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RESEARCH

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Effect of alternative load path design method on preventing progressive collapse and reducing the rehabilitation cost using FEMA-P58

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Abstract

The alternative load path (ALP) method is an important modern design approach to prevent progressive collapse of buildings by redesigning adjacent structural elements to provide an alternative load path in case of partial collapse. This may lead to tangible changes in the mass and stiffness of the building, which means a change in the seismic behavior of the building. This shows the fact that choosing and designing an alternative load path is not an easy task. Studying the improvement in the seismic behavior of the building, whether direct or indirect, as a result of using the ALP, is the main objective of this paper. To achieve this goal, this study examined approximately 96 models, including the study of the possibility of progressive collapse occurring, whether there is or isn't an alternative path for the loads using non-linear static and dynamic analysis, as well as evaluating the study models using various seismic evaluation methods, where both the nonlinear static pushover analysis method and the FEMA-P58 method were used and compared. The study models are divided into two sections, with and without (ALP). The study models are also categorized by building height using heights of 6, 9, 12 and 15 floors. The models were also divided based on the locations of weak points in the original design that may be susceptible to damage and lead to progressive collapse, whether they are internal, side, or corner weak columns. The findings from the examined models indicate that the efficiency of (ALP) systems is more prominent in low and mid-height buildings compared to tall buildings. Additionally, the results demonstrate that the FEMA-P58 method yields similar predictions regarding building behavior as traditional seismic assessment methods like pushover analysis, but in a simplified manner that benefits non-engineers by providing clear insights into the financial and temporal aspects of different structural system solutions.

Keywords: Progressive Collapse, Alternative Load Path, Nonlinear static analysis, Nonlinear dynamic analysis, Assessment of Seismic Behavior, Pushover Analysis, FEMA P-58

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Introduction

The alternate load path method is an important modern design method that helps prevent progressive collapse of buildings. It involves a deeper study of the structural system of the building, identifying weak points, and redesigning adjacent structural elements to represent an alternate load path. This helps prevent the spread of partial collapse to the rest of the building. However, providing an alternative load path leads to noticeable changes in the mass and stiffness of the building, which affects its seismic behavior. So, designing an alternate load path is not easy and requires maintaining a flexible seismic behavior to prevent excessive lateral loads on the structural elements while taking advantage of the alternate load path to improve the seismic behavior of the building. The objectives of this research include investigating the effectiveness of alternative load path design in preventing progressive collapse, describing the benefits of the FEMA-P58 seismic performance assessment method, comparing traditional methods like pushover analysis to FEMA-P58, evaluating the effectiveness of alternative load path design in reducing rehabilitation costs and improving building behavior under different loads, examining the impact of building height and weak structural elements on alternative load path design, and comparing initial and rehabilitation costs for different types of buildings in various seismic scenarios.

Progressive collapse is generally characterized by the failure of the entire structure due to an initial localized event, the collapse can occur in various ways. Over time, there have been significant enhancements in codes and standards to ensure structural safety against progressive collapse. Recently, guidelines have been proposed by organizations such as GSA (2016) [1] and DOD (2016) [2] to improve the strength, ductility, and continuity of existing and new buildings to resist progressive collapse.

(ASCE/SEI 7–22) [3] define two general approaches for preventing and minimizing the possibility of progressive collapse, Direct Design Approaches which involve alternate path method and specific local resistance method. The second approach is Indirect Design to preventing progressive collapse by providing structures with a minimum level of strength, ductility, and continuity.

GSA (2016) [1] Emphasize the need to consider redundancy, structural integrity, ductility, and load reversal capacity during the design process to make a structure more robust and improve its resistance against progressive collapse.

(UFC) 4–023-03 [2] Outlines the structural design process for preventing progressive collapse. The design process involves applying either the direct approach with the alternate path method or the indirect approach with the tie forces method. Tie forces, generated by catenary actions, improve the structure's continuity, ductility, and redundancy by holding the structure together even after the failure of individual structural elements or components.

To assess a structure's ability to withstand abnormal loads, it's essential to conduct a progressive collapse analysis. Different methods can be employed for this purpose, including linear static, nonlinear static, linear dynamic, and nonlinear dynamic analyses, each with its pros and cons. a more comprehensive discussion of the four progressive collapse analysis techniques is available in Marjanishvili's paper [4].

Starossek (2007) [5] classified progressive collapse into four classes and six types based on the trigger and collapse pattern, and provided an explanation of the potential mechanisms for progressive collapse events in structures. This types are Pancake-type, Zipper-type, Domino-type, Section-type, Instability-type and Mixed-type.

S.Halil and G.Kevin, (2009) [6] Investigated the progressive collapse analysis of two existing full-scale buildings, where strain measurements obtained in the field were compared with computer modeling results using the SAP2000 software. The buildings had between 2 and 3 stories, with floor heights ranging from 3.2 m up to 6.2 m, and 9 bays with 8.5 m span for all models. To obtain a more precise numerical simulation, material nonlinearity, three-dimensional effects, and dynamic behavior were considered. The study found that the strain–time curve obtained from strain gages in the field had lower strain values compared to linear and non-linear static analysis. Nonlinear dynamic analysis provided strain values that were more similar to the measured strains than those obtained from linear static analysis.

Lu et al. (2013) [7] Examined potential loss mechanisms in high-rise buildings during severe earthquakes. The findings indicated that structural members may buckle or collapse, causing upper stories to fall onto lower ones and resulting in the collapse of the entire structure, known as pancake-type collapse. Another study also evaluated damage mechanisms associated with severe earthquakes and found that the most common type of damage in high-rise structures is pancake-type progressive collapse. The mechanisms of seismic progressive collapse events are complex and may involve a combination of multiple collapse types during an earthquake. Factors such as the crushing of concrete shear walls and the transfer of extra loads to other structural elements can lead to the extension of damage to vertical load-bearing elements and potentially lead to redistributed progressive collapse of the zipper-type.

S.Brian and S.Halil (2013) [8] Investigated the progressive collapse of steel-framed buildings through both experimental testing of the Ohio Union building and numerical simulations using SAP 2000. 2-D and 3-D analytical models were developed using linear static and non-linear dynamic analysis, with a focus on the sudden removal of column elements. Strain measurements from the experimental investigation were compared to numerical results, with the 3-D model showing lower demand DCR values and vertical displacement than the 2-D model, likely due to the inclusion of a transverse beam in the former. Non-linear dynamic analysis provided strain values closer to the measured strain than linear static analysis, which may produce overly conservative results due to the amplification factor required for dead load analysis.

B.Mircea et al. (2014) [9] Examined the potential for progressive collapse in two low-rise RC framed structures using the Finite Element Method (FEM) and the Applied Element Method (AEM). The study analyzed two structures of 3 and 6 stories in height, both subjected to an interior column damage scenario, using nonlinear dynamic analysis. The study compared the results obtained from the two approaches, focusing on plastic hinge vs. distributed plasticity in the progressive collapse analysis and discussing differences in terms of vertical displacements and rotations. The ABAQUS software package was used for the finite element analysis, while extreme loading was used for the AEM method. the study found that both structures were capable of resisting progressive collapse when the ground floor interior column was damaged. The 3-story model showed a difference of approximately 10% between the two numerical approaches, with vertical displacement levels insufficient to initiate progressive collapse.

G.Taras, (2014) [10] Conducted a parametric analysis of progressive collapse in high-rise buildings, using 2-D structural models with varying building parameters. The study utilized SAP2000 as the primary structural analysis software to model the buildings and focused on the impact of the total number of floors on the building's resistance to progressive collapse. Non-linear dynamic analysis was used to solve for the development of plastic hinges in the structural models. The analysis found that the formation of plastic hinges decreased significantly with an increase in the number of stories in a building. The results of the 2-D analysis demonstrated that tall buildings were relatively resilient to local failure within the structure.

Tavakoli, H.R., and Hasani, A.H. (2017) [11] Examined the behavior of moment resisting frames under progressive collapse due to column removal and analyzed the effect of earthquake characteristics on progressive collapse for critical column removal locations. The study found that the potential for progressive collapse was higher when corner columns were removed compared to interior columns and was higher at higher stories than at lower ones due to the lower number of elements for load redistribution. The 15-story exhibited lower potential for progressive collapse than the 5-story.

The mentioned studies reviewed the factors leading to progressive collapse of buildings and the behavior of the building in resisting progressive collapse. They presented means to prevent progressive collapse, such as the alternate load path. To study the effect of the alternate load path on the seismic behavior of the building, one of the different methods for seismic evaluation of buildings must be used. These methods can be divided into traditional methods such as pushover and the new generation of seismic evaluation methods such as FEMA-P58.

Pushover analysis evaluates the structural response of a building to lateral loads and determines its maximum strength, displacement, weak zones, plastic hinge locations / rotation degree, and creates a base shear vs. top displacement curve. On the other hand, FEMA-P58 uses statistical methods to predict the behavior of both structural and non-structural elements of a building based on probability.

The Federal Emergency Management Agency (FEMA) contracted with the Applied Technology Council (ATC) to develop a program called FEMA P-58 or Seismic Performance Assessment of Buildings, Methodology and Implementation, which provides a general methodology for evaluating the probable seismic performance of individual buildings based on their unique characteristics. The program characterizes seismic performance in probabilistic terms, based on the potential for damage or losses in the form of repair costs, repair time, casualties, unsafe placarding, and environmental impacts. It measures the seismic design objectives including protection of life, limitation of repair costs and repair time, and functional performance using FEMA P-58 procedures and performance metrics. In 2014, R.O. Hamburger [12]. summarized the key points of the next-generation performance-based assessment of buildings, including methodology, performance models, structural analysis, and performance calculation.

The methodology of FEMA-P58 project utilized the performance-based seismic engineering framework which was being developed by PEER [13] to represent earthquake performance in terms of likely values of crucial performance indicators, such as casualties, repair expenses, and loss of occupancy. The likely value of an earthquake loss measure is determined using a complex equation "triple integration of

$\{PM|DS\}\{DS|EDP\}\{EDP|I\}dz$". However, closed-form solutions for this equation are difficult, even for simple structural systems. To address this issue, a modified Monte Carlo approach was developed by Yang et al. [14] to implement the integration using inferred statistical distributions of building response. The FEMA P-58 methodology offers three types of performance assessments, intensity-based, scenario-based, and time-based, each providing performance functions based on different earthquake scenarios and uncertainties.

The FEMA P-58 performance model, including both structural and non-structural components. The components are classified based on their fragility and performance groups. Fragility groups are categorized using a system based on the NIST Uniformat II system [15], and performance groups consist of collections of components that will experience the same demand. The methodology selects damage states to represent a range of damage levels that have distinct consequences, such as specific repair procedures, probability of life loss, or post-earthquake occupancy consequences. The methodology includes Consequence Functions, which are probability distributions that capture the potential outcomes for each damage state, while accounting for uncertainty.

FEMA P-58 provide two methods for structural analysis to predict the median values of essential response parameters, nonlinear dynamic analysis and a simplified analysis method based on the ASCE 41–13 linear static procedure. The simplified method is only suitable for uniform structures without significant higher mode effects, while the nonlinear method is applicable to any type of structure. The simplified method transforms anticipated story drifts and spectral accelerations into median approximations of peak floor acceleration, peak floor velocity, and peak story drift using correlation coefficients for this procedure. while nonlinear analysis requires users to choose a collection of ground motions and adjust them to be consistent with the target spectrum(s).

The FEMA P-58 Performance Calculation uses a Monte Carlo approach to estimate probable loss distributions. In this method, the median response values and dispersions obtained from structural analysis are combined with modeling dispersion and hazard uncertainty. These demands are compiled into a vector of median values and a correlation matrix, which together with the dispersions, are assumed to represent a joint lognormal distribution.

Doubts have been raised regarding the calibration and validation of FEMA-P58 outcomes. In a study by Baker et al. [16] in 2016, the FEMA P-58 performance assessment methodology was evaluated by benchmarking it against observed earthquake data from the 2010–2011 Canterbury earthquake in New Zealand. The study aimed to use the methodology to predict damage and repair costs to buildings, which were then compared to post-earthquake evaluations of the buildings. Initial findings suggest that the P-58 predictions generally agree with observed data, which could provide valuable insights into the methodology's ability to capture building-specific features.

Stakeholders have raised concerns about legal liability associated with seismic performance information that indicates poor performance, with many expressing uncertainties about how to use estimated potential casualty numbers. There are also concerns about the potential for varying results from different engineers performing a FEMA P-58 assessment and potential gaming of the methodology. Both accuracy and credibility of the methodology need to be addressed.

Methods

During the structural design phase of buildings, architectural requirements often restrict the selection of optimal concrete dimensions for columns and other structural elements. This can result in the use of smaller column sections, leading to higher reinforcement ratios and accepting high values of design capacity ratio (DCR). These small, high DCR columns can be considered weak points in the structure, increasing the risk of progressive collapse in the event of extreme seismic activity or accidents. To mitigate this risk, designing an alternative load path is recommended to prevent total progressive collapse. However, incorporating such a path increases the cost of the structure. Therefore, evaluating the performance improvement and cost implications of structures with alternative load paths is necessary. The study models in this research adhere to ECP-203 [17] and ECP-201 [18] and its recommendations regarding the effect of vertical loads on residential buildings in addition to the lateral loads resulting from both earthquakes and winds in order to reach the concrete sections and the appropriate reinforcement values for spans, heights and the number of floors that were assumed for the research.. The effectiveness of designing an alternative load path is evaluated by assuming the presence of four weak columns with high DCR ratios (almost one) on the ground floor of the building. To carry out the analysis for both Progressive Collapse and Pushover, three-dimensional concrete moment resisting frames were subjected to Non-linear static and dynamic analysis using the CSI software SAP2000 version 18.1.1. In addition, the cost and time required for repair after various seismic events were evaluated using two tools of FEMA-P58, PACT (Performance Assessment Calculation Tool) and SPO2IDA (Static Pushover to Incremental Dynamic Analysis). The chosen models in the study are divided into two main groups based on the presence (case B) or absence (case A) of an alternative load path for preventing progressive collapse. This alternative approach involves strengthening the adjacent columns and beams to the weak columns and increasing their reinforcement ratios to enable them to withstand additional distributed stresses in the event of a collapse in one of the weak columns, Table 1 shows the properties of materials, Tables 2, 3, 4, 5 and 6 shows the concrete sections and RFT. For both cases (A)&(B). Each group is further divided into three sub-groups based on the location of weak columns, internal, external and corner. The models represent buildings with 6, 9, 12 and 15 floors and are analyzed using static pushover and FEMA-P58 methods for seismic behavior. Also the models are analyzed using nonlinear static and nonlinear dynamic methods for progressive collapse analysis. SAP2000 provides several methods for conducting nonlinear dynamic analysis. In this study, two methods were employed for the dynamic analysis. The first method is the primary approach, which is direct integration. The second method is the fast nonlinear analysis method (FNA),

Table 1 Properties of materials

Material	Strength (N/mm^2)	Unit Weight (kN/m^3)	Modulus of Elasticity E_s (N/mm^2)	Poisson's Ratio (ν)
Concrete	$F_{cu}=40$	25	27,828	0.20
Reinforcement Steel	F_U / F_y = 600/400	78.5	200,000	0.30

Table 2 Beams dimensions and reinforcement for analysis models

No. of Floors	Model Type	Dimensions (mm)	Reinforcement	
			Bottom	Top
6th Floors	Case (A)	200 × 600	4 ϕ 16	4 ϕ 16
	Case (B)	250 × 700	4 ϕ 18	4 ϕ 18
9th Floors	Case (A)	200 × 600	5 ϕ 16	5 ϕ 16
	Case (B)	250 × 700	5 ϕ 18	5 ϕ 18
12th Floors	Case (A)	200 × 600	6 ϕ 16	6 ϕ 16
	Case (B)	250 × 700	6 ϕ 18	6 ϕ 18
15th Floors	Case (A)	200 × 600	6 ϕ 18	6 ϕ 18
	Case (B)	250 × 700	6 ϕ 22	6 ϕ 22

Table 3 Columns dimensions and reinforcement for 6 floors models

Floors	Col Type	Case (A)		Case (B)	
		Dim. (mm)	RFT.—μ%	Dim. (mm)	RFT.—μ%
1st and 2nd	Main	350 X 350	12T18—2.5%	400 X 400	12T20—2.36%
	Weak	300 X 300	8T16—1.78%	300 X 300	8T16—1.78%
3rd and 4th	Main	250 X 250	8T16—2.6%	300 X 300	8T18—2.25%
	Weak	300 X 300	8T16—1.78%	300 X 300	8T16—1.78%
5th and 6th	Main	250 X 250	8T16—2.6%	300 X 300	8T18—2.25%
	Weak	300 X 300	8T16—1.78%	300 X 300	8T16—1.78%

Table 4 Columns dimensions and reinforcement for 9 floors models

Floors	Col Type	Case (A)		Case (B)	
		Dim. (mm)	RFT.—μ%	Dim. (mm)	RFT.—μ%
1st and 2nd	Main	400X400	12T22—2.85%	450X450	16T20—2.48%
	Weak	350X350	12T16—1.97%	350X350	12T16—1.97%
3rd and 4th	Main	350X350	12T18—2.5%	400X400	12T22—2.85%
	Weak	300X300	8T16—1.78%	300X300	8T16—1.78%
5th and 6th	Main	350X350	12T18—2.5%	350X350	12T18—2.5%
	Weak	300X300	8T16—1.78%	300X300	8T16—1.78%
7th, 8th and 9th	Main	250X250	8T16—2.6%	300X300	8T16—1.78%
	Weak	300X300	8T16—1.78%	300X300	8T16—1.78%

known for its speed, as it utilizes an iterative process to converge and achieve equilibrium. The effectiveness of the FNA formulation primarily arises from the decoupling of the nonlinear-object force vector $R_{NL}(t)$ from the elastic stiffness matrix and the damped equations of motion. The seismic behavior analysis is performed by FEMA-P58 for three different earthquake intensities corresponds to an earthquake with a 95 years, 475 years and 1230 years return period.

Geometry of analysis models consist of 6 bays in the X-dir. and 4 bays in the Y-dir., each with a span of 4 m. These models represent buildings with 6, 9, 12, and 15 floors,

Table 5 Columns dimensions and reinforcement for 12 floors models

Floors	Col Type	Case (A)		Case (B)	
		Dim. (mm)	RFT.—μ%	Dim. (mm)	RFT.—μ%
1st and 2nd	Main	450X450	16T22—3.00%	500X500	16T20—2.01%
	Weak	400X400	12T20—2.36%	400X400	12T20—2.36%
3rd and 4th	Main	400X400	12T22—2.85%	450X450	16T22—3.00%
	Weak	350X350	12T18—2.5%	350X350	12T18—2.5%
5th and 6th	Main	350X350	12T20—3.08%	400X400	12T22—2.85%
	Weak	350X350	12T16—1.97%	350X350	12T16—1.97%
7th, 8th and 9th	Main	350X350	12T16—1.97%	350X350	12T16—1.97%
	Weak	300X300	8T18—2.26%	300X300	8T18—2.26%
10th, 11th and 12th	Main	250X250	8T16—2.6%	300X300	8T16—1.78%
	Weak	300X300	8T16—1.78%	300X300	8T16—1.78%

Table 6 Columns dimensions and reinforcement for 15 floors models

Floors	Col Type	Case (A)		Case (B)	
		Dim. (mm)	RFT.—μ%	Dim. (mm)	RFT.—μ%
1st and 2nd	Main	500X500	16T22—2.43%	550X550	20T20—2.08%
	Weak	400X400	12T22—2.85%	400X400	12T22—2.85%
3rd and 4th	Main	450X450	16T22—3.00%	500X500	16T20—2.01%
	Weak	400X400	12T20—2.36%	400X400	12T20—2.36%
5th and 6th	Main	400X400	12T22—2.85%	450X450	16T22—3.00%
	Weak	350X350	12T18—2.5%	350X350	12T18—2.5%
7th, 8th and 9th	Main	400X400	12T20—2.36%	400X400	12T20—2.36%
	Weak	350X350	12T16—1.97%	350X350	12T16—1.97%
10th, 11th and 12th	Main	350X350	12T16—1.97%	350X350	12T16—1.97%
	Weak	300X300	8T18—2.26%	300X300	8T18—2.26%
13th, 14th and 15th	Main	250X250	8T16—2.6%	300X300	8T16—1.78%
	Weak	300X300	8T16—1.78%	300X300	8T16—1.78%

where the ground floor has a height of 4 m and the typical floors have a height of 3 m (Fig. 1).

One of the key inputs required to assess the seismic behavior of a building using FEMA-P58 is the initial building data, including the construction total cost and time. Table 7 provides the basic data for the study models.

Another significant input for the FEMA-P58 method is the selection of performance groups. These groups represent the anticipated behavior of elements within a group (both structural and non-structural elements) during a seismic event. Tables 8 and 9 illustrate the chosen performance groups per floor for structural and non-structural elements, respectively.

FEMA-P58 main inputs that needed to make an assessment for building seismic behavior using FEMA-P58 include the median estimates of story drift ratio, Δi* and total dispersion, β_{SD}. The purpose of calculating the median estimates of story drift ratio, Δi* is to correct story drift ratios to account for inelastic behavior and higher

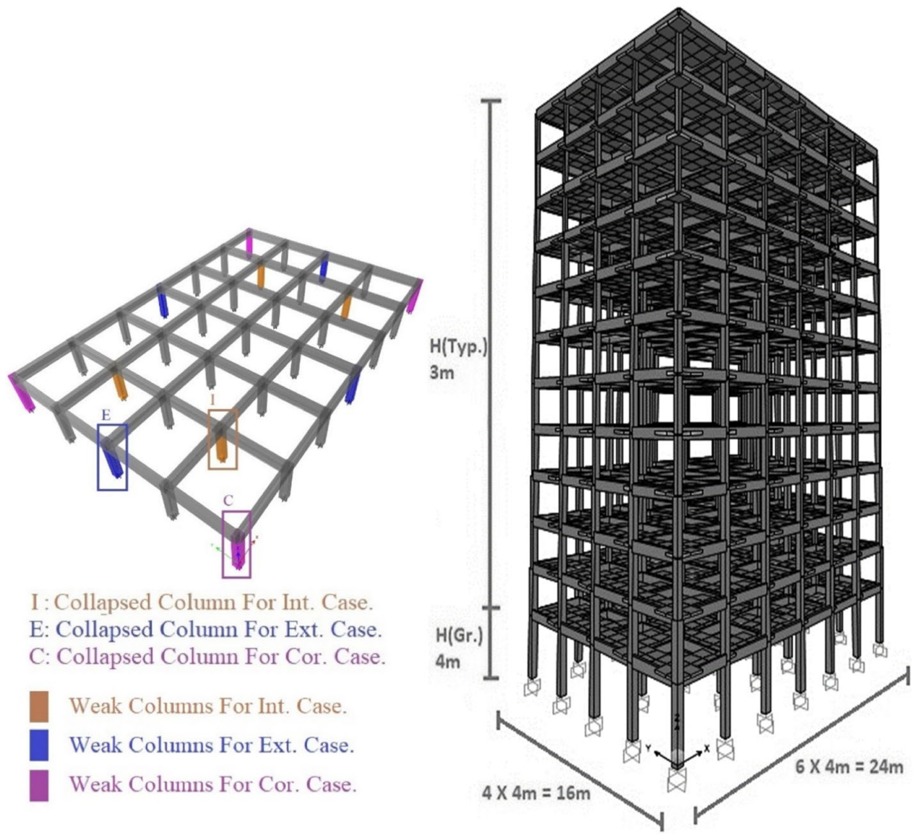

Fig. 1 Analysis models geometry

Table 7 FEMA-P58 basic data for all models

Item	Unit	6^{th} Floors	9^{th} Floors	12^{th} Floors	15^{th} Floors
Floor Area	m^2	384	384	384	384
Building Area	m^2	2304	3456	4608	5760
Total Replacement Cost (Case A)	$	6,200,012	9,300,019	12,400,025	15,500,031
Total Replacement Cost (Case B)	$	6,398,413	9,597,619	12,796,826	15,996,032
C&S* Replacement Cost (Case A)	$	2,480,005	3,720,007	4,960,010	6,200,012
C&S* Replacement Cost (Case B)	$	2,678,405	4,017,608	5,356,811	6,696,013
Replacement Time	Days	158	237	316	395
Carbon Emissions Replacement	kg	2,916,824	4,375,236	5,833,648	7,292,060.03
Embodied Energy Replacement	MJ	40,363,490	60,545,235	80,726,979.5	100,908,724.4

* Replacement cost, time, carbon and embodied energy are calculated based on FEMA-P58 Vol.(2)

* C&S indication to Core and Shell

mode effects. Estimates of median story drift ratio, Δi*, at each level i, can be calculated using the following equation:

$$\Delta_i^* = H_{\Delta i}(S, T_1, h_i, H) \times \Delta_i \quad (1)$$

Table 8 FEMA P58 quantities estimate for structural performance groups per floor

Fragility Number	Fragility Name	Quantity	
		Dir. X	Dir. Y
B1041.052a	ACI 318 OMF with weak beams and weak joints, beam flexural or shear response, Conc Col & Bm = 24" × 36", Beam one side	14.00	10.00
B1041.052b	ACI 318 OMF with weak beams and weak joints, beam flexural or shear response, Conc Col & Bm = 24" × 36", Beam both sides	21.00	25.00

Table 9 FEMA P58 quantities estimate for non-structural performance groups per floor

Fragility Number	Fragility Name	Avg. Quantity	
		Dir	Non Dir
B2022.001	Curtain Walls—Generic Midrise Stick-Built Curtain wall, Config: Monolithic, Lamination: Unknown, Glass Type: Unknown, Details: Aspect ratio = 6:5, Other details Unknown	20.67	-
B3011.011	Concrete tile roof, tiles secured and compliant with UBC94	-	13.23
C1011.001a	Wall Partition, Type: Gypsum with metal studs, Full Height, Fixed Below, Fixed Above	4.96	-
C3011.001a	Wall Partition, Type: Gypsum + Wallpaper, Full Height, Fixed Below, Fixed Above	1.58	-
C3032.001a	Suspended Ceiling, SDC A,B,C, Area (A): A < 250, Vert support only	-	15.71
C3032.001a	Suspended Ceiling, SDC A,B,C, Area (A): A < 250, Vert support only	-	0.83
D1014.011	Traction Elevator – Applies to most California Installations 1976 or later, most western states installations 1982 or later and most other U.S installations 1998 or later	-	0.84
D2021.011a	Cold or Hot Potable—Small Diameter Threaded Steel—(2.5 inches in diameter or less), SDC A or B, PIPING FRAGILITY	-	0.44
D3041.011a	HVAC Galvanized Sheet Metal Ducting less than 6 sq. ft in cross sectional area, SDC A or B	-	0.21
D3041.031a	HVAC Drops / Diffusers in suspended ceilings—No independent safety wires, SDC A or B	-	3.31
D3041.041a	Variable Air Volume (VAV) box with in-line coil, SDC A or B	-	1.65
D4011.021a	Fire Sprinkler Water Piping—Horizontal Mains and Branches—Old Style Victaulic—Thin Wall Steel—No bracing, SDC A or B, PIPING FRAGILITY	-	0.91
D4011.031a	Fire Sprinkler Drop Standard Threaded Steel—Dropping into unbraced lay-in tile SOFT ceiling—6 ft. long drop maximum, SDC A or B	-	0.50
D5012.021a	Low Voltage Switchgear—Capacity: 100 to < 350 Amp—Unanchored equipment that is not vibration isolated—Equipment fragility only	-	0.01

$$\ln(H_{\Delta i}) = a_0 + a_1 T_1 + a_2 S + a_3 \frac{h_{i+1}}{H} + a_4 \left(\frac{h_{i+1}}{H}\right)^2 + a_5 \left(\frac{h_{i+1}}{H}\right)^3 \tag{2}$$

S: The strength ratio which calculated using the Eq. (3.0):

$$S = \frac{W \times Sa(T_1)}{V_{y1}} \tag{3}$$

The values of the coefficients a0 through a5 are provided by FEMA-P58 Vol.(1) [19]. Due to the inherent uncertainty in ground motion, mathematical response modeling, and variations in material properties, the actual response of a building is expected to deviate from the estimated median response. These uncertainties are characterized as record-to-record variability, $\beta_{a\Delta}$, and modeling uncertainty, β_m. The combined dispersion, known as β_{SD}, associated with the peak transient drift, can be expressed as follows:

$$\beta_{SD} = \sqrt{\beta^2{}_{a\Delta} + \beta^2{}_m} \tag{4}$$

The dispersion values that based on the above equation are provided in FEMA-P58 Vol.(2) [20] (Table 10).

Loading factors recommended by GSA were used for structural models according to analysis type as following:

•Static non-linear analysis procedures.

To assess the structural resistance to progressive collapse, a nonlinear static analysis was performed at the column removal location using the full load method. The gravity loads were gradually increased step by step, following the load factor guidelines outlined in GSA, as expressed in Eq. (5):

$$\mathrm{GN} = \Omega\mathrm{N} * (\mathrm{DL} + 0.25\mathrm{LL}) \tag{5}$$

GN = Gravity loads for non-linear static analysis.

DL = Dead loads including structure element self-weight.

LL = Live loads.

ΩNS = Dynamic increase (amplification) factor for non-linear static analysis.

According to GSA dynamic increase factor can assumed to be equal (2.0) for concrete structures.

- Dynamic non-linear time history procedure

Failure of structural members lead to a disproportionate collapse, Columns failed generally based on two scenarios:

- Gradual failure, where structure element loss the stiffness gradually during long time and start to distribute loads for adjacent elements.
- Sudden failure, where structure element loss the stiffness suddenly and during very short time causing more stresses at adjacent structure elements surrounding the location of failure element.

Table 10 Correction factors for story drift ratio for moment resisting frames system

Correction Factors For Story Drift Ratio	a_0	a_1	a_2	a_3	a_4	a_5
6^{th} Floors	0.75	-0.044	-0.010	-2.58	2.30	0
9th Floors	0.75	-0.044	-0.010	-2.58	2.30	0
12^{th} Floors	0.67	-0.044	-0.098	-1.37	1.71	-0.57
15th Floors	0.67	-0.044	-0.098	-1.37	1.71	-0.57

Non-Linear dynamic analysis was applied using SAP2000, the elements subjected to sudden removal was investigated in this research. According to GSA guidelines the following combination was applied to vertical load:

$$\mathrm{GND} = (\mathrm{DL} + 0.25 * \mathrm{LL}) \tag{6}$$

DL = Dead loads including structure element self-weight.

LL = Live loads.

GND = Gravity loads for dynamic Non-linear time history analysis.

To account for the non-linear behavior of the structure after applying vertical loads, a ramp function with a duration of 20 s was used to apply all vertical loads before simulating the collapse of weak columns or progressive collapse phenomena. For simulating the collapse of weak columns, the weak column was replaced by equivalent reaction forces to simulate its absence. A time-history analysis was then performed, in which the equivalent column loads were gradually reduced to zero over a short period of time, matching the duration of the column removal event. A collapse function was created to simulate the collapse of the weak column and progressive collapse.

To simulate progressive collapse as per GSA guidelines as mentioned in Eq. (6), the vertical loads were applied to the models using a gradually increasing time function with a duration of 20 s. After 5 s, sudden collapse was assumed to occur in one of the four weak columns at ground level, and the equivalent reaction force was suddenly reduced to zero using the collapse function time history Fig. 2. The analysis continued for an additional 15 s to capture all possible behavior, causing vertical vibrations at the column removal point. The time history function used for applying vertical load and joint reaction forces is shown in Fig. 3. Vertical displacement curves with time was plotted at the joint of the removed column in results to compare the deflection curves of structural models. Although there are four weak columns in each model at ground level, sudden collapse was assumed to occur in only one of them. Due to the

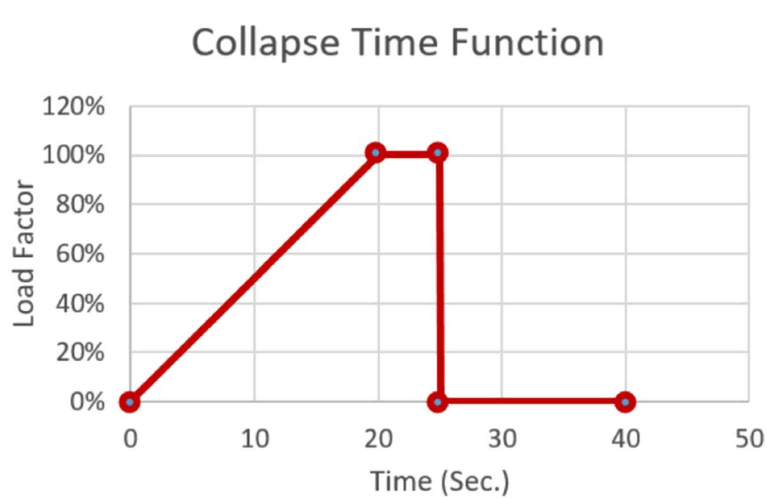

Fig. 2 Time history collapse function

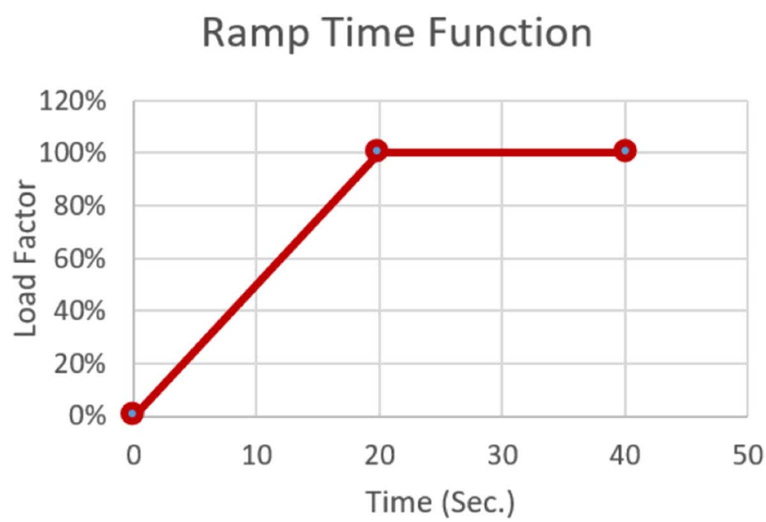

Fig. 3 Time history ramp function

redistribution of stress on the other members, gradual failure was assumed to occur in the other three weak columns.

Plastic hinge definition was use according to FEMA-356 [21] provisions with one degree of freedom plastic hinges (M_3) for beams and three degree of freedom plastic hinges (P -M_2-M_3) for columns for all models.

To idealize the pushover curve into linear relationship, FEMA-P58 idealized method have been used to approximated the force–displacement relationship(s) into idealized piecewise linear representations.

In this study have been used mode to define the lateral pushover load case for all studying cases using the top left point of the middle frame to apply the load case and by using displacement control with multiple steps.

To include the vertical loads in the analysis, The initial condition of pushover case is to continue from state at end of another non-linear case which include only the vertical loads with following loads factors:

$$\mathrm{GN} = (\mathrm{DL} + 0.25\mathrm{LL}) \tag{7}$$

GN = Gravity loads for non-linear static analysis.

DL = Dead loads including structure element self-weight.

LL = Live loads.

The hazard curve associated with the building being assessed must be determined in order to conduct a performance analysis using FEMA P-58. Broadly speaking, a hazard curve represents the average annual frequency of exceeding a certain level of spectral acceleration. This curve needs to be calculated specifically for the site of interest, which is identified by its latitude and longitude. To determine the hazard curve for any given building, the hazard curve for the city where the building is located must first be established. To establish the hazard curve for the city of Cairo, seismic data and a unified hazard curve for Cairo at return periods of 224, 615, 1230, and 4745 years, provided by A. Badawy et al. in 2016 [22], were utilized. This data was then adjusted to align with the

Table 11 Acceleration values for 1 s. Spectral period for different return periods

Probability of Exceedance	Return Period (years)	λ	Sa(g) (T = 1 s.)
10% in 10 years	95	0.010526316	0.0299
20% in 50 years	224	0.004464286	0.05
10% in 50 years	475	0.002105263	0.0749
15% in 100 years	615	0.001626016	0.08
15% in 200 years	1230	0.000813008	0.11
10% in 500 years	4745	0.000210748	0.18

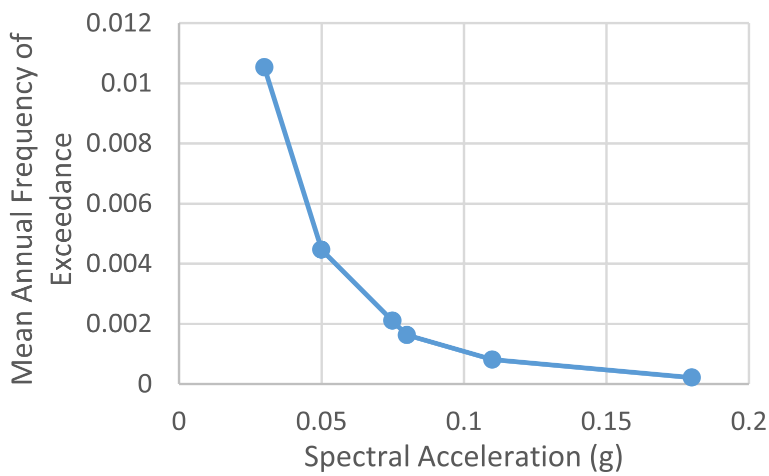

Fig. 4 Hazard curve for Cairo for 1 s. spectral period

FEMA P-58 methodology, which focuses on the relationship between the average annual frequency of exceedance and the spectral acceleration for a 1-s time period. To carry out this adjustment, the spectral acceleration values for various return periods at the 1-s period were utilized. Through the application of interpolation and extrapolation techniques, estimated values for the spectral acceleration for additional return periods were derived. These values are presented in Table 11.

Using the presented data in Table 11, the hazard curve for Cairo city for 1 Sec. spectral period can be illustrated as shown in the Fig. 4:

Using the same procedure, it becomes straightforward to determine the hazard curve for any building based on its specific time period.

Three different return periods were utilized to calculate the building acceleration, Sa(T), in order to capture the building's response to various seismic events. The first one is 95 years, this represents the threshold specified by the Egyptian code (ECP-201), where all buildings must possess the capability to withstand this seismic event without sustaining any damage, including non-structural elements. The second return period is 475 years, this return period signifies the limit set by the Egyptian code (ECP-201), requiring buildings to have sufficient strength to withstand such seismic events. However, some elements of the building may experience damage due to the high expected displacement of the building floors. The last return period is 1230 years, this return

Table 12 Plastic hinges Modeling Parameters and Acceptance Criteria

No. of Floors	ALP Case	Modeling Parameters			Plastic Rotation Angle, radians			
		Plastic Rotation Angle, radians		Residual Strength Ratio	Performance Level			GSA Acceptance Criteria
		a	b	c	IO	LS	CP	
6th	A	0.01	0.02	0.2	0.0015	0.005	0.01	0.02
	B	0.015	0.02	0.2	0.002	0.007	0.015	0.02
9th	A	0.018	0.025	0.2	0.0028	0.009	0.018	0.025
	B	0.018	0.025	0.2	0.0028	0.009	0.018	0.025
12th	A	0.02	0.03	0.2	0.004	0.01	0.02	0.03
	B	0.02	0.03	0.2	0.004	0.01	0.02	0.03
15th	A	0.02	0.03	0.2	0.005	0.01	0.02	0.03
	B	0.02	0.03	0.2	0.005	0.01	0.02	0.03

Table 13 Plastic hinges rotation results for static models

No. of Floors	Performance Level	No. of Plastic Hinges That Reach Each Performance Level					
		Case (A)			Case (B)		
		Int	Ext	Cor	Int	Ext	Cor
6^{th}	A to IO	560	576	601	584	627	645
	IO to LS	132	118	47	112	69	51
	LS to CP	4	2	33	0	0	0
	> CP	0	0	15	0	0	0
9^{th}	A to IO	875	872	937	910	908	978
	IO to LS	167	157	95	134	134	66
	LS to CP	2	15	12	0	2	0
	> CP	0	0	0	0	0	0
12^{th}	A to IO	1198	1182	1248	1227	1223	1294
	IO to LS	168	171	119	163	153	91
	LS to CP	26	26	14	2	15	7
	> CP	0	13	11	0	1	0
15^{th}	A to IO	1540	1501	1565	1568	1551	1623
	IO to LS	172	188	136	166	165	101
	LS to CP	28	32	22	4	15	13
	> CP	0	19	17	0	7	1

period represents a severe seismic event that could result in the loss of resistance capabilities for many buildings.

Results and discussion

When studying progressive collapse, it is crucial to consider various indicators as outlined by relevant codes like GSA. Key indicators include the number of plastic hinges formed in the building due to the failure of a weak element and the degree of rotation, which provides insights into the behavior of remaining elements after partial collapse. Additionally, indicators such as the design/capacity ratio for columns and the

Table 14 Plastic hinges rotation results for dynamic models

No. of Floors	Performance Level	No. of Plastic Hinges That Reach Each Performance Level					
		Case (A)			Case (B)		
		Int	Ext	Cor	Int	Ext	Cor
6th	A to IO	682	679	684	692	692	696
	IO to LS	14	17	12	4	4	0
	LS to CP	0	0	0	0	0	0
	> CP	0	0	0	0	0	0
9th	A to IO	1014	1007	1022	1026	1026	1034
	IO to LS	30	37	22	18	18	10
	LS to CP	0	0	0	0	0	0
	> CP	0	0	0	0	0	0
12th	A to IO	1328	1329	1361	1358	1358	1374
	IO to LS	64	63	31	34	34	18
	LS to CP	0	0	0	0	0	0
	> CP	0	0	0	0	0	0
15th	A to IO	1666	1665	1701	1699	1681	1709
	IO to LS	74	75	39	39	57	29
	LS to CP	0	0	0	0	0	0
	> CP	0	0	0	0	0	0

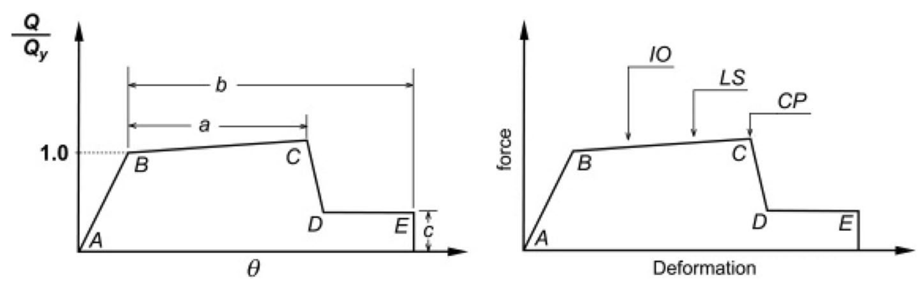

Fig. 5 Plastic Hinges Parameters and Rotation Levels

examination of deflection values at locations of partial collapse help assess the likelihood of progressive collapse. Figure 5 and Table 12 illustrates the numerical values of the plastic hinge parameters and the GSA acceptance criteria. Tables 13 and 14 shows the plastic hinges results for nonlinear static and nonlinear dynamic analysis models respectively.

The results indicate that models designed with an alternative load path exhibit a clear advantage in terms of the degree of rotation of plastic hinges compared to models without it Figs. 6, 7 and 8. The presence of an alternative load path leads to lower rotation values for plastic hinges and a lower number of joints reaching the collapse prevention stage. In static analysis, the location of the weak column has a slight effect on the plastic joint rotation. For 6-story models, corner case causes the highest rotation. In 9-story models, outer columns cause the highest rotation, while in 12-story models, the performance of plastic hinges is similar for inner columns and edges, but corners rotate to a lesser degree. Similar trends are observed in 15-story models compared to 12-story models. Models with an alternative load path exhibit lower rotation values for plastic hinges compared to those without, indicating protection against progressive collapse.

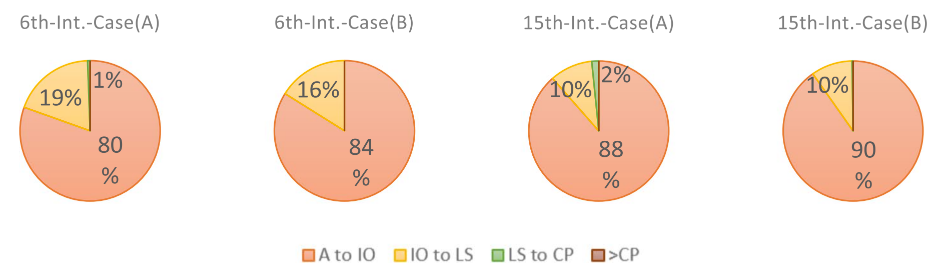

Fig. 6 Percentage of Plastic Hinges That Reach Each Performance Level for 6^{th}&15^{th}-Int.-Static models

Table 15 Number of columns reached high (DCR) for all models

No. of Floors	DCR	Case (A)			Case (B)		
		Int	Ext	Cor	Int	Ext	Cor
6^{th}	1.25 ~ 1.50	102	28	30	8	16	10
	1.50 ~ 2.0	11	25	29	28	22	23
	> 2.0	28	14	46	0	8	17
	Tot. Col. With DCR > 1	141	67	105	36	46	50
9^{th}	1.25 ~ 1.50	36	17	25	0	30	42
	1.50 ~ 2.0	0	56	37	56	36	21
	> 2.0	56	24	26	0	22	31
	Tot. Col. With DCR > 1	92	97	88	56	88	94
12^{th}	1.25 ~ 1.50	0	41	36	28	34	23
	1.50 ~ 2.0	56	33	49	28	28	40
	> 2.0	0	9	4	0	8	6
	Tot. Col. With DCR > 1	56	83	89	56	70	69
15^{th}	1.25 ~ 1.50	28	44	52	28	35	18
	1.50 ~ 2.0	28	30	34	0	29	50
	> 2.0	0	6	3	0	5	4
	Tot. Col. With DCR > 1	56	80	89	28	69	72

Comparing different analysis methods for progressive collapse reveals that dynamic analysis aligns with static analysis results. The plastic hinge results demonstrate the positive influence of the alternative load path in reducing hinge rotation. However, in dynamic analysis, rotation values fluctuate over time before stabilizing at a certain value, typically lower than the maximum rotation recorded. Static analysis for progressive collapse requires a dynamic correction factor to account for dynamic effects on structural behavior, often conservative (reaching up to 2), leading to increased rotation values in the static analysis.

The design capacity ratio for columns is a significant indicator when assessing a building's resistance to progressive collapse. It is important to note that column capacity, as determined by codes, incorporates conservative safety factors. Therefore, the allowed design capacity ratio before considering column collapse is typically greater than 1 in progressive collapse analysis. Dynamic analysis results generally give lower design capacity ratio compared to static analysis due to the conservatism in the applied dynamic correction factor. Table 15 presents the design capacity ratio for columns in the analyzed models. It is clear that models with an alternative load path exhibit a minimal number of

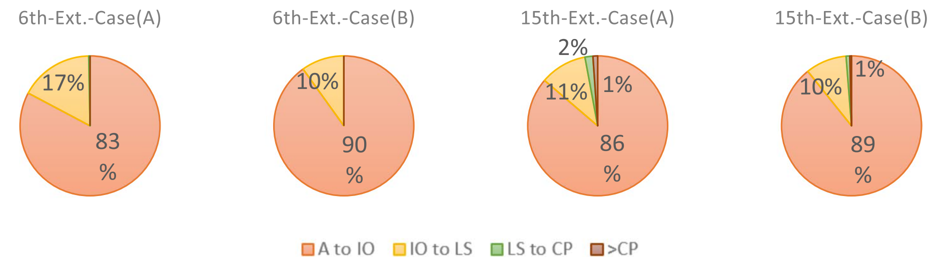

Fig. 7 Percentage of Plastic Hinges That Reach Each Performance Level for 6th&15th-Ext.-Static models

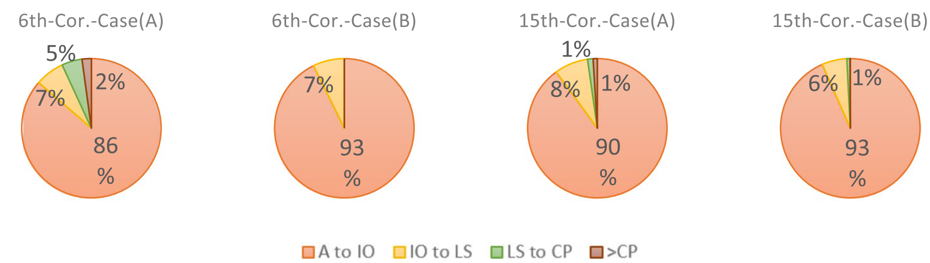

Fig. 8 Percentage of Plastic Hinges That Reach Each Performance Level for 6th&15th-Cor.-Static models

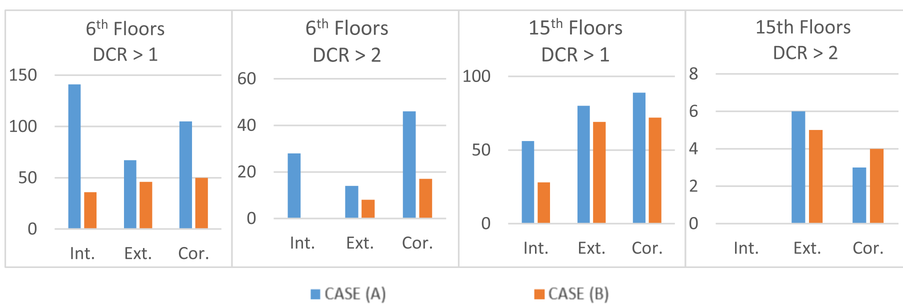

Fig. 9 Number of columns reached high DCR for 6th & 15th floors models

columns exceeding the acceptance criteria, indicating a lower likelihood of progressive collapse. In contrast, models without an alternative load path show a noticeable number of columns surpassing the acceptance criteria set by GSA, indicating a higher possibility of progressive collapse (Fig. 9).

The increase in deflection values following column damage significantly raises the internal forces on surrounding elements, greatly increasing the risk of their collapse. Deflection values were examined at various locations of weak columns using three analysis methods: static analysis, direct integration dynamic analysis, and fast nonlinear dynamic analysis (FNA). FNA results consistently exhibited a similar pattern to dynamic analysis but with different values, lacking the same accuracy in capturing the rate of deflection change over time. FNA can be used for quick analysis to obtain approximate results regarding the building's behavior but should not be solely relied upon for highly accurate results (Fig. 10). Static analysis yielded greater deflection values compared to dynamic analysis due to the conservative nature of the dynamic correction factor used in static analysis (Fig. 11). The investigation of the alternative load path's effectiveness in enhancing building behavior and reducing progressive collapse risk revealed significant

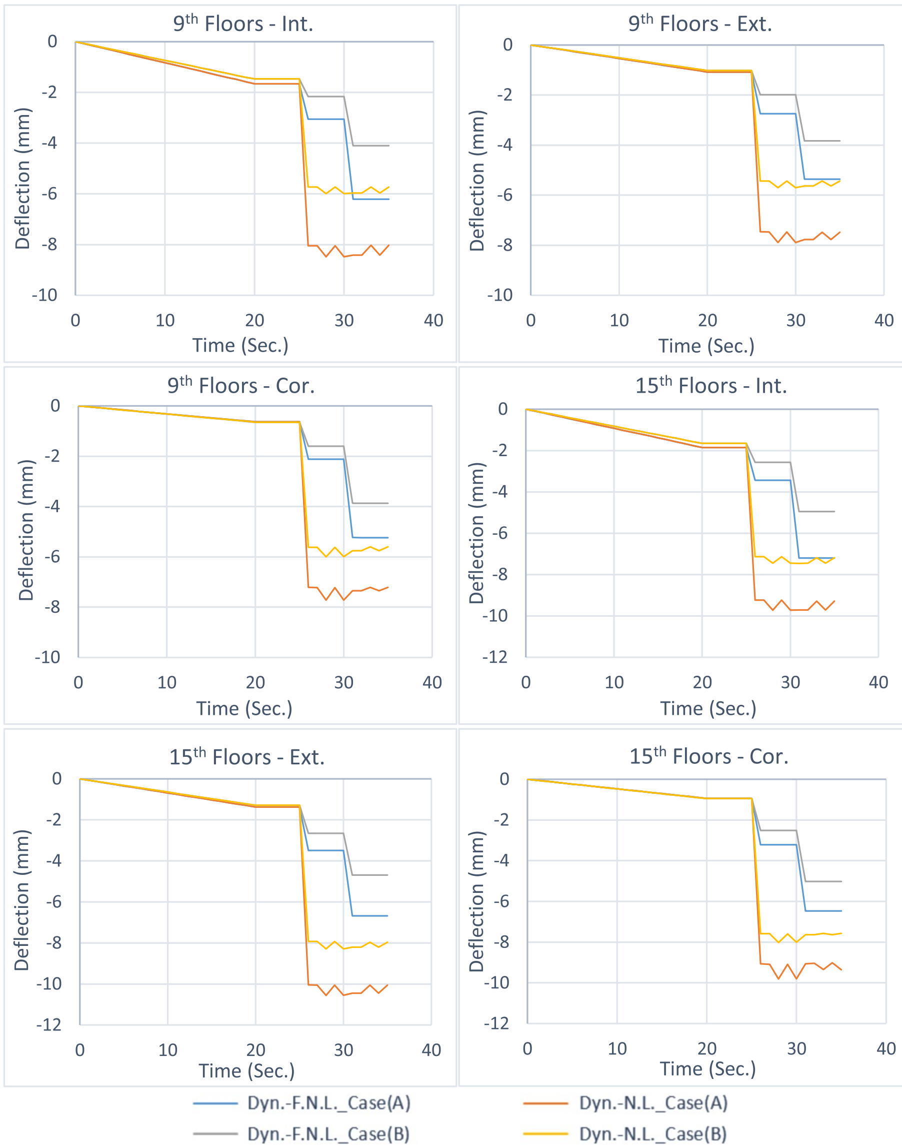

Fig. 10 Deflection at removed column point with time for 9[th] & 15[th] floors for dynamic models

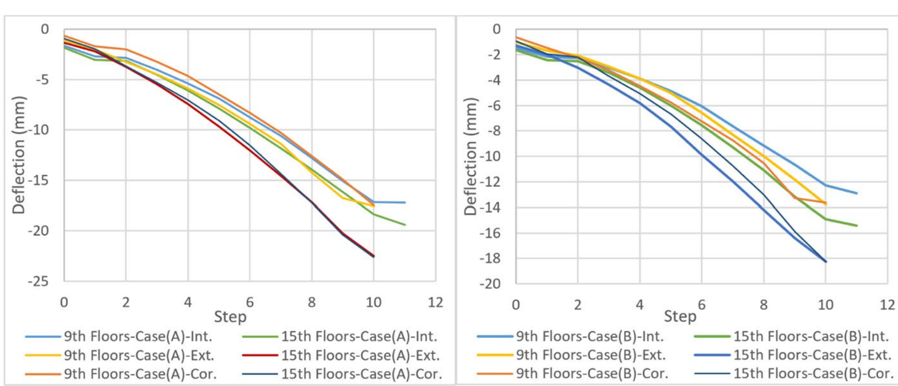

Fig. 11 Deflection at removed column point for 9[th] & 15[th] floors for static models

improvements in deflection values for models incorporating the alternative load path (Table 16).

Having reviewed the results of the progressive collapse analysis, it is now time to present the findings of the seismic assessment conducted using Pushover and FEMAP58. Non-linear static pushover analysis is widely recognized as a popular method for evaluating building performance during seismic events. The pushover results for all the models studied are summarized in Table 17.

Figure 12 illustrates the pushover curves for the 6th and 15th floor models. The FEMA-P58 idealized method was employed to approximate the force–displacement relationship using piecewise linear representations. For the 6th floor models, it is evident that regardless of the weak column location, the models in case (B) exhibit higher resistance to lateral forces compared to those in case (A). This increase in building capacity does not compromise the building's ductility, as both case (A) and case (B) show similar ranges of ductility index. Regarding the 15th floor models, the models in case (B) also demonstrate superior resistance to lateral forces regardless of the weak column location. However, the improvement in behavior is limited, particularly when compared to the 6th floor models. It is important to consider the substantial cost difference associated with implementing the alternative load path in these models.

The results of the studied models after assessing them using FEMA-P58 will focus on the expected cost and time of repair which considered the most important results provided by the new assessment method FEMA-P58. As indicated before, three different intensities were used to evaluate the models, 10% in 10 years-(95 years), 10% in 50 years-(475 years) and 15% in 200 years-(1230 years). It has been noticeable that for intensity that corresponding the 95 years return period, The results of the evaluation showed that the possibility of damage to any of the performance groups that make up the building is almost zero. And this is fully consistent with the philosophy of the Egyptian Code for Loads, which requires that the building have sufficient capacity to resist earthquakes with a close return time, specifically 95 years, without causing any damage even to

Table 16 Max. deflection values for different analysis methods

Analysis Method	No. of Floors	Max. Deflection Value (m)					
		Case (A)			Case (B)		
		Int	Ext	Cor	Int	Ext	Cor
Static-N.L	6th	0.01624	0.0155	0.0159	0.0117	0.0103	0.01155
	9th	0.01724	0.0176	0.0176	0.01289	0.01375	0.01364
	12th	0.01922	0.02075	0.02075	0.01475	0.01683	0.01628
	15th	0.01941	0.02249	0.0226	0.01541	0.01824	0.01826
Dynamic-N.L	6th	0.00800	0.00632	0.00647	0.0052	0.00427	0.00465
	9th	0.00803	0.00745	0.00721	0.00573	0.00543	0.0056
	12th	0.00899	0.00903	0.00827	0.00662	0.00705	0.00664
	15th	0.00928	0.01006	0.00936	0.00718	0.00796	0.00757
Dynamic-F.N.A	6th	0.00609	0.00462	0.00402	0.00394	0.00333	0.00218
	9th	0.00621	0.00536	0.00524	0.00409	0.00383	0.00387
	12th	0.00683	0.00638	0.00564	0.00476	0.00456	0.00461
	15th	0.00720	0.00668	0.00647	0.00495	0.00469	0.00502

Table 17 Pushover analysis main results for studied models

Input/Output Type	No. of Floors	Case (A)			Case (B)		
		Int	**Ext**	**Cor**	**Int**	**Ext**	**Cor**
Fundamental Period T_1 (sec.)	6th	1.36	1.36	1.37	1.07	1.07	1.07
	9th	1.55	1.55	1.54	1.3	1.3	1.28
	12th	1.85	1.85	1.85	1.57	1.56	1.55
	15th	2.1	2.09	2.1	1.82	1.81	1.8
Yielding Base Shear V_y (kN)	6th	1423.4	1467.9	1378.9	2468.8	2023.9	2112.9
	9th	1912.7	1912.7	2001.7	2446.5	2646.7	2668.9
	12th	2112.9	1912.7	2290.8	2757.9	2735.6	2958.1
	15th	2802.4	2379.8	2780.1	3047	3002.5	3047
Ultimate Base Shear V_u (kN)	6th	2235.3	2216.1	2186.9	3569.1	3543.2	3542.4
	9th	3639.5	3590.4	3547.1	5058	5039.8	4933.1
	12th	4538.5	4490.2	4422.2	6152.9	6165.1	6213.2
	15th	5490.1	5434.2	5364.7	6936.1	6950.5	7190.1
Yielding Roof Disp. Бy (m)	6th	0.04	0.041	0.041	0.04	0.031	0.033
	9th	0.055	0.055	0.059	0.041	0.047	0.048
	12th	0.071	0.061	0.067	0.052	0.054	0.065
	15th	0.083	0.091	0.082	0.077	0.063	0.077
Ultimate Roof Disp. Бu (m)	6th	0.075	0.074	0.073	0.07	0.072	0.073
	9th	0.139	0.135	0.133	0.138	0.127	0.117
	12th	0.22	0.216	0.197	0.218	0.217	0.213
	15th	0.275	0.31	0.266	0.285	0.275	0.3
Ductility Index μ	6th	1.89	1.8	1.79	1.74	2.31	2.2
	9th	2.54	2.47	2.27	3.34	2.69	2.47
	12th	3.1	3.54	2.96	4.17	4.04	3.27
	15th	3.32	3.39	3.23	3.72	4.38	3.91

non-structural elements. Therefore, the total repair cost and time results presented in this paper will be limited to only two intensities:

- Intensity (1): whish represent a seismic event have return period = 475 years.
- Intensity (2): whish represent a seismic event have return period = 1230 years.

Tables 18 and 19 show the estimated total repair cost and time respectively.

Figures 13 and 14 illustrate the same concept as the pushover curve results, indicating that models with an alternative load path are expected to require lower repair costs and less time compared to models without this alternative path. This observation holds true for both the 9th floor and 12th floor models. However, the enhancement in behavior for the 12th floor models is relatively limited, especially for Intensity (2), considering the significant cost difference associated with implementing the alternative load path in these models. Not only that, but the results of the corner models for 12th floors also indicate that the alternative load path may has a negative impact on the expected cost and time for repairs.

Conclusions

Selecting an alternative load path to mitigate the risk of progressive collapse while maintaining the seismic behavior of the building presents a challenge. in this research the enhancement in the seismic behavior of the building as a result of using the alternative

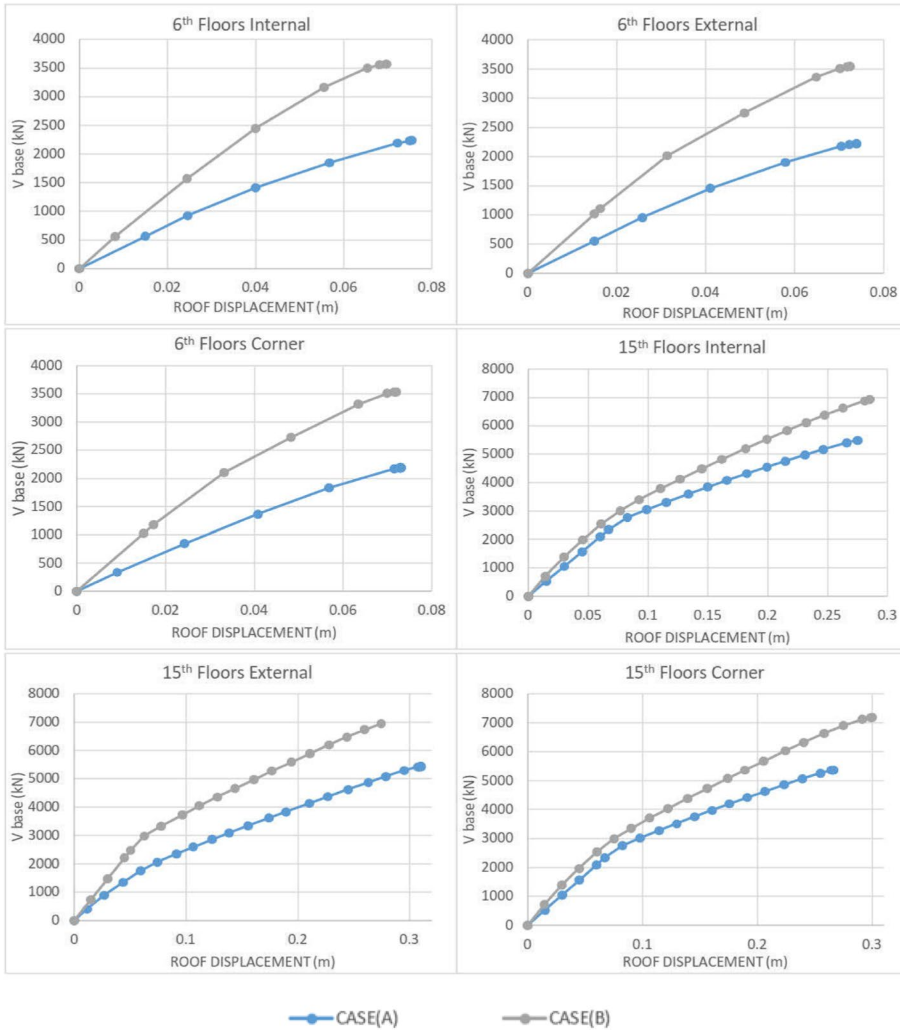

Fig. 12 Pushover curves for the 6th and 15th floor models

Table 18 FEMA-P58 estimated total repair cost for all models

Intensity Type	No. of Floors	Repair Cost ($)					
		Case (A)			Case (B)		
		Int	Ext	Cor	Int	Ext	Cor
Intensity (1)	6th	21,102	21,315	29,780	15,620	8938	8938
	9th	17,911	19,637	19,822	6681	7129	8396
	12th	33,923	33,210	33,974	15,347	15,560	16,114
	15th	46,253	46,133	46,693	25,493	25,166	24,540
Intensity (2)	6th	6,200,012	6,200,012	6,200,012	6,200,012	6,200,012	6,200,012
	9th	679,729	819,062	874,935	500,496	480,346	642,243
	12th	1,058,729	914,651	634,093	860,218	844,220	943,981
	15th	993,827	1,804,120	890,185	1,307,107	874,407	1,552,845

Table 19 FEMA-P58 estimated repair time for all models

Intensity Type	No. of Floors	Repair Time (Days)					
		Case (A)			Case (B)		
		Int	Ext	Cor	Int	Ext	Cor
Intensity (1)	6th	0.4	0.4	0.6	0.3	0.2	0.2
	9th	0.4	0.4	0.4	0.2	0.2	0.2
	12th	0.7	0.7	0.7	0.3	0.3	0.3
	15th	1	1	1	0.5	0.5	0.5
Intensity (2)	6th	158	158	158	158	158	158
	9th	11.8	14.2	15.1	8.4	9.4	11.8
	12th	20.6	17.9	12.6	16.8	16.4	18
	15th	18.8	37.2	16.7	28.7	17.6	31.3

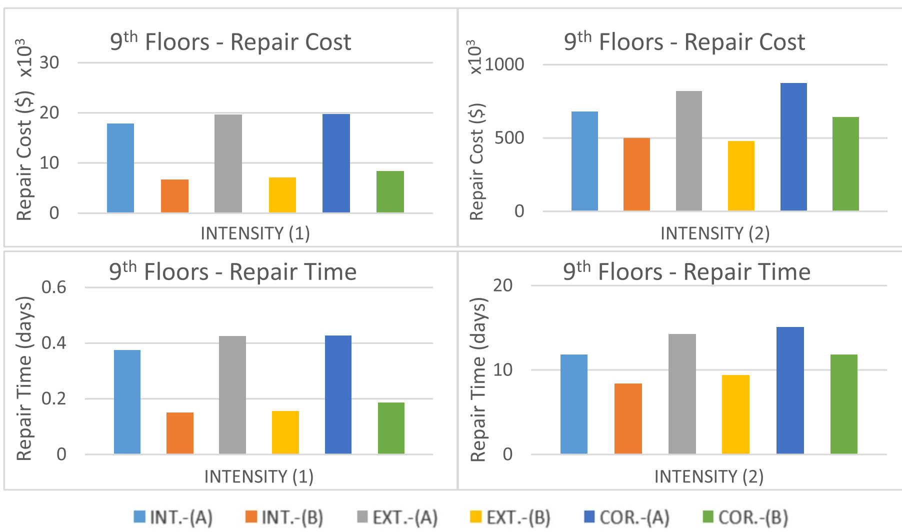

Fig. 13 Comparison between expected cost and time of repair for 9th floors models (Int. & Ext. & Cor.) for Intensities (1) & (2)

load path has been investigated. The main conclusions of this research are presented as follows:

1. Plastic hinges in alternative load path systems have lower rotation values than in ordinary systems, which support the effectiveness of these systems. These systems also have fewer columns exceeding the D/C ratio after weak column collapse.
2. Collapsing corner columns results in more high DCR columns, while collapsing interior columns leads to fewer high DCR columns.
3. Deflection values of slabs at collapsed column locations are generally lower in alternative load path systems, which also supporting the effectiveness of alternative load path systems.
4. The efficiency of alternative load path systems is significantly higher in low and mid-height buildings compared to tall buildings. This is mainly due to the higher lateral forces that tall buildings are designed to withstand in ordinary systems without alternative load paths, resulting in larger column and beam dimensions with higher

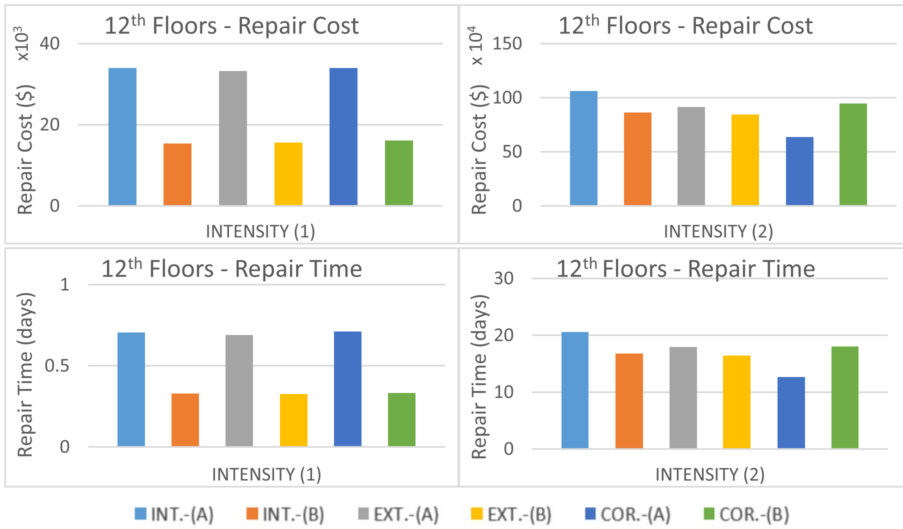

Fig. 14 Comparison between expected cost and time of repair for 12th floors models (Int. & Ext. & Cor.) for Intensities (1) & (2)

reinforcement ratios. This concept aligns with the idea of alternative load paths in the event of a collapse caused by vertical loads only.

5. Non-linear static analysis with dynamic impact factors yields results close to dynamic analysis, but the commonly used dynamic factor of two is considered conservative.
6. Models with alternative load path systems exhibit significant improvements in both yield base shear and ultimate base shear compared to ordinary models.
7. For earthquakes with a probability of occurrence exceeding 10% in 50 years, alternative load path systems reduce rehabilitation costs and time compared to ordinary systems.
8. Using the alternative load path method increases building structure system cost by about 10%. However, the total cost of rehabilitation for ordinary buildings is more than double that of buildings with the alternative load path, mainly due to non-structural elements. The effectiveness of the alternative load path system depends on the cost ratio of non-structural elements to structural elements.
9. For severe earthquakes with a probability of occurrence exceeding 15% in 200 years, alternative load path systems reduce losses in medium-height buildings. However, the benefits decrease as building height increases, and the alternative load path may has a negative impact on the expected cost and time for repairs.
10. Seismic assessment methods, such as non-linear pushover analysis and the FEMA-P58 method, provide similar predictions for building behavior.
11. FEMA P-58 method providing a simplified understanding of the behavior of buildings under seismic events in compare with traditional seismic assessment methods like pushover. It particularly benefits non-engineers by offering clear financial and temporal insights into various structural system solutions.

Abbreviations

PM	Performance Measure (e.g., repair cost)
DS	Damage State
EDP	Engineering Demand Parameter
I	Intensity of ground motion
dz	Integration over the range of seismic hazards
Δ_i^*	Estimates of median story drift ratio.
Δ_i	Uncorrected story drift ratio.
$H_{\Delta i}$	Drift correction factor.
β_{SD}	Total dispersion.
$\beta_{a\Delta}$	Record-to-record variability.
β_m	Modeling uncertainty.
$MAFE(\lambda)$	Mean Annual Frequency of Exceedance.
IO	Immediate Occupancy level
LS	Life Safety level
CP	Collapse Prevention level

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Authors' contributions
M.N.A. and W.A.A., conceptualization and methodology; W.A.A., validation, verification, and supervision; M.N.A., experimental investigation, writing original draft and editing. All authors have read and approved the manuscript.

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Declarations

Competing interests
The authors declare that they have no competing interests.

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References

1. General Services Administration, GSA, (2016), Alternate path analysis and design guidelines for progressive collapse resistance revision (1).
2. Department of Defense (DoD) (2016), Design of buildings to resist progressive collapse, Unified Facilities Criteria (UFC 4- 023–03) Washington DC.
3. American Society of Civil Engineers, ASCE, (2022), Minimum design loads for buildings and other structures, Report ASCE/SEI, 7–22.
4. Marjanishvili SM (2004) Progressive analysis procedure for progressive collapse. J Perform Constr Facil ASCE 18(2):79–85. https://doi.org/10.1061/(ASCE)0887-3828(2004)18:2(79)
5. Starossek U (2007) Typology of progressive collapse. Eng Struct 29(9):2302–2307. https://doi.org/10.1016/j.engstruct.2006.11.025
6. Sezen H. and Giriunas K.A. (2009), Progressive Collapse Analysis of An Existing Building" M Sc. thesis, The Ohio State University.https://www.aisc.org/globalassets/aisc/research-library.
7. Lu X, Lu X, Guan H, Ye L (2013) Collapse simulation of reinforced concrete high-rise building induced by extreme earthquakes, earthq eng struct. Dyn 42(5):705–723. https://doi.org/10.1002/eqe.2240
8. Song B, Sezen H (2013) Experimental and analytical progressive collapse assessment of steel frame building. J Eng Struct. 56:664–672. https://doi.org/10.1016/j.engstruct.2013.05.050. (Columbus OH, USA.)
9. Botez M, Bredean L, Ioani A. (2014), Numerical methods in progressive collapse assessment of rc framed structures, international congress on computational mechanics simulation, 5, 978–981, Cluj-Napoca, Romania. https://doi.org/10.3850/978-981-09-1139-3_213.
10. Gamaniouk T. (2014), Parametric analysis of progressive collapse in high-rise buildings, M Sc. thesis, massachusetts institute of technology.https://www.dspace.mit.edu/bitstream/handle/1721.1/89848/890135822-MIT.pdf.
11. Tavakoli, H.R., and Hasani, A.H. Effect of Earthquake characteristics on seismic progressive collapse potential in steel moment resisting frame", Earthquakes and Structures. 2017;5(12):529–541. https://doi.org/10.12989/eas.2017.12.5.529.
12. R. O. Hamburger, SE. (2014), FEMA P58 Seismic Performance Assessment of Buildings, Tenth U.S. National Conference on Earthquake Engineering, Frontiers of Earthquake Engineering, Anchorage, Alaska.https://www.researchgate.net/publication/289270679_FEMA_P58.
13. Moehle, J.P. and Deirelein, G.G. (2003), A framework methodology for performance-based earthquake engineering, 13th world conference on earthquake engineering, proceedings, paper 679.https://www.researchgate.net/publication/228706335.

14. Yang, T.Y., Moehle, J.P., Stodjadinivic, B. and Der Kiureghian. (2006), A. an application of the peer performance-based earthquake engineering methodology, 8th u.s. national conference on earthquake engineering, proceedings, paper 1448. https://www.researchgate.net/publication/264881598.
15. National institute of standards and technology (1999), (Uniformat) II Elemental classification system for building specifications, cost estimating and cost analysis, report No. NISTIR 6389, national institute of standards and technology, Gaithersburg, Maryland.
16. Baker, J., Cremen, G., Giovinazzi, S., and Seville, E. (2016). Benchmarking FEMA P-58 performance predictions against observed earthquake data a preliminary evaluation for the Canterbury earthquake sequence. In 2016 NZSEE conference, Christchurch, New Zealand.https://www.jackwbaker.com/Publications/Baker_et_al_(2016)_P58.
17. Egyptian code for design and construction of reinforced concrete buildings - ECP-203. (2020), Research center for housing and building, Giza, Egypt.
18. Egyptian code for calculation of loads and forces for buildings - ECP-201. (2012), Research center for housing and building, Giza, Egypt.
19. Applied technology council. (2018), FEMA P-58 Next-generation seismic performance assessment for buildings, Vol. 1 – methodology, federal emergency management agency- second edition, Washington, D.C.
20. Applied Technology Council. (2018), FEMA P-58 Next-generation seismic performance assessment for buildings, Vol. 2 – Implementation guide, federal emergency management agency - second edition, Washington, D.C.
21. Federal emergency management agency - FEMA-356. (2000), Prestandard and commentary for the seismic rehabilitation of buildings, Washington, D.C., U.S.A.
22. Badawy A, Korrat I, El-Hadidy M, Gaber H (2016) Probabilistic earthquake hazard analysis for Cairo. Egypt, J Seismol,. https://doi.org/10.1007/s10950-015-9537-5

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