine the seismic index based on the capacity curve, the capacity curve was converted to a bilinear curve (elastoplastic) using the procedure described in Section 3.2. A linear line with a gradient equal to the elastic stiffness (K_e) can be determined by the value of step 1 of the pushover data, as shown in Table 10.

Table 10 Capacity Curve of Buildings A and B

(A) Longitudinal		**(A) Transversal**		**(B) Longitudinal**		**(B) Transversal**	
Displ. (m)	**Base Force (kN)**	**Displ. (m)**	**Base Force (kN)**	**Displ. (m)**	**Base Force (kN)**	**Displ. (m)**	**Base Force (kN)**
0.0	0.0	0.0	0.0	0.00	0.0	0.00	0.0
0.06	7,248.1	0.05	7,578.0	0.06	2,771.8	0.06	3,361.5
0.08	9,717.7	0.06	9,341.5	0.13	5,961.0	0.12	6,954.1
0.11	11,003.8	0.10	11,247.8	0.22	7,748.6	0.22	9,487.9
0.16	11,775.0	0.13	11,944.6	0.28	8,374.8	0.31	10,578.1
0.20	12,019.4	0.33	14,231.0	0.31	8,529.3	0.35	10,892.9
0.42	12,707.4	0.38	14,676.7	0.35	8,793.1	0.41	11,240.8
0.58	13,116.6	0.57	15,325.7	0.43	9,126.9	0.42	11,283.7

The maximum elastic strength (V_y) is divided by the total weight of a structure to obtain the strength index (C_y). Calculation of the seismic index for building A in the longitudinal direction was done as follows:

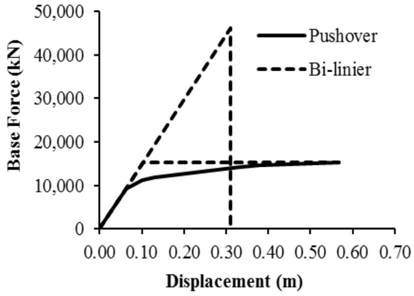
$$K_e = V_s / \Delta_s = 7,248.1 / 0.06 = 131,418.6$$

$$\mu = \Delta_u / \Delta_y = 0.56 / 0.09 = 5.83$$

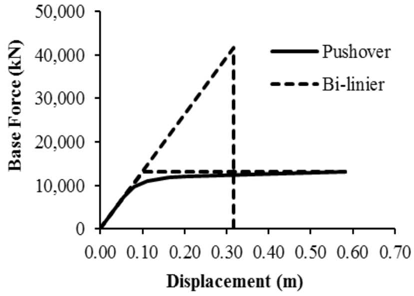
$$F = \sqrt{2\mu - 1} = \sqrt{2 \times 5.83 - 1} = 3.27$$

$$C_y = V_y / W = 12,730/80,240 = 0.16$$

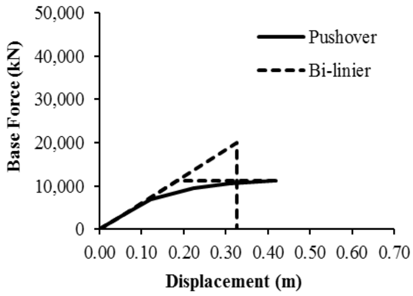
$$E_o = C_y \times F = 0.16 \times 3.27 = 0.52$$



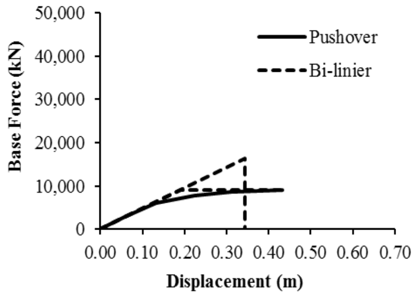
(a) I_s of building A in the longitudinal direction



(b) I_s of building A in the transversal direction



(c) I_s of building B in the longitudinal direction



(d) I_s of building B in the transversal direction

Figure 6 Seismic index (I_s) of buildings A and B.

The bilinear curves for buildings A and B in both directions from Table 10 were plotted in graphs, as shown in Figure 6. Table 11 summarizes the calculations for buildings A and B for both directions.

Table 11 Seismic index using pushover analysis.

Case	μ	F	C_y	$E_o = C_y \cdot F$	I_s
A_Long	5.83	3.27	0.16	0.52	0.52
A_Trans	5.50	3.16	0.18	0.58	0.58
B_Long	2.26	1.88	0.19	0.35	0.35
B_Trans	2.27	1.88	0.22	0.42	0.42

A seismic index comparison using the first-level procedure of the Japanese seismic code and a seismic index based on pushover analysis can be seen in Table 11. The seismic index value for the entire structure is represented by the index for the 1st floor from the level-1 calculation.

As can be seen in Table 12, the result of the seismic index from the pushover analysis was larger than the seismic index based on the Japanese seismic code. Due to the fact that the calculation of the structural capacity was carried out until post-elastic conditions and was stopped when the ultimate capacity was reached, whereas the calculation based on the Japanese seismic code for level 1 was only based on Eqs. (2) and (3). The shear stress value used was the average value of 0.7 to 1.0 N/mm^2.

Table 12 Seismic index comparison.

Case	I_s (Level 1)	I_s (Pushover)
A_Long	0.30	0.52
A_Trans	0.32	0.58
B_Long	0.26	0.35
B_Trans	0.26	0.42

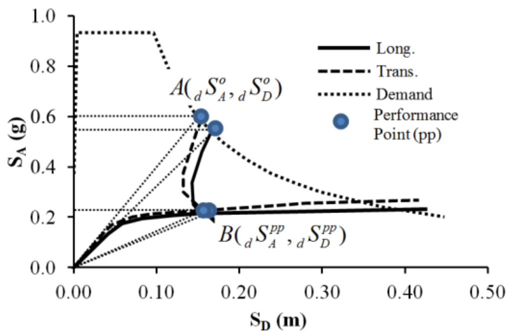
4.3 Evaluation of Structure Performance

The seismic demand index (I_{so}) was taken as 0.8 based on the Japanese seismic code. This demand index was estimated based on hazard conditions due to earthquake loads in Japan. The response spectrum at specific locations other than Japan can be determined based on the code of that location. The response spectrum then becomes a demand curve, which can be compared to the capacity curve in the acceleration-displacement response spectrum (ADRS) format. The intersection of these two curves is the approximation of the performance point of the structure.

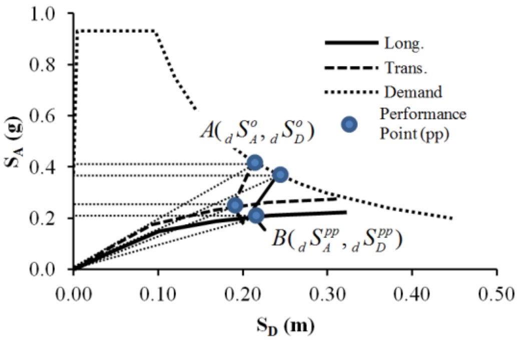
In this study, the performance points of buildings A and B were investigated based on Indonesian seismic loads. The demand response spectrum of spectral acceleration is 1.4 at 0.2 seconds and 0.6 at 1 second in Padang city. The site coefficient was taken as 1.0 and 1.5. Figure 7 shows the performance points of buildings A and B.

A similar procedure for determining the seismic index can also be applied in determining the seismic demand index. The seismic demand index is given by the product of the strength index (C_y) and the ductility index (F). The ductility index (F) is equal to the ratio between $A({}_dS_A^o, {}_dS_D^o)$ and $B({}_dS_A^{pp}, {}_dS_D^{pp})$ as shown in Figure 7.

In Figure 7, point A is the acceleration response on the demand curve and B is the acceleration response at the performance point. The seismic demand index I_{so} was taken equal to E_{so} since no reduction factor was considered. Using this method, the seismic demand index was smaller than the seismic demand based on the Japanese seismic code.



(a) Building A



(b) Building B

Figure 7 Performance points of buildings A and B.

Table 13 shows the evaluation of the structure's seismic performance for both buildings. The seismic index value (I_s) as shown in Table 12 is larger or equal to the seismic demand index (I_{so}) in Table 13, so that the buildings are in safe condition.

Table 13 Evaluation of seismic performance.

Case	A $({}_dS_A^o, {}_dS_D^o)$	B $({}_dS_A^{pp}, {}_dS_D^{pp})$	F (A/B)	C_y	I_{so} $(E_{so} = F \cdot C_y)$
A_Long	0.540	0.210	2.57	0.16	0.41
A_Trans	0.580	0.225	2.58	0.18	0.47
B_Long	0.370	0.195	1.90	0.18	0.35
B_Trans	0.420	0.255	1.65	0.22	0.36

5 Conclusion

Two existing buildings were evaluated in this paper. The first building had five stories while the second building had four stories. Both buildings consisted of a moment-resisting frame system made of reinforced concrete. An index represented the seismic performance of the existing buildings according to the Japanese seismic code. The index is a function of strength and ductility. The structure had different seismic indexes in the lateral and transversal directions because of the difference in stiffness in both directions. According to the evaluation result, building A had a seismic index in the transversal direction larger than in the longitudinal direction. Meanwhile, building B had the same seismic index in both directions.

The application of a seismic index based on the Japanese seismic code needs adjustment for other countries. In this paper, a set procedure was proposed to determine the seismic index for Indonesia based on the result of a pushover analysis. The result of the seismic index was higher than the one obtained using the Japanese seismic code, because the calculation of the structural capacity was carried out until post-elastic conditions, whereas the calculation based on the Japanese seismic code for level 1 is based on the average shear stress on the lateral force resisting elements. Furthermore, the Indonesian seismic demand index was smaller than the Japanese seismic demand index. The final assessment of the seismic performance by the proposed method indicated that both buildings were in safe condition.

The method proposed in this study can be useful in determining the performance index of existing structures with a seismic index. In addition, the ductility index can also be used to estimate the response modification factor for the structure.

Acknowledgment

The first author would like to acknowledge the support from the Indonesian Endowment Fund for Education (LPDP), Ministry of Finance and the Directorate General of Higher Education (DIKTI), Ministry of Research, Technology and Higher Education, Republic of Indonesia.

References

[1] Kurniawandy, A., Nakazawa, S., Hendry, A., Ridwan & Firdaus, R., *Structural Building Screening and Evaluation*, in International Conference of Global Network for Innovative Technology and Awam International Conference in Civil Engineering (IGNITE-AICCE’17), AIP Conference Proceedings 1892, Penang, Malaysia, 2017. DOI: 10.1063/1.5005662.

[2] The Japan Building Disaster Prevention Association (JBDPA), *Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings (English Version)*. Tokyo, Japan, 2001.

[3] Applied Technology Council ATC 40, *Seismic Evaluation and Retrofit of Concrete Buildings*, **1**,California, USA, 1996.

[4] Nakano, Y., Maeda, M., Kuramoto, H. & Murakami, M., *Guideline For Post-Earthquake Damage Evaluation And Rehabilitation of Buildings in Japan*, in the 13th World Conference on Earthquake Engineering, Canada, Paper No.124, pp. 1-15, 2004.

[5] Mehani, Y., Taleb, R. & Bechtoula, H., *Seismic Vulnerability Evaluation of Existing Reinforced Concrete Building Retrofitted with RC Wing Walls*, 15th World Conf. Earthq. Eng. Lisbon Port., 2000, 2012.

[6] American Society of Civil Engineers, *ASCE 41-13, Seismic Evaluation and Retrofit of Existing Buildings*. USA, 2014.

[7] Serror, M.H. & Abdelmoneam, M.N., *Seismic Performance Evaluation of Egyptian Code-Designed Steel Moment Resisting Frames*, HBRC J., **14**(1), pp. 37-49, 2016.

[8] Habibi, A. & Asadi, K., *Development of Drift-based Damage Index for Reinforced Concrete Moment Resisting Frame with Setback*, Int. J. Civ. Eng., **15**(4), pp. 487-498, 2017.

[9] Shojaei, F. & Behnam, B., *Seismic Vulnerability Assessment of Low-Rise Irregular Reinforced Concrete Structures Using Cumulative Damage Index*, Adv. Concr. Constr., **5**(4), pp. 407-422, 2017.

[10] Chintanapakdee, C. & Chopra, A.K., *Seismic Response of Vertically Irregular Frames: Response History and Modal Pushover Analyses*, J. Struct. Eng., **130**(8), pp. 1177-1185, 2004.

[11] Bhosale, A.S., Davis, R. & Sarkar, P., *Vertical Irregularity of Buildings: Regularity Index versus Seismic Risk*, ASCE-ASME J. Risk Uncertain. Eng. Syst. Part A Civ. Eng., **3**(3), p. 04017001, 2017.

[12] Chopra, A.K. & Goel, R.K., *A Modal Pushover Analysis Procedure to Estimate Seismic Demands for Unsymmetric-Plan Buildings*, Earthquake Engineering and Structural Dynamics, pp. 903-927, 2004. DOI: 10.1002/eqe.380.

[13] Indonesian code *SNI 03-1727-1989 Design Load Procedure for Homes and Buildings*, Standardization Agency of Indonesia (BSN), Jakarta, 1989.

[14] Indonesian code *SNI 03-1726-2002, Earthquake Resistance Design Code for Building*, Standardization Agency of Indonesia (BSN), Jakarta, 2002.

[15] Indonesian code *SNI 1726:2012 Earthquake Resistance Design Code for Building and non-Building of Indonesia*, Standardization Agency of Indonesia (BSN), Jakarta, 2012.

[16] Irsyam, M., Asrurifak, M., Budiono, B. & Triyoso, W., *Development of New Seismic Hazard Maps of Indonesia 2017*, in Proceedings of the 19th International Conference on Soil Mechanics and Geotechnical Engineering, pp. 1525-1528, 2017.

[17] Irsyam, M., Asrurifak, M., Mikhail, R., Wahdiny, I.I., Rustiani, S. & Munirwansyah, M., *Development of Nationwide Vs30 Map and Calibrated Conversion Table for Indonesia using Automated Topographical Classification*, J. Eng. Technol. Sci., **49**(4), p. 457, 2017.

[18] Sengara, I.W., Sidi, I.D., Mulia, A., Asrurifak, M. & Hutabarat, D., *Development of Risk Coefficient for Input to New Indonesian Seismic Building Codes*, J. Eng. Technol. Sci., **48**(1), pp. 49-65, 2016.

[19] Rahmat, P.R., Kiyono, J., Ono, Y. & Parajuli, H.R., *Seismic Hazard Analysis for Indonesia*, J. Nat. Disaster Sci., **33**(2), pp. 59-70, 2012.

[20] American Society of Civil Engineers, *ASCE 7-10, Minimum Design Loads for Building and other Structures*. USA, 2010.

[21] Helou, S.H. & Muhammad, I., *Equivalent Lateral Load Method vs. Response Spectrum Analysis Which Way is Forward*, Asian J. Eng. Technol., **02**(05), pp. 2321-2462, 2014.

[22] Mohammadi, R K. & El Naggar, H., *Modifications on Equivalent Lateral Force Method*, in 13th World Conference on Earthquake Engineering, Canada, 2004.