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An Investigation of Seismic Behaviour of Prestressed Concrete Frame Structures using Pushover Analysis

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Abstract: The prestressed concrete system is a prominent material in civil engineering, with applications ranging from buildings, bridges, foundations, piling, silos, stadiums, roadways, and other infrastructure. However, this work is primarily concerned with its applicability to building frame constructions subjected to seismic loading. The prestressed concrete system for frame constructions not only reduces the amount of structural components, but it is also a cost-effective technology that provides outstanding strength and stiffness, as well as quick and simple site erection. Hence, we will be able to use these prestressed concrete components to their full capacity, unlocking engineering and social benefits that were previously impossible. Prestressed concrete designs, as opposed to reinforced concrete designs, can give structures with substantially longer spans, no cracks, or fewer cracks with tiny crack widths. Nonetheless, research on the seismic behaviour of prestressed concrete frame structures is relatively restricted, with the majority of studies focusing on reinforced concrete structures. Finally, the provisional versions of modern building codes such as ACI 318, GB50010, and EC2 do not provide thorough processes for seismic design of prestressed concrete structures. In this work, the seismic design and behaviour of prestressed concrete building frames are explored using the most recent design code. Three model-building codes will be investigated and compared, including ACI 318-14, Eurocode 2004-2, and Chinese GB500010-2010. The 10-story prestressed concrete frame system is subjected to gravity and seismic loading, and the strong-column weak beam mechanism is archived to conform with the newly implemented building design code. To examine the behaviour of prestressed concrete frame structures, nonlinear static pushover analysis is used to capture the reaction under earthquake loading. The stiffness and strength needed will be designed in compliance with the three major building codes indicated. In addition, the finite element model will be created using CSI Sap200 to capture its actions.
Keywords: seismic behaviour, seismic design, prestressed concrete structures, post-tensioned concrete, pushover analysis.

I. INTRODUCTION

Historically, seismic design has been based on the single criterion of ensuring life safety during a design level earthquake. This goal has been found to be most economically accomplished by allowing the structure to respond in-elastically to earthquake loading, which limits the maximum force that any specific structural element will encounter while also lowering the peak response of the structures due to hysteretic energy dissipation. However, due to this design philosophy, most structures are expected to incur deformation after a moderate to severe earthquake; this damage may include structural system stiffness and strength loss, as well as lingering deformations. This may result in costly structural repairs and unfavourable restrictions on building use while those maintenances are performed. Structural engineers are increasingly recognizing that life safety is an important but not always sufficient performance goal. It would be wonderful if a structure could be built cheaply to assure life safety while also remaining uninjured or substantially undamaged in the case of a major earthquake.

II. LITERATURE REVIEW

An investigation of the use of prestressed concrete structures under seismic loads has been carried out by Nakano, Sutherland, and Lin. The findings show that structures made of prestressed concrete are sufficiently resilient to withstand mild to severe seismic excitation[1][2][3]. Inomata examined the behaviour of the twelve reinforced concrete and prestressed concrete beam samples under the scenario of cyclic loading[4]. To increase ductility and lessen the excessive loss of stiffness and strength, the researcher recommended that prestressed concrete members be constructed in compliance with FIP-CEB guidelines. When developed utilizing the recommended method, the prestressed member has significant ductility in comparison to the standard reinforced concrete part.

Hawkins examines the phenomenon of analysis and experimental outcomes of twenty-eight sub-assemblages of precast and prestressed concrete joints. He recommended conducting a systematic test to determine the practical design of prestressed concrete joints. Furthermore, a thorough examination of the factors of durability, anchorage rotation, kind of prestressing steel, and the ultimate strain needed for a prestressing system in a seismic zone is necessary[5].

To establish load-displacement hysteresis curves for fully and partially prestressed concrete joints under severed earthquake force, Thompson and Park conducted cyclic load tests. The test's findings show that prestressed beams' ductility was enhanced when non-prestressed reinforcing was present in the members' compression zones [6]. The load-displacement hysteresis curve was later adjusted by Nishiyama, who also examined the prestressed concrete structure under seismic tension [7]. In Japan, Nishiyama presented the prestressed concrete building seismic design process and contrasted it with traditional reinforced concrete constructions. According to the study, prestressing tendons in column-beam joints enhance the joint's ability to absorb energy and enhance its shear behaviour[8]. Priestley and Tao evaluate the earthquake resistance of precast prestressed concrete frames for tall buildings featuring partially deboned tendons[9]. The shear performance of the joint was found to be enhanced by the suggested approach, which only required nominal stirrups. According to the study, in order to ensure that the plastic hinges on the beam function properly, two percent of specific spiral reinforcement should be positioned in the area at the end of the beam. [9]. For constructions with partially deboned tendons, the ductility requirements would not be higher than those for fully bonded tendons in situations where prestress diminishes due to inelastic strains of the prestressing tendon, as several dynamic inelastic computations show.

An analytical and experimental investigation of the hysteresis restoring force characteristic of unbounded prestressed concrete frame structures subject to earthquake loading has been carried out by Nishiyama, Mugurama, and Watanabe. The analysis comes to the conclusion that unbounded prestressed concrete can be used in the earthquake scenario since the rotation of the beam ends is small, the tendon force is low, and the frame deformed in a significant inelastic range[10]. To evaluate the hysteresis characteristic of prestressed concrete frames subjected to variable loading histories, Su and Zhu developed a new analytical algorithm[11]. A study on the seismic behaviour and design of unbounded post-tensioned precast concrete frames was carried out by Kurama, Pessiki, Sause, and Lu. The study discovered that local ductility demand variables are necessary in order to raise the local ductility needs for the beam-column connections[12]. Due to the small energy dissipation, the maximum displacement of unbounded post-tensioned precast frames under seismic loading is expected to be higher than that of conventional cast-in-place reinforced concrete frames, while the cumulative residual displacement is expected to be much smaller. These local ductility requirement factors are higher for frames intended for high seismicity locations than for frames intended for moderate seismicity regions. By ysing the capacity spectrum method, Nishiyama has undertaken a study to anticipate how prestressed concrete buildings will respond to earthquake loading. The findings have indicated that prestressed concrete has a somewhat greater damping energy in comparison to ordinary reinforced concrete, and that its hysteresis energy is much greater than that of reinforced concrete. As a result, in the case of prestressed concrete, the seismic energy is dispersed kinematically[7].

A. *American Code Seismic Design Criteria*

According to ACI318, there shall be three classes in the design of prestressed concrete members. Class T is the partially cracked member, Class U is the uncracked member, and Class C is the cracked member. Tensile stresses in the concrete should be reduced and the cover elevated in accordance with section 20.6.1.4 of ACI-318 to reduce the likelihood of cracking under service loads. ACI-318 suggests that prestressed concrete members be designed as gross sections for Class U and Class T to meet the serviceability criterion. In the interim, Class C will analyse using the cracked section. As per ACI-318, the following tables shall satisfy the permitted tensile and compressive stress in the prestressed concrete members. *[13].*

TABLE I
PERMISSIBLE TENSILE STRESS OF PRESTRESSED CONCRETE MEMBER[13]

Assume Behaviour	Class	Limits of
Uncracked	U	$f_t \leq 0.62\sqrt{f_c'}$
Transition between uncracked and cracked	T	$0.62\sqrt{f_c'} < f_t < 1.0\sqrt{f_c'}$
Cracked	C	$f_t > 1.0\sqrt{f_c'}$

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TABLE II
PERMISSIBLE COMPRESSIVE STRESS OF PRESTRESSED CONCRETE MEMBER[13]

Location	Concrete compressive stress limits
End of simply supported members	$0.70 f'_{ci}$
All other locations	$0.60 f'_{ci}$

TABLE III
PERMISSIBLE TENSILE STRESS OF PRESTRESSED CONCRETE MEMBER AFTER TRANSFER[13]

Location	Concrete tensile stress limits
End of simply supported members	$0.5\sqrt{f'_{ci}}$
All other locations	$0.25\sqrt{f'_{ci}}$

ACI318 suggested that prestressed concrete member beams with bonded or unboned tendons, the minimum longitudinal reinforcement shall be determined by

$$A_{s,min} = 0.004 A_{ct} \qquad (1\text{-}1)$$

Where A_{ct} is the area of a portion of the cross-section located between the gross section's centroid and the flexural tension face

TABLE IV
MINIMUM SHEAR REINFORCEMENT FOR PRESTRESSED CONCRETE[13]

<table>
<tr><th>Beam Type</th><th colspan="3">$A_{v,min}/s$</th></tr>
<tr><td rowspan="2">Non-prestressed and prestressed with $A_{ps}f_{se} < 0.4(A_{ps}f_{pu} + A_s f_y)$</td><td rowspan="2">Greater of:</td><td colspan="2">$0.062\sqrt{f'_c}\frac{b_w}{f_{yt}}$</td></tr>
<tr><td colspan="2">$0.35\frac{b_w}{f_{yt}}$</td></tr>
<tr><td rowspan="3">Prestressed with $A_{ps}f_{se} \geq 0.4(A_{ps}f_{pu} + A_s f_y)$</td><td rowspan="3">Leaser of</td><td rowspan="2">Greater of:</td><td>$0.062\sqrt{f'_c}\frac{b_w}{f_{yt}}$</td></tr>
<tr><td>$0.35\frac{b_w}{f_{yt}}$</td></tr>
<tr><td colspan="2">$\frac{A_{ps}f_{pu}}{80 f_{yt} d}\sqrt{\frac{d}{b_w}}$</td></tr>
</table>

TABLE V
SUMMARY OF SEISMIC DESIGN CRITERIA FOR CONCRETE FRAME TO AIC 318[13]

Type of Check/Design	Ordinary Moment Frame (Non-seismic)	Intermediate Moment Frames (seismic)	Special Moment Frame (seismic)
Column Design (interaction)	Specified Combination	Specified Combination	Specified Combination
	$1\% < \rho < 8\%$	$1\% < \rho < 8\%$	$1\% < \rho < 8\%$
Column Shears Design	Specified Combination	Modified Combinations (earthquake load is increase by Ω_0)	Specified Combination
	(if SDC = B, and $h/B \le 5$, same as Intermediate	Column Shear Capacity $\phi = 1.0$ and $\alpha = 1.0$	Column Shear Capacity $\phi = 1.0$ and $\alpha = 1.25$ and $V_c = 0$ (conditional)
Beam Design Flexure	Specified Combination	Specified Combination	Specified Combination
	$\rho \le 0.04$	$\rho \le 0.04$	$\rho \le 0.025$
	$\rho \ge \frac{3\sqrt{f'_c}}{f_y}, \rho \ge \frac{200}{f_y}$	$\rho \ge \frac{3\sqrt{f'_c}}{f_y}, \rho \ge \frac{200}{f_y}$	$\rho \ge \frac{3\sqrt{f'_c}}{f_y}, \rho \ge \frac{200}{f_y}$
	or	or	
	$A_{s(min)} \ge \frac{4}{3} A_{s(reqired)}$	$A_{s(min)} \ge \frac{4}{3} A_{s(reqired)}$	
Beam Minimum Moment Override Check	No requirements	$M^+_{u,end} \le \frac{1}{3} M^-_{u,end}$	$M^+_{u,end} \le \frac{1}{2} M^-_{u,end}$
		$M^+_{u,span} \le \frac{1}{5} max\{M^+_u, M^-_u\}_{end}$	$M^+_{u,span} \le \frac{1}{4} max\{M^+_u, M^-_u\}_{end}$
		$M^-_{u,span} \le \frac{1}{5} max\{M^+_u, M^-_u\}_{max}$	$M^-_{u,span} \le \frac{1}{4} max\{M^+_u, M^-_u\}_{max}$
Beam Shear design	Specified Combination	Modified Combinations (earthquake load doubled)	Specified Combination
		Beam Shear Capacity (V_c) with $\phi = 1.0$ and $\alpha = 1.0$ plus V_{D+L}	Beam Shear Capacity (V_c) with $\phi = 1.0$ and $\alpha = 1.25$ plus V_{D+L}
			$V_c = 0$ (conditional)
	$\phi = \phi_s$ (default 0.75)	$\phi = \phi_s$ (default 0.75)	$\phi = \phi_s$ (or $\phi_{s,seismic}$)
Joint Design	No requirement	No requirement	Shear Check
Strong Column Weak Beam mechanism check	No requirement	No requirement	Checked

B. European Code Seismic Design Criteria

In Eurocode 2, there are stress thresholds that indicate crack control and are the same for bounded and unbonded systems [14].

1) For frequent load condition:

Concrete compression	$= 0.60f_{ck}$
Concrete tension	$= 0.30f_{ck}^{2/3}$
Mild steel Tension	$= 0.80f_{yk}$
Prestressing Steel Tension	$= 0.75f_{pk}$

2) For quasi-permanent load condition

Concrete compression = $0.45f_{ck}$

Concrete tension = $0.30f_{ck}^{2/3}$

Eurocode 2 allows the permitted hypothetical tensile stress to be exceeded in concrete, provided that cracking is controlled to remain within the selected design crack width.

3) For initial load condition:

Tensile stress for bonded and unbonded = $0.30f_{ci}^{2/3}$

Compressive stress = $0.60f_{ci}$

Eurocode 2 suggested that the minimum area of mild steel shall be calculate by

$$0.0013\, b_t d \geq A_{smin} \geq \frac{0.26 f_{ctm} b_t d}{f_{yk}} \tag{1-2}$$

TABLE VII

SUMMARY OF SEISMIC DESIGN CRITERIA FOR CONCRETE FRAME TO EC2[14]

Type of Check/Design	Ductility Class Low MRF	Ductility Class Medium MRF	Ductility Class High MRF
Column Design (interaction)	Specified Combinations	Specified Combinations	Specified Combinations
	$0.2\% < \rho < 4\%$	$1\% < \rho < 4\%$	$1\% < \rho < 4\%$
Column Shears Design	Specified Combinations	Specified Combinations	Specified Combinations
		Column Shear Capacity $\gamma_{rd} = 1.1$	Column Shear Capacity $\gamma_{rd} = 1.3$
Beam Flexural Design	Specified Combinations	Specified Combinations	Specified Combinations
	$\rho_{min} \geq \begin{cases} 0.26\left(\frac{f_{ctm}}{f_{yk}}\right) \\ 0.0013 \end{cases}$	$\rho_{min} \geq 0.5\left(\frac{f_{ctm}}{f_{yk}}\right)$	$\rho_{min} \geq 0.5\left(\frac{f_{ctm}}{f_{yk}}\right)$
	$\rho_{min} \leq 0.004$		
Beam Minimum Moment Override Check	No requirement	$M^+_{u,end} \geq \frac{1}{2} M^-_{u,end}$	$M^+_{u,end} \geq \frac{1}{2} M^-_{u,end}$
		$M^-_{span} \geq \frac{1}{2} max\{M^+, M^-\}_{max}$	$M^+_{span} \geq \frac{1}{4} max\{M^+, M^-\}_{end}$
			$M^-_{span} \geq \frac{1}{2} max\{M^+, M^-\}_{end}$
Beam Shear Design	Specified Combinations	Specified Combinations	Specified Combinations
		Beam Shear Capacity (V_e)	Beam Shear Capacity (V_e)
		$\gamma_{Rd} = 1.0$	$\gamma_{Rd} = 1.2$
Joint Design	No requirement	No requirement	Check for Shear
Strong Column Weak Beam Check	No requirement	No requirement	Checked

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C. Chinese Code Seismic Design Criteria

Table VII show the limits height for prestressed concrete member design to the Chinese code.

TABLE VII
LIMITS HEIGHT FOR BUILDING STRUCTURES AS PER JGJ140-2004 [15]

Structural System	Earthquake Intensity		
	6	7	8
Frame Structure	60	55	45
Frame-Shear wall	130	120	100
Partially Frame Shear Wall	120	100	80
Fame-Tube Core	150	130	100
Slab Column-Shear Wall	40	35	30
Slab Column-Frame Structure	22	18	

A prestressed concrete frame shall be developed as a ductile structure that has considerable deformation capacity and seismic energy dissipation capacity, according to Chinese code JGJ-140-200. Before bending, its structural member must prevent shear failure, and its joints must not fail before its connecting members [15]. The highest point at which prestressed concrete structures with seismic design can be built in accordance with JGJ 140, section 3.2.1. For structures with uneven plane and vertical geometry, structures constructed on Class IV sites, or structures with extensive spans, the applicable maximum height must be suitably lowered. [15].

JGJ required the pressed concrete beam members width of the section shall be more than 250mm, height to width ratio shall be less than 4 (h/b < 4), and beam height shall be between (1/12 - 1/22) of the span. The minimum reinforcement for non-prestressing steel shall be 2.5 percent for steel grade HRB400 [15]. Shear capacity shall be determined as the following

$$V_j \le \frac{1}{\gamma_{RE}}(0.30\beta_c \eta_j f_c b_j h_j) \quad (1\text{-}3)$$

Where

V_j = design shear force in joint

β_c = concrete strength coefficient

η_j = restrain coefficient

b_j = effective width

h_j = effective height

γ_{RE} = seismic bearing capacity

III. STRUCTURAL CONFIGURATION AND ANALYSIS

A. Structural Configuration

Figures 1 and 2 show the planned layout of a prestressed concrete frame construction. Three bays for longitudinal and transverse orientations are found in the prototype prestressed concrete structure that was chosen for this investigation. Three by eight meters in the x direction and three by seven meters in the y direction make up the frame's grid arrangement. While the average story height is 3.5 meters, the first storey is 4 meters tall. The structure is elevated 35.5 meters in total. The seismic behaviour of 2D prestressed concrete frame constructions in grid 2 is examined, as seen in Figure 1..

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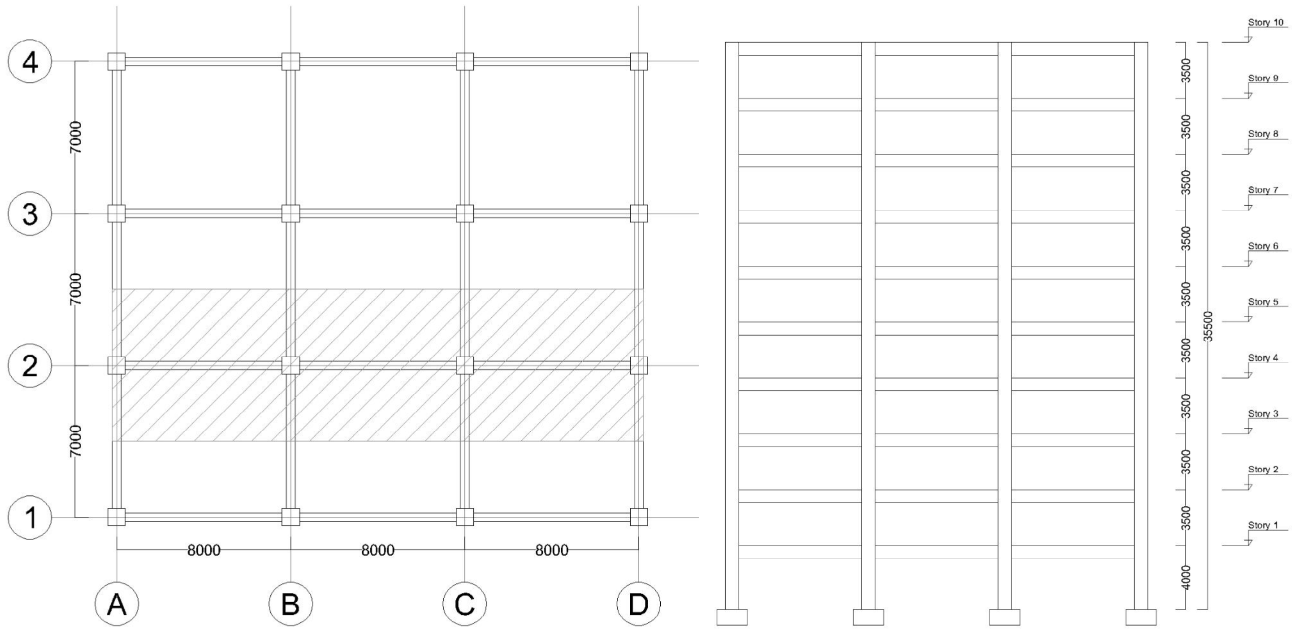

Fig. 1 Structural Configuration of Prestressed Concrete Frame

The prestressed and reinforced concrete frame structures has the same section properties for column. Column C1 is used from story 1 to story 4, column C2 is used from story 5 to story 7, and the column C3 is used for story 8 to story 10. The RC and PT beam are shown in the below Figure.

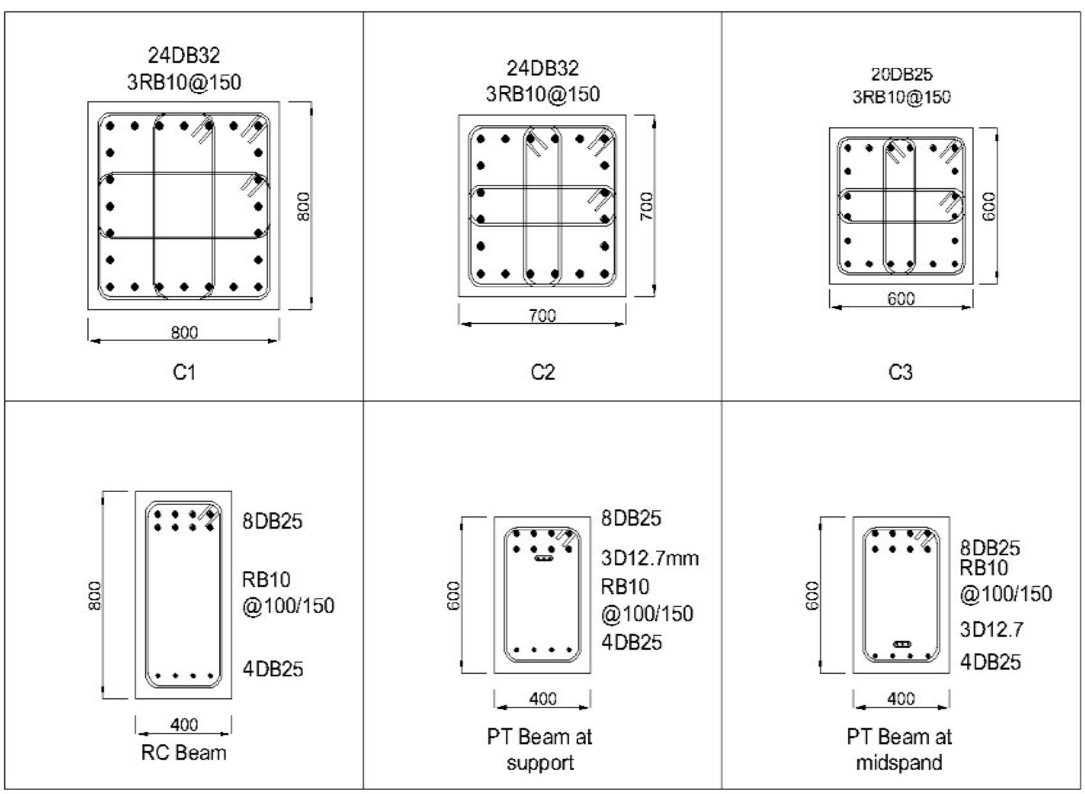

Fig. 2 Section Property

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B. *Material Data*

Material use for structural analysis and design of prestressed concrete frame is selected based on each design code.

TABLE VIII
Material Data

	American standard	European standard	Chinese standard
Concrete Grade	5000psi	C35/45	C55
Compressive Strength (MPa)	35	35	35.5
Concrete density (kN/m^3)	23.5	25	25
Concrete Modulus of Elasticity (MPa)	28000	34000	35500
Shear Modulus (MPa)	11579	14166	14791
Rebar Grade	A615Gr60	S400	HRB400
Yield strength (MPa)	414	400	400
Rebar Density (kN/m^3)	78.5	78.5	78.5
Tendon Grade	A416Gr270	Grade 1637	GBfpk1860
Tensile strength (MPa)	1860	1860	1860
Tendon density	78.5	78.5	78.5
Tendon Modulus of Elasticity (GPa)	193	196.5	195

C. *Loading Input*

The live and dead loads for the proposed construction, which is intended to be a residential building, must comply with ASCE 7, Eurocode 1, and GB 50009 codes. According to ASCE-7, a live load of 1.92 kPa is chosen, and correspondingly, 2 kPa is chosen for both the Chinese and Eurocode [16] [17][18]. So the gravity loading apply on the beam for 7meters bay width is calculate as below.

Table IX
Gravity Loading

	American Code	European Code	Chinese Code
Live load (kN/m)	13.44	14	14
Slab self-weight (kN/m)	24.675	26.25	26.25
Super imposed dead load (kN/m)	14	14	14

Seismic loading parameter is selected based on the study of conversion relationships among the parameters of seism design code by Luo and Wang [19]. For this study, site class C is selected for ASCE-7, where Ss and S1 is given the San Francisco mapped acceleration, and site class B in Eurocode 8, and lastly site class II in Chinese code accordingly.

For ASCE-7, the seismic parameter for short period spectral acceleration, Ss = 1.5g, 1-second spectral acceleration, S_1 = 0.568g, long period transition period, T_L = 8s, site acceleration coefficient, F_a = 1.2, site velocity coefficient, F_v = 1.432, $S_{DS} = (2/3)F_aS_s$ = 1.2, $S_{D1} = (2/3)F_vS_1$ = 0.5423, response modification factor R = 8, over strength factor taken as 3, deflection amplification factor C_d = 5.5, and seismic important factor 1.0.

For Eurocode 8, the seismic coefficient for ground acceleration, a_g/g = 0.3g, spectrum type 1, soil Factor, S = 1.2, spectrum period, T_b = 0.15, spectrum Period, T_d = 2, lower bound factor, β = 0.2, behaviour factor, q = 5.85, correction Factor, λ=1.

For Chinese GB50011, the seismic coefficient for max influence factor α_{max} = 0.16, seismic intensity 8, characteristic period T_g = 0.77, period time discount factor 1, enhancement factor 2.2.

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D. Structural Analysis Method

An approach used to study the seismic behaviour of prestressed concrete frame structures is nonlinear static pushover analysis. One of the most useful seismic tools in civil engineering is the use of a nonlinear static process to forecast seismic demands and the horizontal capacity of structures. The push over method is becoming more and more popular for nonlinear structural analysis due to its ease of use and quick turnaround time. It could be done in a way that is controlled by force or displacement. The first approach, in which the amount of the applied load is unknown, is used when the structure weakens or specific drifts are examined. However, the second one is used when the load is known and it is expected that the structure will support the load.

In order to evaluate the seismic performance and behaviour of prestressed concrete frame structures, we will use the ATC-40 capacity spectrum approach in this study. Plastic hinge properties and acceptability standards are predicated on the ATC-40 clause. The intersection of the capacity curve and the spectrum response curve is the performance point in the capacity spectrum method. A representation of the performance point is shown in Figure 3. Owing to the elements' morphological changes once the plastic junction occurs, details about the lifespan of the building structure and its effective structural stiffness can be obtained at this point. This information can be used to determine other structural reactions, such as the degree of deviation and the placement of plastic hinges.

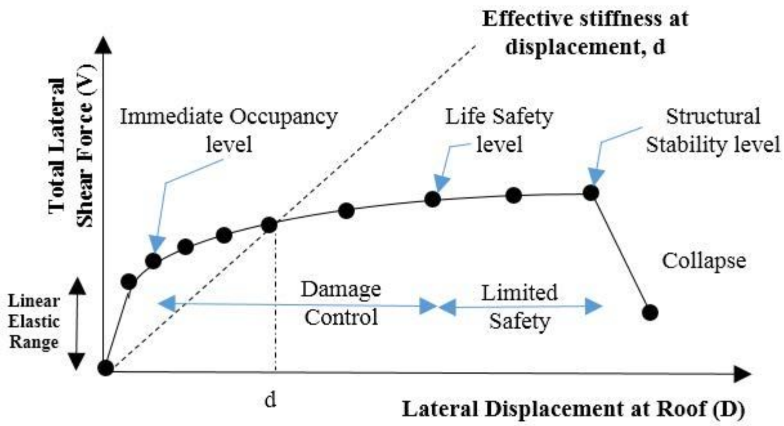

Fig. 3 Performance point curve[20]

A structure classified as immediate occupancy (IO) can withstand an earthquake without experiencing any damage, either structural or non-structural. After an earthquake, structures in this category of IO can resume operations. Buildings that are inhabited or contain people are safe from structural damage during an earthquake since the structure can withstand the shaking. We call this life safety (LS). Despite suffering severe structural damage, a structure with collapse prevention (CP) did not collapse after an earthquake. Structural stability (SS) is the result of a reduction in the strength and stiffness of the lateral force retention system following partial or total damage to a structure [19].

TABLE X
LIMITATION OF ROOF DRIFT RATIO ACCORDING TO ATC-40 [21]

Parameter	Performance Level			
	IO	DC	LS	SS
Maximum Total Drift	0.01	0.01-0.02	0.02	0.33Vi/Pi
Maximum Inelastic Total Drift	0.005	0.005-0.015	No limit	No limit

IV. RESULT AND DISCUSSIONS

The section illustrates how prestressed concrete frame buildings built in accordance with Chinese, European, and American norms behave seismically. The following figures show story displacement, inter-story drift, pushover capacity curve, and plastic hinge mechanism. They are based on the provisions and acceptance criteria that were established in ATC-40 together with the relevant code.

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The story displacement of prestressed and reinforced concrete frame structures developed in accordance with American code ACI-318 is depicted in Figure 4(a). Prestressed concrete frames have a top story displacement of 76.77mm, while reinforced concrete frames have a top story displacement of 60.75mm. The prestressed concrete frame is displaced by roughly 18% more than the reinforced concrete frame, according to these measurements. This could be because the reinforced concrete beam is 400x800mm, whereas the reduced beam section is 400x600mm. While there are no restrictions on seismic story displacement, ASCE-7 advised a top story displacement limit of approximately H/400 to H/600 for wind loading. Assuming we use the standard practice limit of H/200 for seismic drift, the upper story displacement limit is 177.5 mm > 76.77 mm. In addition to being developed with smaller beam sections than traditional reinforced concrete, the prestressed concrete frame complies with all requirements specified by the relevant design code.

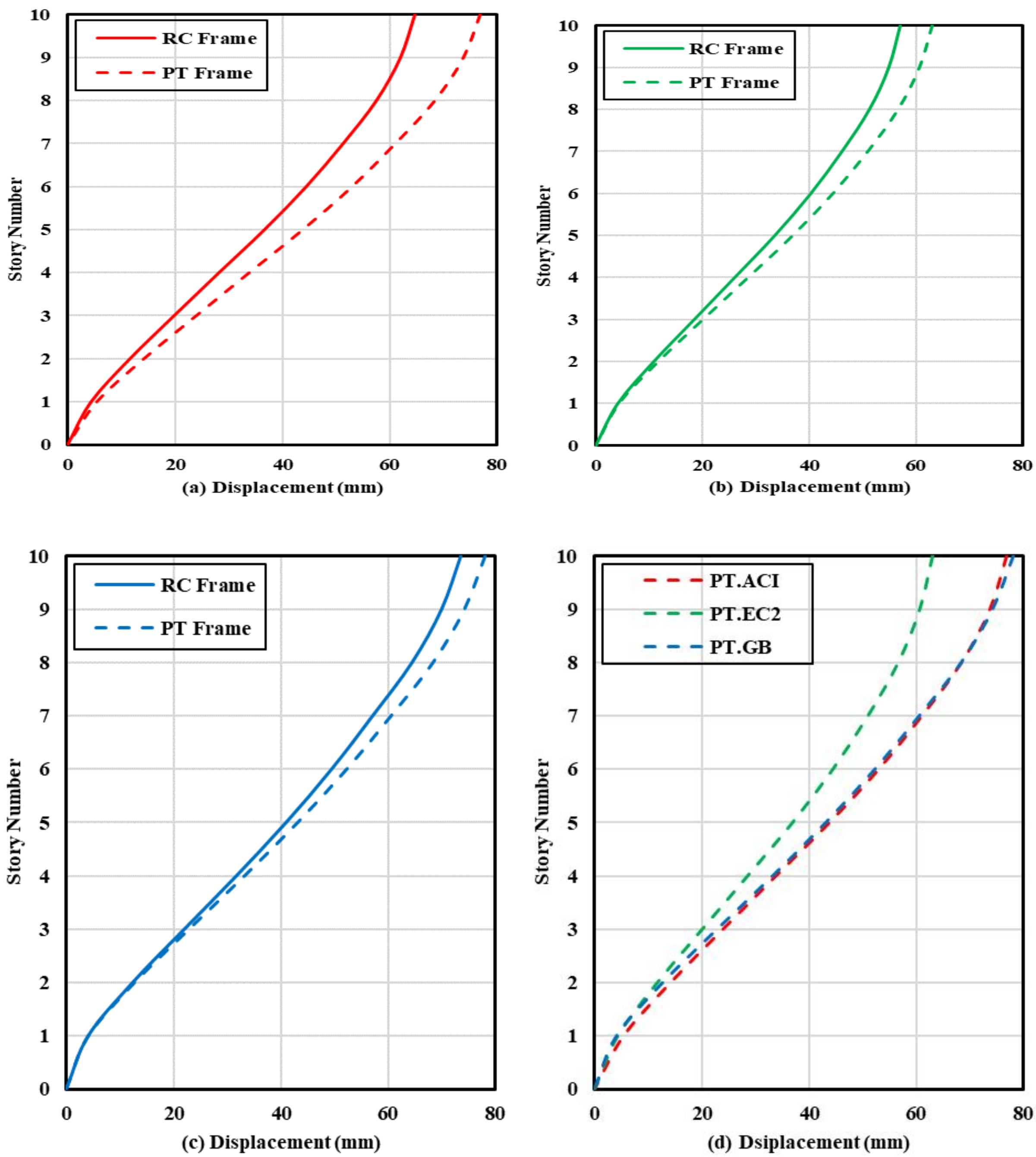

Fig. 4 Top story displacement (a) RC and PT Frame design to ACI318. (b) RC and PT Frame design to EC2. (c) RC and PT Frame design to GB50010. (d) PT Frames.

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Reinforced and prestressed concrete frames for the top story replacement measure 57.14 mm and 62.95 mm, respectively. Because smaller beam sections are constructed into prestressed concrete frame structures, a prestressed concrete frame has a 10% higher displacement than a reinforced concrete frame. Nonetheless, the frame made of prestressed concrete remains within the permissible limits as stipulated by Eurocode 2 as shown in Figure 4(b).

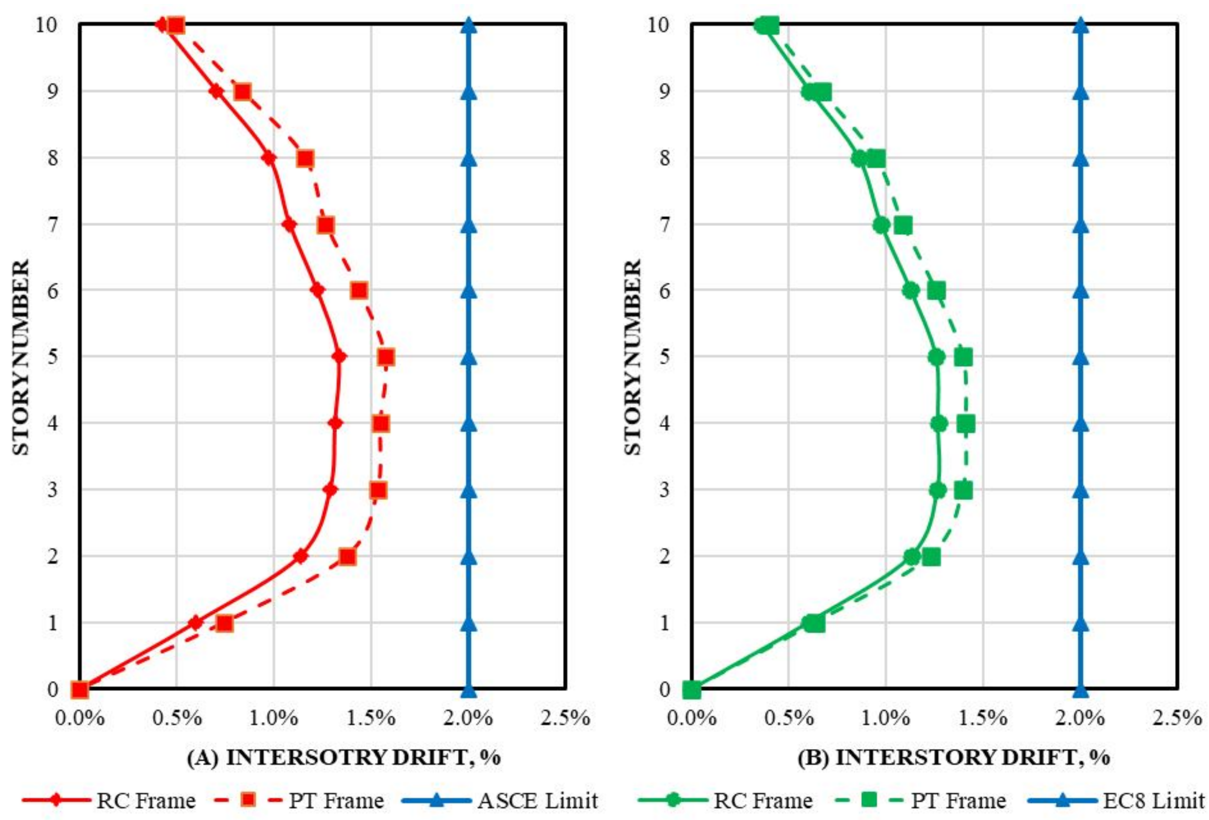
The story displacement for prestressed and reinforced concrete frame structures developed in accordance with the Chinese code GB50010-2010 is depicted in Figure 4(c). Both constructions have about the same storey displacement. The top displacement of the reinforced frame is 73.52mm, whereas the prestressed concrete frame provides a displacement of 78mm. Precast and reinforced concrete frames are both permitted within the bounds set forth in GB50010-2010.

The top story displacement between prestressed concrete frames developed in accordance with GB50010-2010, Eurocode 2, and ACI-318 is depicted in Figure 4(d). The displacement of the prestressed concrete frame developed in accordance with ACI-318 and GB50010-2010 is nearly the same; both are roughly 20% greater than that of Eurocode 2. The material qualities and safety considerations listed in the relevant code are to blame for this outcome.

The inter-story drift of reinforced and prestressed concrete frame structures built in accordance with American code ACI-318 is depicted in Figure 5(a). As recommended by ASCE-7 provisioned, the aforementioned equation (3.10) is used to determine the inelastic drift. The reinforced concrete frame structure has a story drift of roughly 1.3%, while the largest story drift of the prestressed concrete frame structure is approximately 1.6% at story 5. Both of the reinforced concrete frame constructions meet the ASCE-7's acceptable standards.

The inter-story drift of the prestressed concrete frame structure, developed in accordance with the following Eurocode 2 regulation, is depicted in Figure 5(b). Table 13.4 is used to determine the inelastic drift. A 1.4% story drift is produced by the prestressed concrete frame and a 1.3% drift by the reinforced concrete frame. Regarding seismic drift, both systems are similar and meet Eurocode 8's acceptance requirements.

The inter-story drift of prestressed and reinforced concrete frame structures designed in accordance with Chinese code GB50010-2010 is depicted in Figure 5(c). The graphs demonstrate that while the drifts in the prestressed and reinforced concrete frame structures are nearly comparable, the prestressed concrete frame structure's drift is still more than the reinforced concrete's. Nevertheless, neither structure exceeds the GB50011 limit.



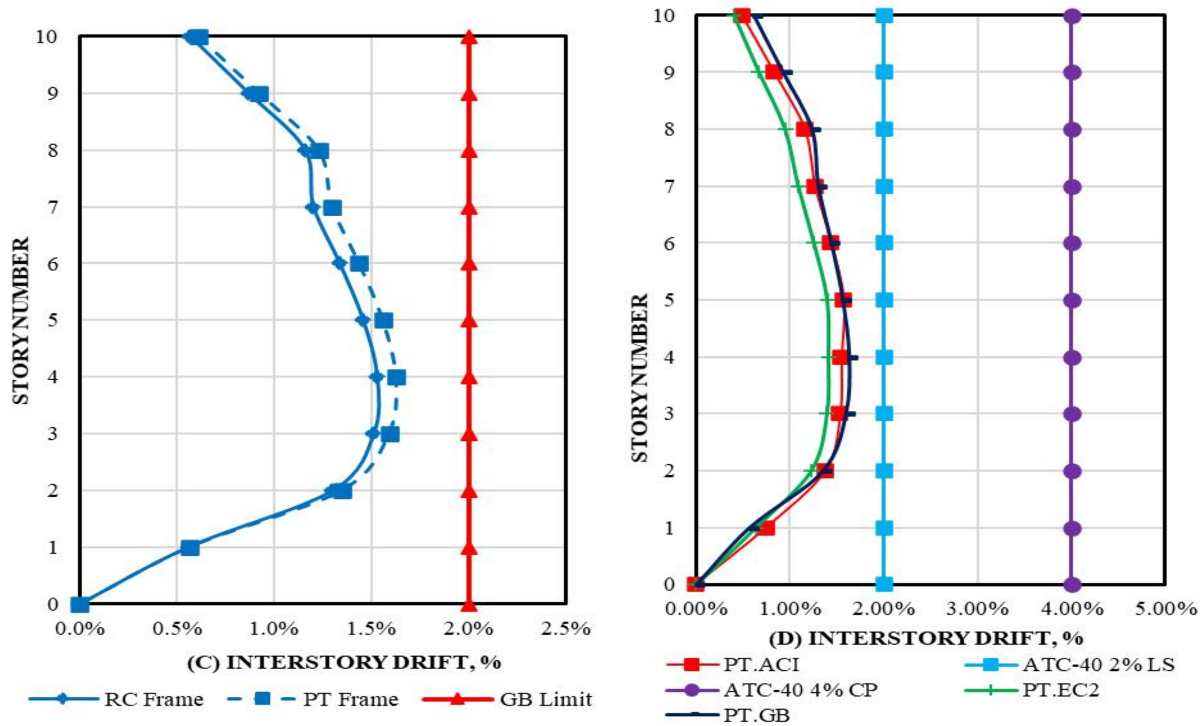
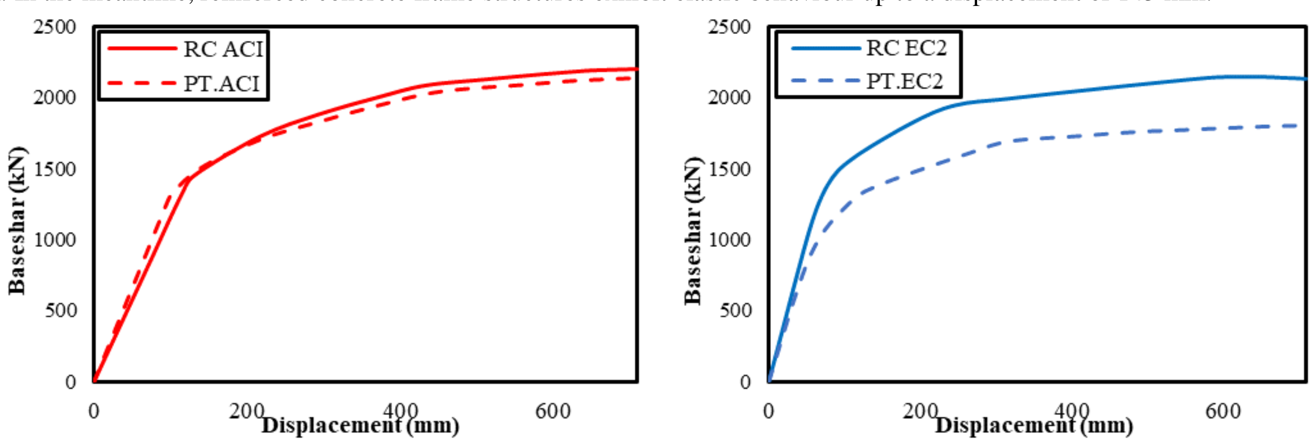


Fig. 5 Inter-story drift (a) RC and PT Frame design to ACI318. (b) RC and PT Frame design to EC2. (c) RC and PT Frame design to GB50010. (d) PT Frames

The inter-story drift of the prestressed concrete frame structure developed following the Chinese, European, and American codes to the standard of ATC-40 approval criteria is depicted in Figure 5(d). The results demonstrate that every frame created in accordance with its corresponding code satisfies the ATC-40 restriction. There are no LS or CP performance level reaches that demonstrate the prestressed concrete frame structures' exceptional earthquake performance.

E. Pushover Curve

The resulting pushover curve of prestressed and reinforced concrete structures is depicted in the pictures below. Since we used the ATC-40 capacity spectrum approach, the benchmark monitor displacement for both systems is the recommended 2% roof drift target displacement. The monitored displacement, then, is 2% x 35.5m = 710mm. The pushover curve illustrates the state of the structure with a fathered increase in displacement in the elastic and plastic range. The pushover capacity of prestressed and reinforced concrete frame structures designed in accordance with GB50010, EC2, and ACI-318 is depicted in Figure 6. Evidence indicates that the prestressed concrete frame remains elastic until the displacement reaches 115 mm, even after it has shifted by 94 mm. In the meantime, reinforced concrete frame structures exhibit elastic behaviour up to a displacement of 143 mm.



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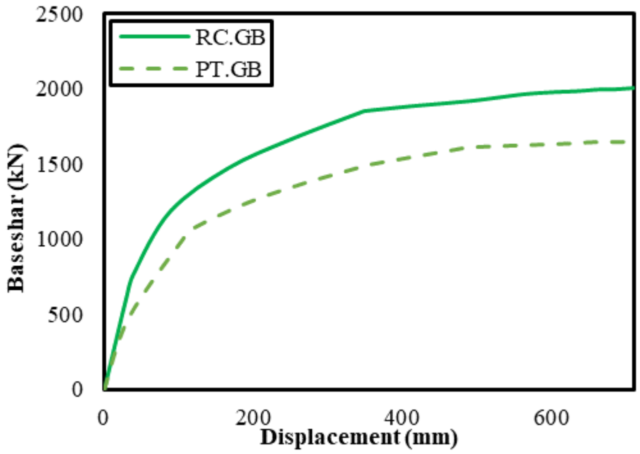

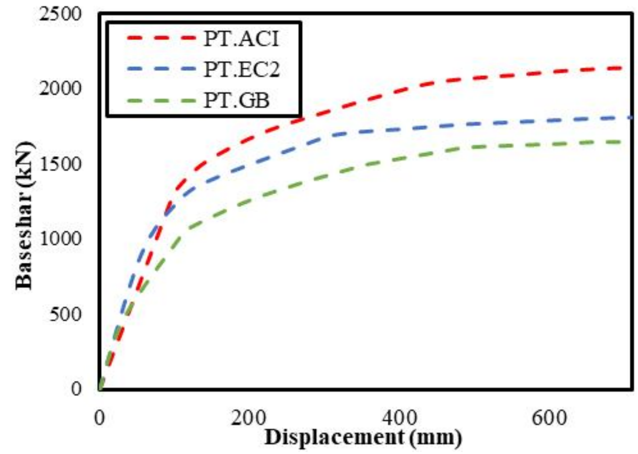

Fig. 6 Pushover curve of prestressed and reinforced concrete frame structure design to ACI-318, EC-2, GB50010

The pushover capacity of reinforced and prestressed concrete frame structures designed in accordance with Eurocode 2 demonstrates that the prestressed concrete frame remains elastic until the displacement reaches 105 mm, at which point it ceases to be elastic. In the meantime, reinforced concrete frame structures exhibit elastic behaviour up to a displacement of 110 mm. According to the Chinese code GB50010-2010, the prestressed concrete frame remains in the elastic range until the displacement reaches 80mm, and it continues to behave elastically until the displacement reaches 117mm, as demonstrated by the pushover curve of reinforced concrete frame constructions. In the meantime, reinforced concrete frame structures exhibit elastic behaviour up to a 90 mm displacement.
The results of the study demonstrate that any prestressed concrete frame built in accordance with its specific regulation has enough seismic action capacity. Their seismic performance level is very high when compared to the ATC-40 standard. While using a smaller section than reinforced concrete buildings, prestressed concrete frames nonetheless meet acceptance criteria even if they have a bigger capacity than traditional reinforced concrete frames.

TABLE XI
PERFORMANCE POINTS OF PRESTRESSED CONCRETE FRAMES

Target Variable	ACI-318	Eurocode 2	GB50010
V (kN), D(mm)	1587.73, 171	1433.489, 170.276	1241.433, 193.182
S_a (g) , S_d (g)	0.145, 0.137	0.135, 0.130	0.119, 0.147
T_{eff} , B_{eff}	1.933, 0.159	1.954, 0.187	2.222, 0.193

Based on the capacity spectrum in ATC-40, Table 10 displays the prestressed concrete frame design's performance point in relation to the applicable codes. The outcome demonstrates the sufficient capability of the prestressed concrete frame. Their seismic performance level is very high when compared to the ATC-40 standard. While using a smaller section than reinforced concrete buildings, prestressed concrete frames nonetheless meet acceptance criteria even if they have a bigger capacity than traditional reinforced concrete frames.

F. Plastic Hinge Formation

Figures 7 through 9 show the stages and places where the plastic hinges for instant occupancy (IO), life safety (LS), and collapse prevention (CP) rotate in the beams and columns. The last push of frame buildings to the 2% roof drift recommended by ATC-40 is depicted in every illustration. In accordance with applicable standards, plastic hinges were typically produced in the lower section of all reinforced and prestressed concrete frame systems. Furthermore, when a consistent lateral stress pattern is given to the constructions, plastic hinges have a tendency to form from the base column.

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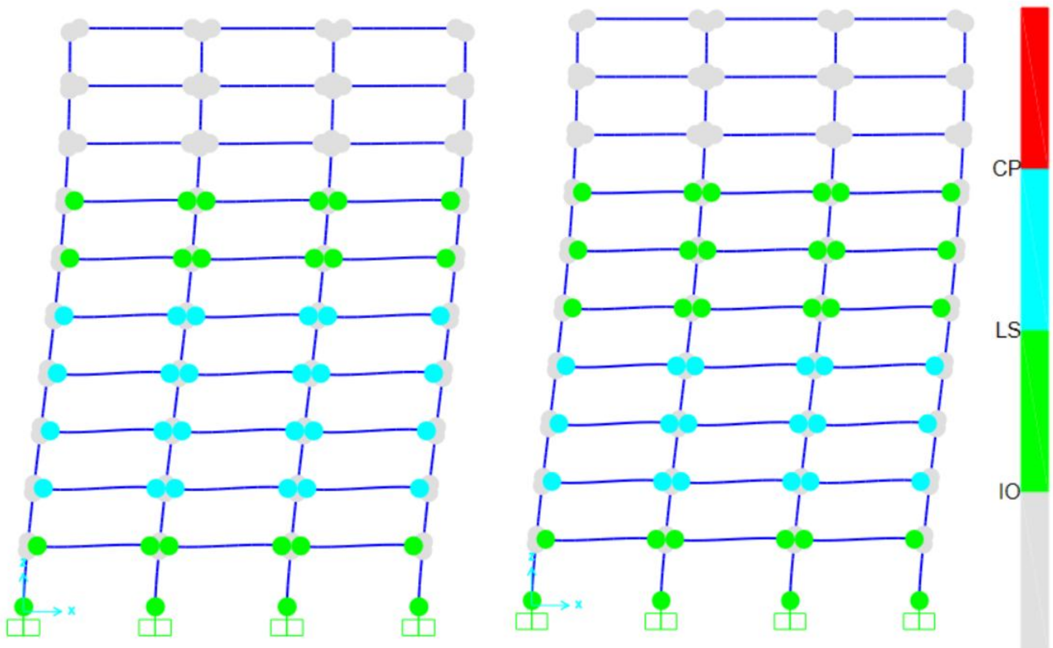
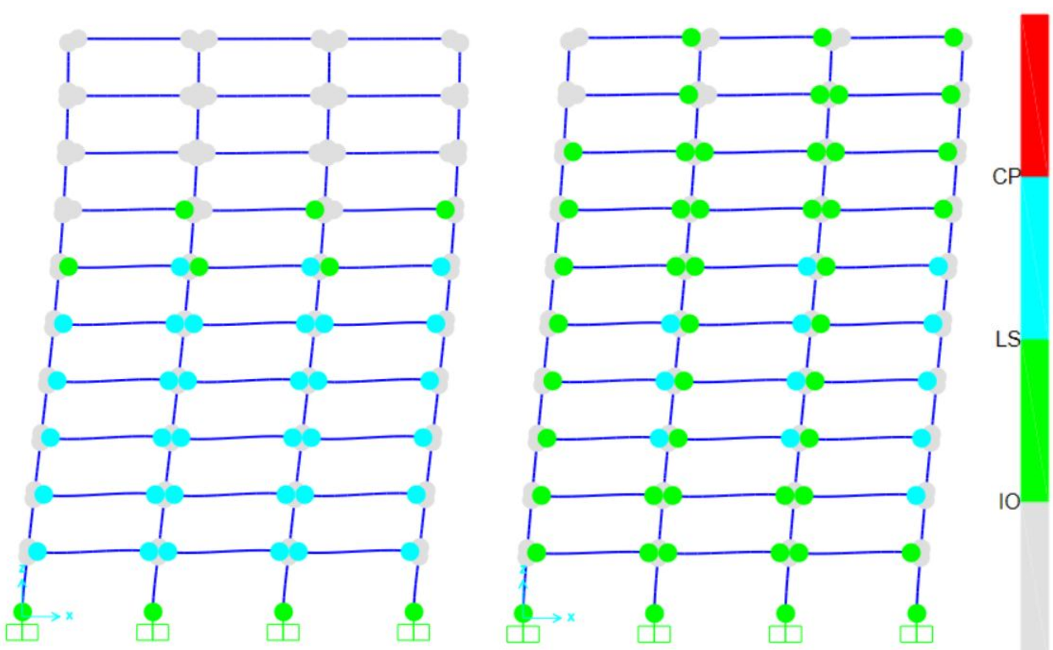
At the last stage of 2% roof drift, Figure 7 depicts the plastic hinge creation in prestressed and reinforced concrete frame structures designed in accordance with ACI-318. While the foundation stories of both frame systems have (IO) hinge rotation of the columns, the prestressed concrete frame has more (LS) hinge rotation of the beams from story 2 to story 5 than the reinforced concrete frame can boast. Nonetheless, because there is no (CP) hinge rotation form at the structure when pushing to monitor displacement, both prestressed and reinforced concrete frame designs in accordance with ACI-318 exhibit outstanding performance. When the displacement reaches 91 mm, the (IO) hinge rotation form in the prestressed concrete beam at joint 30, story 3. When the displacement reaches 204mm the same joint, the (LS) hinge rotation forms.



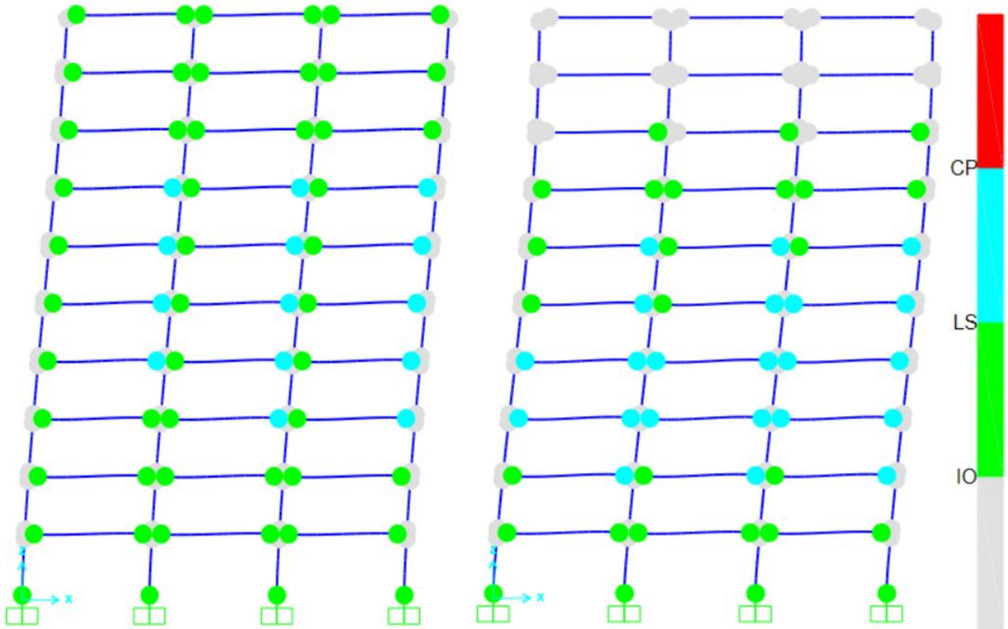
(a) Prestressed Concrete Frame ACI (b) Reinforced concrete Frame ACI

Fig. 7 Plastic hinge formation.



(a) Prestressed Concrete Frame EC2 (b) Reinforced concrete Frame EC2

Fig. 8 Plastic hinge formation



(a) Prestressed Concrete Frame GB50010 (b) Reinforced concrete Frame GB50010

Fig. 9 Plastic hinge formation

Figure 8 illustrates how prestressed and reinforced concrete frame constructions designed in accordance with Eurocode 2 rotate on plastic hinges. When the last push reach is displaced, both frame systems have (IO) hinge rotation in the columns at the base story; no (CP) hinge rotation form exists. Nonetheless, compared to reinforced concrete frame buildings, prestressed concrete frame structures have higher (LS) hinge rotation. Since there is no (CP) hinge rotation form in the columns or beams, both prestressed and reinforced concrete frames have demonstrated good performance levels.

The plastic hinge rotation of prestressed and reinforced concrete frame constructions designed in accordance with Chinese code GB50010-2010 is depicted in Figure 9. It is evident that prestressed concrete frame structures exhibit (LS) hinge rotation in the top part of the structure when it reaches the monitor displacement, but reinforced concrete frame constructions exhibit greater (LS) hinge rotation forms in their beams. As per the aforementioned pattern, neither the beams nor the columns for either system have any (CP) hinge rotation form.

V. CONCLUSION

The American, European, and Chinese codes for prestressed concrete frame structure design and their seismic behavior are the main topics of this study. Using the finite element program CSI Sap2000, the nonlinear static pushover process recommended by ATC-40, which is based on the capacity spectrum method, is used to analyze the seismic performance of the structures. The primary deductions drawn from the analytical outcome are:

1) The structure of the prestressed concrete frames is strong enough to withstand seismic activity while yet adhering to the damage restriction guidelines specified by the relevant design code.
2) The highest capacity is found in prestressed concrete frame designs according to ACI-318, which is followed by those according to Eurocode 2 and Chinese code GB50010, in that order. This is because each applicable code has a slight variation in the material attribute, safety factor, and loading assumption.
3) Though conventional reinforced concrete has a larger capacity, precast concrete frame structures maintain design satisfaction with smaller dimension properties.
4) The strong column weak beam mechanism is archived in the prestressed concrete frame structures when pushing to monitored displacement.
5) The 3 major design codes has a similar acceptance criteria to ATC-40 benchmark.

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