

Height-Dependent Seismic Performance of Ordinary Moment Frame Buildings with Solid Slabs on Dropped Beams in Tripoli, Libya: A Nonlinear Pushover Analysis

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Abstract

Reinforced concrete (RC) buildings with ordinary moment frames (OMFs) and solid slabs on dropped beams represent the predominant mid-rise residential structural system in Tripoli, Libya. These buildings are typically designed for gravity loads alone, with little or no explicit seismic detailing. This study investigates how building height affects the seismic response, damage evolution, and performance compliance of this typology. Three configurations — 8, 10, and 12 stories — share identical plan geometry, material properties, and site hazard parameters. Each configuration was modeled in ETABS and analyzed using nonlinear static pushover analysis, with concentrated plastic hinges assigned at the ends of beams and columns. Performance was evaluated against ASCE 41-17 acceptance criteria at the target displacement corresponding to the unreduced MCE_R demand.

The findings showed that the three configurations satisfied the Immediate Occupancy performance level in both principal directions. Increasing height reduced lateral stiffness, lengthened the fundamental period, and progressively degraded displacement ductility along the stronger direction, highlighting the need for explicit seismic detailing as building height increases.

Keywords: *Pushover Analysis, Ordinary Moment Frame (OMF), Building Height, Performance-Based Earthquake Engineering, Target Displacement, Inter-Story Drift, Tripoli.*

1. Introduction

Reinforced concrete (RC) moment-resisting frames (MRFs) are among the most widely used structural systems in seismic regions, owing to their spatial flexibility, functional adaptability, and economy. In many developing urban centers, they serve as the sole lateral force-resisting system (LFRS), combining open floor plans with the ability to resist lateral load through beam-column action. Their seismic performance depends on lateral stiffness, deformation capacity, energy dissipation, and how uniformly inelastic demand distributes across the elevation as the structure grows taller. Performance-Based Earthquake Engineering (PBEE) provides the analytical tools to quantify these characteristics directly, tracing a building's

behavior from first yield through progressive damage to collapse, rather than relying on code-level strength checks alone.

Libya lies near the boundary between the African and Eurasian tectonic plates, with seismicity concentrated along its northern coastal strip. Historical seismicity confirms a moderate regional hazard: The 1963 Al Marj earthquake ($M_b > 5.6$) and the 1977 Hun Graben earthquake ($M_w > 5.9\text{--}6.0$) seriously damaged masonry and poorly reinforced concrete buildings [1]. Offshore earthquakes near Tripoli occurred in 1974 and 1976, and tremors of magnitude 4–5.6 were recorded across northern Libya between 1990 and 2019 [2]. Recent probabilistic seismic hazard assessments (PSHA) for Libya show non-negligible hazard along the coast, particularly in Tripoli

[2], with the northeast generally more hazardous than the northwest, where Tripoli is located [3], [4]. Earlier regional studies relied on coarser source models and tended to underestimate hazard on the Jifarah Plains near Tripoli [3]–[5].

Design practice in Libya, however, has long assumed negligible seismic hazard, relying on the outdated Draft Suggested Libyan Standard (DSLS-1977), which remains the only codified reference and is widely regarded as outdated and inconsistently enforced [6]. Ngab and Tajouri [7] addressed this gap with updated, site-specific response spectra calibrated to regional fault sources and historical seismicity, along with related hazard mapping [7]. These parameters place Tripoli in Seismic Design Category (SDC) C or D, depending on soil conditions and occupancy. This study adopts the response spectrum of Ngab and Tajouri [7] as the most current characterization available for the city.

Despite these updated hazard maps, standard construction practice in Tripoli still relies on Ordinary Moment Frames (OMFs) with solid slabs cast monolithically on dropped beams. ACI 318-19 [8] and ASCE 7-16 [9] classify RC moment frames into three ductility categories — ordinary (OMF), intermediate (IMF), and special (SMF) — with OMFs the least ductile of the three. OMFs carry minimal seismic detailing requirements — wide stirrup spacing, no seismic hooks, and no strong-column/weak-beam check, in contrast to the close hoop spacing and capacity-design provisions required for IMF and SMF systems — along with a low response modification factor ($R = 3$). Accordingly, ASCE 7-16 restricts OMFs to Seismic Design Categories A and B, where seismic demand is low, and excludes their use in higher categories (SDC C through F) [9].

In Tripoli, however, mid-rise buildings of 8 to 12 stories are routinely sized for gravity combinations alone ($1.4D$ and $1.2D + 1.6L$), with lateral wind or seismic demand omitted from initial member sizing [10]. This practice developed on the assumption that regional seismic forces are small enough to ignore — an assumption that has never been reconciled with the design categories

established by recent hazard studies. The result is a clear mismatch between as-built vulnerability and current seismic hazard expectations.

Building height is a governing parameter for nonlinear seismic performance, and it makes these detailing concerns more severe. Taller buildings carry more mass, have longer fundamental periods, and are more sensitive to P- Δ effects — changes that alter how inelastic deformation distributes through the structure. A detailing deficiency that matters little in a short OMF building may therefore govern the response of a taller one. Studies of non-ductile RC frames show that taller OMF buildings are prone to premature collapse mechanisms, especially where member stiffness or mass changes abruptly up the height [11], [12]; failure is often tied to column moment-strength discontinuities between adjacent stories, a mechanism that becomes more pronounced as story count increases. Incremental dynamic analyses and pushover studies show that although flexible, taller frames may attract lower spectral accelerations, their larger deformation demands can still exhaust local ductility capacity, producing soft stories and concentrated drift [13], [14]. Comparative pushover studies further find that the gap between seismic demand and available ductility widens with height, driven by longer periods, greater P- Δ amplification, and higher-mode contributions. The resulting risk is therefore not uniform across the city's building stock — it grows with story count, which motivates treating height as an explicit, isolated variable in this study.

Floor slab typology also shapes lateral load distribution and nonlinear behavior. Solid RC slabs on dropped beams are the standard floor system for residential and commercial mid-rise construction across North Africa [15], [16], offering greater lateral stiffness than flat-plate or ribbed alternatives. Dropped beams — typically projecting 300–400 mm below the slab soffit — add substantial flexural stiffness relative to flat plates or post-tensioned flat slabs, raising the column-to-beam flexural strength ratio ($\Sigma M_c / \Sigma M_b$). This benefit carries a trade-off: post-earthquake field investigations of similar construction confirm that heavy dropped beams often push plastic hinging out of the beams and into the columns, producing weak-column/strong-beam mechanisms that threaten global stability [17]–[19]. Dropped beams do, however, avoid the punching-shear vulnerability of flat-plate systems and form well-defined hinge zones at the beam ends, consistent with the lumped-plasticity model used here.

They also avoid the joint limitations associated with wide, shallow beams. ACI 318-19 limits the effective joint width to the column width where a beam is wider than the column [8], so the portion of a wide beam beyond the column faces contributes no joint shear area — reducing effective moment transfer and energy dissipation capacity in such configurations.

Nonlinear static pushover analysis (POA), codified in FEMA 356 [20], ATC-40 [21], and ASCE 41-17 [22], is an established performance-based tool for identifying nonlinear failure modes, plastic hinge sequences, and capacity limits in RC structures. By applying a monotonically increasing lateral load, POA traces the load-deformation response from elastic behavior through hinge formation to failure, capturing stiffness degradation, force redistribution, and the sequence of hinging toward collapse, and enabling direct comparison between the capacity curve and the demand spectrum. For regular mid-rise buildings where the first mode dominates, POA gives reliable estimates of yield displacement, peak base shear, and hinge sequence. This reliability is particularly relevant for OMF buildings in Tripoli, where the gap between elastic code compliance and actual inelastic behavior is the central open question.

Several studies apply POA to RC frames in comparable contexts. Bekkar et al. [23] found that Algerian RC frames designed to older codes show limited ductility and early strength degradation, with failure dominated by column hinging. This pattern is consistent with gravity-governed OMF detailing. Chaulagain et al. [24] reached similar conclusions after the 2015 Gorkha earthquake and attributed brittle failure to inadequate frame classification and detailing.

The present study adopts the same conventional (non-adaptive) pushover procedure with a fixed modal load pattern. Antoniou and Pinho [25] found that adaptive procedures offer only marginal improvement for regular frame configurations.

The literature review leaves three gaps for North African urban centers.

1. Building height is rarely isolated as a standalone variable governing inelastic OMF performance; comparative studies instead vary ductility classification [13] or floor system type [15] while holding story count constant.
2. Nonlinear performance data concentrate in high-seismicity regions such as Algeria [23], Nepal [24], and Turkey [17]–[19], leaving moderate-seismicity regions with distinct construction practices — such as Libya, despite Tripoli's SDC C/D characterization [7] — without empirical performance metrics calibrated to local hazard demand.
3. No study—to the author's knowledge—has evaluated how mid-height column-size reductions interact with rigid dropped-beam floor systems under nonlinear lateral loading for gravity-governed OMF buildings in Tripoli.

This paper addresses these gaps through a nonlinear static pushover analysis of 8-, 10-, and 12-story residential RC OMF buildings with solid slabs on dropped beams in Tripoli. The objectives are: (1) to characterize failure mode through hinge formation sequences and capacity-curve parameters; (2) to quantify ductility ratio, yield and ultimate displacement, and inter-story drift for each height; and (3) to establish ASCE 41-17 performance-level compliance and how it evolves with height.

2. Methodology

2.1 Structural Models

Three structural models isolate building height as the primary variable. Each is an OMF building with a solid slab on dropped beams, sharing plan geometry, materials, and site hazard, and differing only in story count:

- 8-story model (**H8**) — baseline model, representative of common mid-rise residential construction in Tripoli.
- 10-story model (**H10**) — intermediate model, the same configuration extended by two stories.
- 12-story model (**H12**) — upper-bound model, extended by two further stories, representing the upper range of mid-rise construction observed in the city.

2.2 Geometry and Material Properties

Each model shares a $22\text{ m} \times 15\text{ m}$ plan: four bays in the X-direction (6, 5, 5, 6 m) and three equal 5 m bays in the Y-direction (Figure 1). All members follow standard Libyan gravity-design practice under ACI 318-19 [8] ultimate strength combinations. The ground story is 3.5 m to accommodate commercial or lobby clearance; typical upper stories are 3.0 m. Table 1 gives the total height and aspect ratio for each configuration.

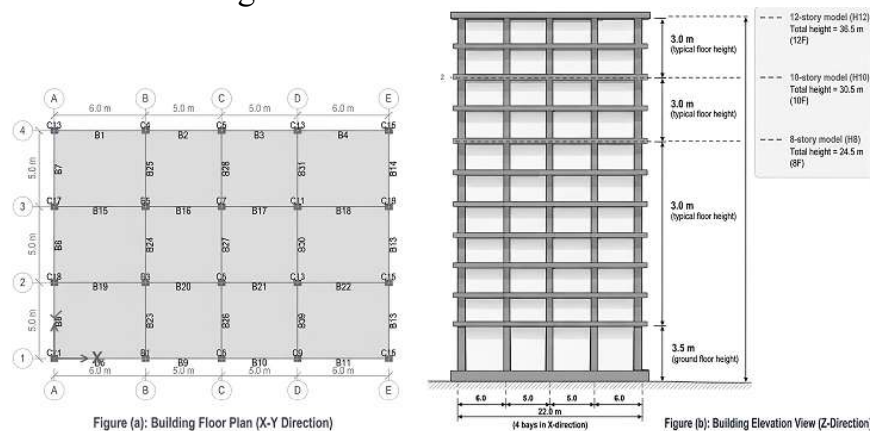


Table 1: Building height and aspect ratio by case (ground floor 3.5 m; typical floors 3.0 m).

Model	Stories	Total height (m)	$(H/B)_x$	$(H/B)_y$
H8	8	24.5	1.1	1.6
H10	10	30.5	1.4	2.0
H12	12	36.5	1.6	2.4

To reflect regional practice, where column dimensions are stepped down once over the height to reduce material cost, each configuration is split into two column groups — Lower and Upper — with a single 100 mm reduction at the mid-height transition (Table 2). This one transition is the only geometric discontinuity between groups.

Table 2: Column dimensions by story group.

Model	Stories	Group	Interior (mm)	Edge/corner (mm)
H8	1–4	Lower	600 × 600	500 × 600
	5–8	Upper	500 × 500	400 × 500
H10	1–5	Lower	600 × 600	500 × 600
	6–10	Upper	500 × 500	400 × 500
H12	1–6	Lower	600 × 600	500 × 600
	7–12	Upper	500 × 500	400 × 500

The solid slab is 200 mm thick, cast monolithically with 300 mm × 500 mm dropped beams (interior and edge), leaving 300 mm of beam depth below the slab soffit. Normal-weight concrete has a compressive strength (f'_c) of 30 MPa and a modulus of elasticity $E_c = 4700\sqrt{f'_c} = 25,743$ MPa. Reinforcement is Grade 400: $f_y = 400$ MPa for longitudinal bars and $f_{yt} = 300$ MPa for transverse ties. ETABS v21 auto-designs the reinforcement for each model configuration under identical detailing rules.

2.3 Loading and Effective Seismic Weight

Typical floors carry a superimposed dead load of 3.0 kN/m² and a live load of 2.0 kN/m²; the roof carries 2.0 kN/m² dead and 1.0 kN/m² live. ETABS applies slab self-weight (4.80 kN/m²) directly as structural dead load. Seismic mass follows a mass source of 1.0D + 0.25L. ASCE 7-16 §12.7.2 [9] requires a live-load contribution only for storage areas; the 0.25L term is adopted here as a conservative allowance for permanent furniture and sustained occupancy.

2.4 Seismic Hazard and Design Response Spectrum

The design response spectrum for Tripoli follows the ASCE 7-16 [9] procedure using the hazard parameters of Ngab and Tajouri [7]: short-period spectral acceleration $S_s = 0.32g$ and 1-second spectral acceleration $S_1 = 0.05g$, applied to all three models with $R = 3$ and $C_d = 2.5$ per ASCE 7-16 Table 12.2-1 [9]. These parameters correspond to Site Class D (stiff soil), representative of standard Tripoli coastal soils. For the nonlinear pushover evaluation under ASCE 41-17 [22] acceptance criteria, the unreduced MCE_R spectrum was used

directly, pushing structural capacity through the full inelastic range up to the target limits.

2.5 Modeling of Nonlinearity and Hinge Backbone Curves

3D structural modeling and nonlinear analysis were carried out in ETABS v21. Nonlinear behavior was captured with concentrated lumped-plasticity hinges at relative clear lengths of 0.05L and 0.95L: M3 moment hinges for beams and coupled P-M2-M3 hinges for columns, capturing biaxial bending-axial interaction consistent with ASCE 41-17 [22] and standard practice for avoiding numerical instability at joint boundaries.

Hinge properties follow the modeling parameters and acceptance criteria of ASCE 41-17 Tables 10-7 (beams) and 10-8 (columns) [22], with backbone curves derived member-by-member from reinforcement ratio, axial load ratio, and shear span-to-depth ratio using ETABS's "From Current Design" option, so hinge properties reflect each case's actual design. Effective stiffness modifiers of $0.35I_g$ (beams), $0.25I_g$ (slabs), and $0.70I_g$ (columns) follow ACI 318-19 Table 6.6.3.1(a) [8]. Geometric nonlinearity (P- Δ) is active in all analyses, which matters most for the taller cases.

2.6 Nonlinear Static Pushover Procedure and Target Displacement

Static pushover analyses follow ASCE 41-17 Chapter 7 [22]. A monotonically increasing lateral load, proportional to the fundamental mode shape, is applied in each principal direction — preferred here over a uniform pattern to better represent inertial force distribution in these regular configurations. The control node sits at the roof-level diaphragm center of mass; the analysis is displacement-controlled, with increments applied until roof drift reaches 4% of building height or a significant loss of lateral capacity occurs, consistent with collapse-prevention objectives.

All analyses were continued to at least 1.5 times the target displacement, as required by ASCE 41-17 [22]. Ultimate displacement (δ_u) is taken at 20% loss of peak capacity; where that level was not reached, the maximum analyzed displacement is used instead, so displacement ductility ($\mu = \delta_u / \delta_y$) is reported as a lower-bound estimate.

Performance levels — Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP) — are identified on the capacity curve along the inelastic post-yield backbone (B→C→D→E) using ASCE 41-17 acceptance criteria [22].

Maximum expected roof displacement under MCE_R demand is known as Target displacement and denoted (δ_t) follows the coefficient method of §7.4.3.3:

$$\delta_t = C_0 C_1 C_2 S_a \left(\frac{T_e}{2\pi} \right)^2 g, \quad (1)$$

$$T_e = T_i \sqrt{\frac{k_i}{k_e}} \quad (2)$$

where T_e the effective period, C_0 relates spectral to roof displacement, C_1 and C_2 account for inelastic displacement and hysteresis shape, S_a is the spectral acceleration at T_e , and $g = 9.81 \text{ m/s}^2$.

Where T_e exceeds 1.0 s in both directions, $C_1 = C_2 = 1.0$ throughout. Rather than the flat approximation in ASCE 7-16 Table 7-5, C_0 is computed directly in ETABS from each model's modal participation factor, as permitted by §7.4.3.3, giving case- and direction-specific values reported with the results in Section 3. S_a is taken from the unreduced (elastic) MCE_R spectrum: since C_1 and C_2 already account for the transition from elastic to inelastic demand, reducing S_a further would double-count that effect and understate δ_t .

3. Results

This section presents the pushover results for H8, H10, and H12, organized by parameter — dynamic properties, capacity curves, displacement and drift demand, and hinge formation — with all three models reported together within each section.

3.1 Dynamic Characteristics of the Model Configurations

Table 3 gives the fundamental periods and modal participation mass ratios, identifying the translational and torsional modes that govern seismic response.

Table 3: Fundamental periods and modal participation.

Model	Mode	Period (s)	Direction	U_x	U_y
H8	1	1.76	Y	0	0.805
	2	1.72	X	0.802	0
	3	1.59	Torsional	~ 0	~ 0
H10	1	2.22	Y	0	0.801
	2	2.17	X	0.799	0
	3	2.00	Torsional	~ 0	~ 0
H12	1	2.70	Y	0	0.799
	2	2.63	X	0.798	0
	3	2.42	Torsional	~ 0	~ 0

The fundamental period lengthens almost in proportion to story count in both directions, while the first translational mode carries about 80% of modal mass throughout — periods rise 53–58% from H8 to H12, tracking the 50% increase in story count. This dominant mode governs the effective period (T_e) and the modal load pattern used in the pushover analysis. Cumulative mass participation reaches the 90% threshold required by ASCE 7-16 §12.9.1 [9] in both directions

for all three configurations, converging faster in the Y-direction — exceeding 90% by the fourth mode — while the X-direction exceeded the limit by the fifth mode, consistent with its lower stiffness and more distributed modal mass.

3.2 Pushover Response Parameters

Table 4 summarizes the bilinear idealization of the capacity curves — yield, target, and ultimate displacements and corresponding base shears — for each configuration in both orthogonal directions. Figure 2 shows the corresponding capacity curves with δ_y and δ_t marked.

Table 4: Pushover response parameters.

Property	H8 (X)	H8 (Y)	H10 (X)	H10 (Y)	H12 (X)	H12 (Y)
δ_y (mm)	22.42	21.71	57.79	26.69	67.90	31.77
V_y (kN)	730.1	674.1	1465.2	643.5	1398.5	618.0
δ_t (mm)	65.74	67.60	83.36	85.74	101.28	105.14
V_t (kN)	2034	1981	2076.9	1945.4	2043.6	1922.3
δ_u (mm)	359.60	326.57	459.31	403.52	496.33	476.28
V_{max} (kN)	6545.6	5832.5	6931.4	6049.3	7368.2	6169.3
$\mu(\delta_u/\delta_y)$	16	15	8	15	7	15
S_a (g)	0.0697	0.0679	0.0549	0.0536	0.0451	0.0440

Yield displacement increased in both directions but at markedly different rates: δ_y rose 46% in Y (21.71 → 31.77 mm) against 203% in X (22.42 → 67.90 mm). Yield base shear diverged further

— up about 92% in X from H8 to H12 (peaking at H10 before declining), down about 8% in Y over the same range. Target displacement rose by comparable margins in both directions (54% in X, 55% in Y), while target base shear stayed nearly constant (under 1% in X, about 3% in Y). Ultimate displacement rose 38% in X and 46% in Y; ultimate base shear rose about 13% in X and 6% in Y. The ductility ratio stayed essentially flat in Y ($\mu \approx 15$ across all heights) but fell 56% in X, from $\mu = 16$ at H8 to $\mu = 7$ at H12. Spectral acceleration fell about 35% in both directions across this height range.

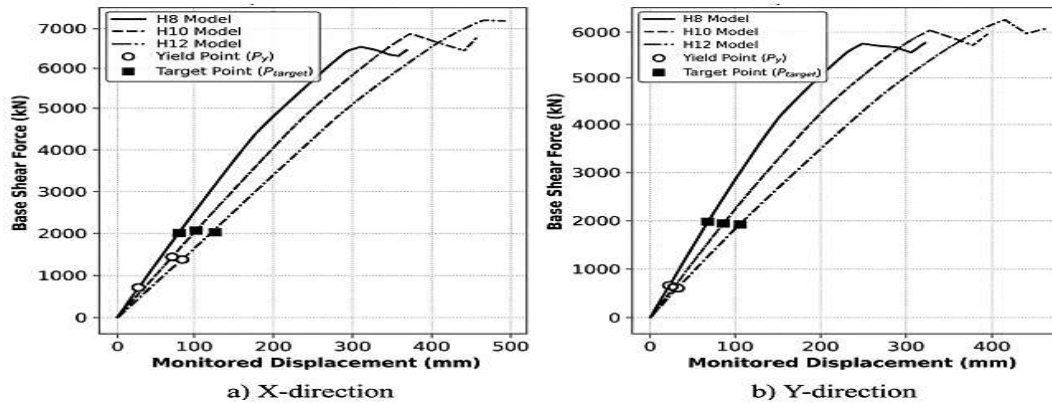


Figure 2: Capacity Curves and Limit States for H8, H10, and H12 Models: a) X-direction, b) Y-direction

In both directions, the H8 curve has the steepest initial slope, consistent with its lowest yield displacement (22.42 mm in X, 21.71 mm in Y), reaching a distinct peak followed by a mild plateau and slight softening beyond roughly 280–300 mm. The H10 and H12 curves start with progressively lower initial stiffness and yield further right (57.79–67.90 mm in X; 26.69–31.77 mm in Y), reflecting the added flexibility of the taller configurations. Beyond about 350–400 mm displacement, the H10 and H12 curves cross above the H8 curve and continue rising, ultimately reaching the highest peak base shears of the three ($V_{max} = 6931\text{--}7368$ kN in X; $6049\text{--}6169$ kN in Y). The target points for all three configurations cluster within a narrow base-shear band ($\sim 2034\text{--}2077$ kN in X; $\sim 1922\text{--}1981$ kN in Y) despite occurring at increasingly larger displacements with height, placing the target point on the near-flat, post-yield segment of the capacity curve in every configuration.

3.3 Milestone Response and Drift Concentration

Table 5 compares plastic-hinge performance states between beams and columns at ultimate displacement (δ_u) for all three models.

Table 5: Element-based Performance at Ultimate Displacement.

Model / Dir. (X/Y)	Element Type	Total hinges	A→IO	IO→LS	LS→CP	>CP
H8 X/Y	Beams	496	496 (100%)	0	0	0
			496 (100%)	0	0	0
H10 X/Y		620	620 (100%)	0	0	0
			620 (100%)	0	0	0
H12 X/Y		744	744 (100%)	0	0	0
			744 (100%)	0	0	0
H8 X/Y	Columns	320	265 (82.8%)	28 (8.8%)	0	27 (8.4%)
			274 (85.6%)	31 (9.7%)	5 (1.6%)	10 (3.1%)
H10 X/Y		400	350 (87.5%)	24 (6.0%)	4 (1.0%)	22 (5.5%)
			354 (88.5%)	28 (7.0%)	5 (1.3%)	13 (3.3%)
H12 X/Y		480	433 (90.2%)	34 (7.1%)	2 (0.4%)	11 (2.3%)
			434 (90.4%)	27 (5.6%)	6 (1.2%)	13 (2.7%)

Beam hinges remain entirely within A→IO in every model and direction — none reach even minor yielding at δ_u . Column response is bimodal: 82.8–90.4% of hinges remain within A→IO, but a small, localized fraction (2.3–8.4%) exceeds Collapse Prevention, and it is this fraction that governs the overall column performance level. All three models show the same qualitative pattern; they differ only in where along the height that fraction concentrates.

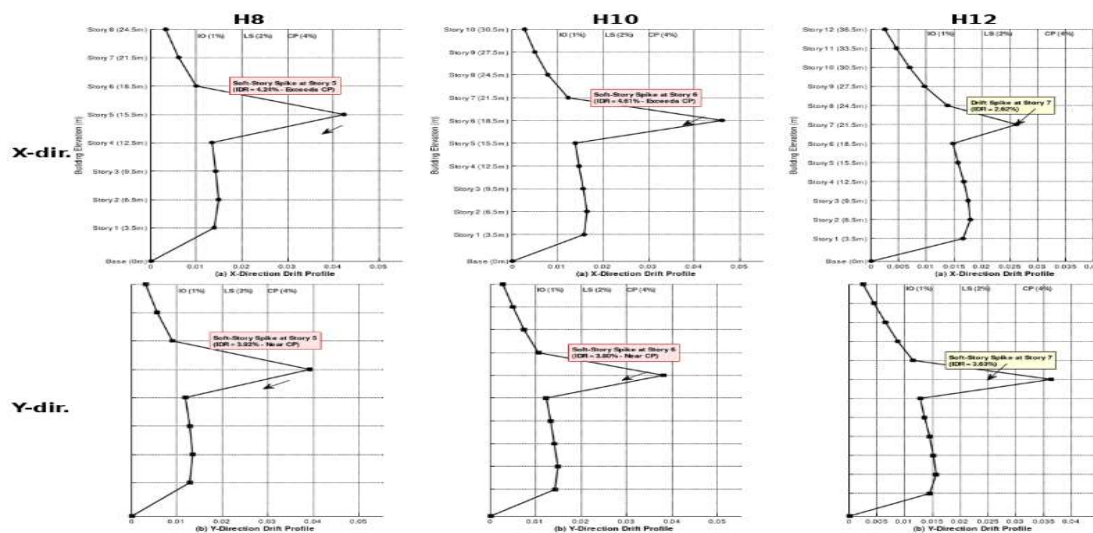
Table 6 summarizes the step, displacement, base shear, and hinge distribution at δ_t and δ_u for all three models. Because the analytical δ_t rarely coincides with a discrete ETABS output step, the closest sequent step (equal or slightly larger displacement) is reported.

Table 6: Hinge States at Target and Ultimate Displacements.

Model / dir.	Milestone	Disp. (mm)	Base shear (kN)	A→B	B→C	C→D	A→IO	IO→LS	LS→CP	>CP
H8 X	δ_t	72.64	2240.4	710	106	0	816	0	0	0
	δ_u	359.60	6545.6	462	347	7	761	28	0	27
H8 Y	δ_t	74.03	2162.6	713	103	0	816	0	0	0
	δ_u	326.57	5832.5	485	321	10	770	31	5	10
H10 X	δ_t	97.60	2409.7	896	124	0	1020	0	0	0
	δ_u	459.31	6931.4	639	358	23	970	24	4	22
H10 Y	δ_t	85.74	1945.4	896	124	0	1020	0	0	0
	δ_u	403.52	6049.3	617	391	12	974	28	5	13
H12 X	δ_t	116.80	2337.1	1072	152	0	1224	0	0	0
	δ_u	496.33	7368.2	772	448	4	1177	34	2	11
H12 Y	δ_t	104.61	1913.1	1074	150	0	1224	0	0	0
	δ_u	476.28	6169.3	756	462	6	1178	27	6	13

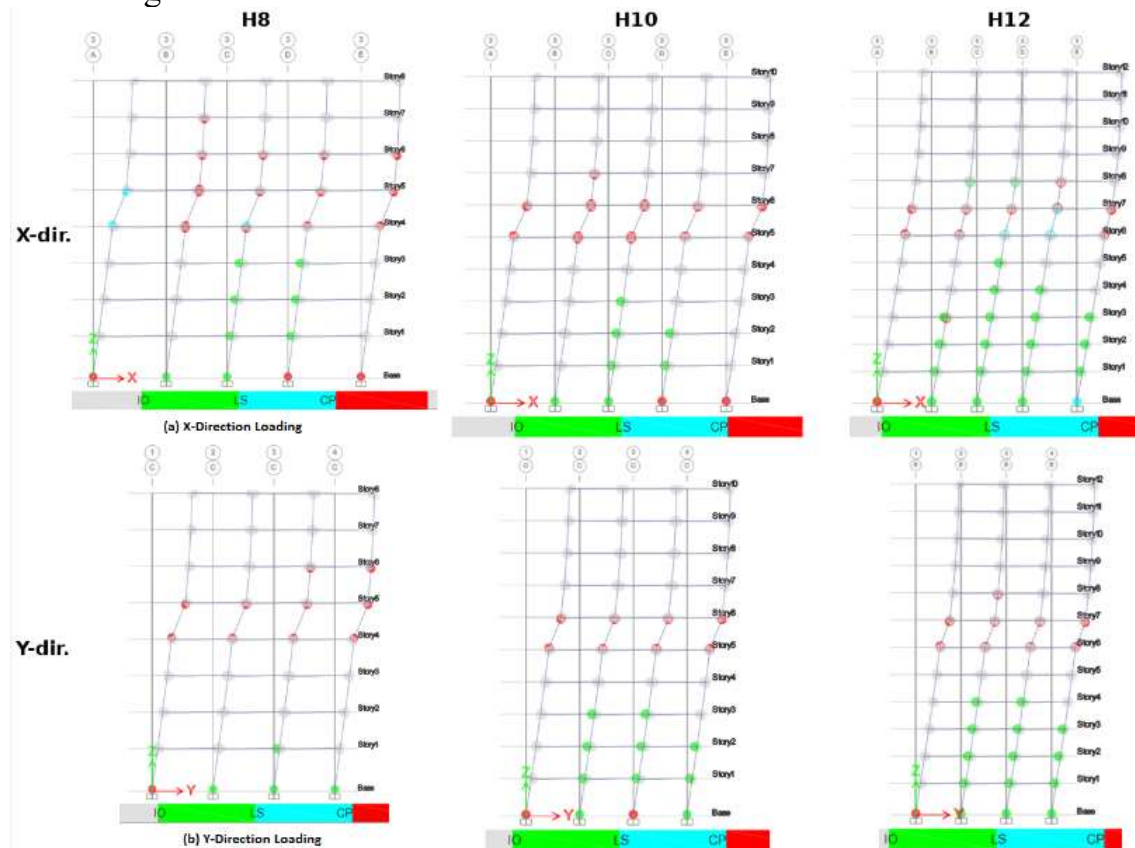
At δ_t , every model and direction remain entirely within A→IO: no hinge crosses the B–C (yielded) boundary while staying outside the Immediate Occupancy limit, and 12–13% of hinges have started to yield (106/816 in H8, 124/1020 in H10, 152/1224 in H12, X-direction). First yield occurs much earlier — at 3.8–15.9% of δ_u , engaging under 2% of hinges — with Y generally yielding first, though H10 shows near-identical yield onset in both directions; the stiffer X-direction then accumulates B–C hinges faster. Peak base shear is reached at 80–100% of δ_u across all cases; the X-direction retains a 17–20% post-peak displacement margin in H8 and H10, while H12 and both Y-direction cases reach peak strength concurrently with, or just before, the ultimate limit. At δ_u , hinges beyond Collapse Prevention (>CP) remain a small fraction of column hinges in every case (2.3–8.4%), consistent with Table 5.

Figure 3 shows the height-wise inter-story drift ratio (IDR) profile for each model at δ_u , plotted against the ASCE 41-17 thresholds for Immediate Occupancy (1%), Life Safety (2%), and Collapse Prevention (4%).



In every model, drift below the transition story stays uniform and well-controlled (1.0–1.8%, within Life Safety limits), while a sharp, single-story spike develops at the first story above the column-size transition: Story 5 in H8 (peak IDR 4.24% X / 3.92% Y), Story 6 in H10 (4.61% X / 3.80% Y), and Story 7 in H12 (2.62% X / 3.63% Y). Above the spike, drift falls rapidly, dropping

below the 1% IO limit within one to two stories. The peak drift is more severe in the stiffer X-direction for H8 and H10 but reverses in H12, where the Y-direction governs.



The hinge maps corroborate the drift profiles directly. In every model, hinges beyond Collapse Prevention (red markers) cluster at the column ends of the story immediately above the transition, while lower stories show only moderate yielding within IO–LS (green markers) and the stories above the spike remain essentially undamaged. A secondary, smaller concentration of >CP hinges also appears at the base columns, mainly in the X-direction, but it does not govern the global drift profile in any model. Beam elements show no inelastic action beyond IO anywhere in the elevation, confirming the strong-beam, weak-column behavior already evident in Table 5.

4. Discussion

4.1 Period Elongation and Directional Ductility Trends

The dynamic response of these OMF structures is governed chiefly by overall flexibility, total height, and plan aspect ratio — $(H/B)_X$ versus $(H/B)_Y$. As height increases from 8 to 12 stories, fundamental periods elongate from 1.76 s to 2.70 s. Under Tripoli's design spectrum, this shift moves structural demand into the constant-velocity and constant-displacement spectral regions, where spectral accelerations drop below 0.07g. Despite the absence of seismic detailing, all three heights satisfy ASCE 41-17 Immediate Occupancy criteria at δ_t .

Beyond its effect on stiffness and period, plan aspect ratio also governs the *type* of lateral deformation each direction develops. Aspect ratios below approximately 1.5 are associated with squat, shear-influenced response, where stiffness dominates and inelastic action concentrates rapidly at low drift; ratios above approximately 2.5 are associated with slender, flexure-dominated response, where deformation distributes more gradually through curvature and rotation; the intermediate 1.5–2.5 range reflects a transitional zone combining both mechanisms [26]. This framework, established for RC wall behavior, extends naturally to describe the global deformation character of the frame system here, since it is the same height-to-width geometry that governs how lateral stiffness and inelastic demand distribute across the structure. The nonlinear capacity metrics reveal a clear directional divergence in displacement ductility (μ), consistent with this contrast in orthogonal aspect ratios:

1. **Y-direction — ductility holds steady.** Along the narrower Y-axis, aspect ratios $((H/B)_Y = 1.63\text{--}2.43)$ fall within the moderate-to-slender range, where flexural action increasingly governs the response as height increases. This added global flexural slenderness lets inelastic rotations distribute more uniformly across stories before hinges degrade severely. Yield (δ_y) and ultimate (δ_u) displacement scale up together, holding global ductility near $\mu_Y \approx 15.0$ across all three heights.
2. **X-direction — ductility degrades.** Along the wider, stiffer X-axis, aspect ratios $((H/B)_X = 1.11\text{--}1.66)$ remain largely below the moderate threshold, placing the system's global response closer to the shear-influenced regime. Lower aspect ratios drive rapid base shear accumulation at low drift, consistent with stiffness-dominated behavior. As height increases, P- Δ moment magnification under heavy gravity load accelerates column yielding: initial yield displacement jumps 203% (22.4 mm at H8 to 67.9 mm at H12), while ultimate displacement capacity rises only 38%. System ductility (μ_X) falls 56% as a result, from 16.0 to 7.3.

This X-direction ductility loss reflects the combined effect of a shear-influenced global deformation mode, greater initial stiffness, and secondary P- Δ

magnification: as taller frames displace laterally, gravity loads acting through larger roof drifts accelerate column yield initiation without a matching gain in ultimate displacement capacity.

4.2 Localized Soft-Story Failure at Column Section Transitions

All three models show a persistent strong-beam/weak-column pattern. The 300 × 500 mm dropped beams are stiff relative to the supporting columns; because members were sized for gravity combinations alone, beams carry substantial reserve flexural capacity while columns have little moment reserve under lateral racking.

Damage is not distributed uniformly up the height — it concentrates sharply at the story immediately above the column-section transition (~60% of total height). In H8, for example, interior columns step from 600 × 600 mm (Stories 1–4) to 500 × 500 mm (Stories 5–8), cutting the gross moment of inertia (I_{ug}) of interior columns by 51.8%. Story shear decreases gradually with height, but column flexural and shear capacity drop discretely at this boundary. The resulting stiffness and strength step forces inelastic deformation to concentrate at the top and bottom of columns in the transition story — Story 5 in H8, Story 6 in H10, Story 7 in H12. As roof drift pushes into the post-yield range, hinges in these transition columns rapidly exceed Collapse

Prevention (>CP), forming a mid-height soft-story mechanism while the surrounding stories remain comparatively undamaged.

4.3 Engineering and Policy Implications for Local Construction

These findings carry three practical implications for structural practice and code development in Tripoli and similar North African cities:

1. **Compliance reflects period elongation, not systemic ductility.** Gravity-designed OMF buildings up to 12 stories meet ASCE 41-17 Immediate Occupancy criteria under Tripoli's moderate MCE_R spectrum, but that compliance depends heavily on period elongation reducing spectral demand — it does not indicate genuine systemic ductility.
2. **Mid-height column transitions need targeted inspection.** Field inspection typically focuses on ground-story columns, but this study shows that for mid-rise frames with a stepped column schedule, damage concentrates at the mid-height transition instead. Retrofitting measures such as FRP jacketing should prioritize these mid-height column-beam connections.
3. **A practical height limit for gravity-designed OMFs.** The severe X-direction ductility loss at 12 stories suggests that extending this typology — solid slabs on dropped beams, OMF detailing — beyond 12 stories in SDC C/D zones carries an unacceptable risk of soft-story formation under stronger ground motion.

5. Conclusions

Compliance at target demand. The 8-, 10-, and 12-story RC OMF buildings studied here, with solid slabs on dropped beams, satisfy ASCE 41-17 Immediate Occupancy criteria under the unreduced MCE_R spectrum: at δ_t , zero plastic hinges exceed the IO threshold in any configuration.

1. **Weak-column mechanisms dominate.** Inelastic behavior across all three models is governed entirely by column hinging. The heavy 300×500 mm dropped beams stay elastic (A→IO) up to ultimate displacement, which prevents the more desirable beam-sway mechanism from developing.
2. **Damage concentrates at the mid-height soft story.** Stepping column dimensions down once at roughly 60% of building height creates a sharp stiffness and strength discontinuity. Inter-story drift peaks at 2.62–4.61% at this transition story, and plastic hinging beyond Collapse Prevention concentrates almost exclusively in these transition columns.
3. **Ductility loss is height- and direction-dependent.** Fundamental periods elongate roughly linearly with height. Displacement ductility along the weaker Y-axis stays stable ($\mu \approx 15.0$) across all heights, but ductility along the stronger X-axis drops from 16.0 at 8 stories to 7.3 at 12 stories, driven by a sharp rise in yield displacement rather than a loss of ultimate capacity.
4. **Gradual column reduction is needed.** Abrupt single-step reductions in column size should be avoided in mid-rise frames. Column sections should change gradually over the height. This distributes drift more uniformly and prevents soft-story failure.

References

- [1] S. S. Suwihli and T. R. Paradise, "Seismic risk perception in Al-Marj, Libya: A case study after the 1963 earthquake," *Open J. Earthq. Res.*, vol. 9, no. 4, pp. 349–366, 2020.
- [2] R. Lagesse, M. Free, and Z. Lubkowski, "Probabilistic seismic hazard assessment for Libya," in *Proc. 16th World Conf. Earthq. Eng. (16WCEE)*, Santiago, Chile, 2017, pp. 1–12.
- [3] F. M. Layas, V. Karakale, and R. E. Suleiman, "Updated probabilistic seismic hazard assessment for Benghazi City, Libya," *Build. Eng.*, vol. 1, no. 1, Art. no. 1190, 2024.
- [4] A. Z. Bishta, "Reassessment of the seismicity and seismic hazards of Libya," in *EGU Gen. Assem. Conf. Abstr.*, vol. 11, 2009, Art. no. 4462.
- [5] A. S. Ngab, E. Korban, and S. Hamuda, "Seismic hazard analysis for Libya," in *Proc. 9th Nat. Conf. Build. Mater. Struct. Eng.*, Tripoli, Libya, 2022.
- [6] I. A. Mohammed and S. A. Khatrush, "Seismic response of reinforced concrete buildings as predicted by the draft of Libyan standard (DSLS-1977) and (IBC-2009)," in *Proc. 1st Conf. Eng. Sci. Technol. (CEST-2018)*, Tripoli, Libya, 2018, pp. 469–475.
- [7] A. S. Ngab and G. M. Tajouri, "Seismic response spectrum curves for selected cities in Libya," *Acad. J. Basic Appl. Sci. (AJBAS)*, vol. 5, no. 2, pp. 196–204, Jul. 2023.
- [8] ACI Committee 318, *Building Code Requirements for Structural Concrete (ACI 318-19) and Commentary*, American Concrete Institute, Farmington Hills, MI, USA, 2019.
- [9] American Society of Civil Engineers, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, ASCE/SEI 7-16, Reston, VA, USA, 2016.
- [10] L. O. Gargab, "Comparative seismic performance of OMF vs. IMF for reinforced concrete residential multi-story buildings in Tripoli, Libya," *Acad. J. Basic Appl. Sci. (AJBAS)*, vol. 8, no. 1, 2026, doi: 10.5281/zenodo.20398162.
- [11] C. B. Haselton, A. B. Liel, G. G. Deierlein, and J. W. Baker, "Seismic collapse safety and behavior of modern reinforced concrete moment frame buildings," *Struct. Saf.*, vol. 61, pp. 28–39, Jul. 2016.
- [12] Y. Zulshae, Herwani, and M. Yusuf, "Design of reinforced concrete structure with ordinary moment frame in low seismic region," *J. Tek. Sipil*, vol. 24, no. 1, pp. 721–735, 2024.
- [13] M. Ismail, H. Alam, and O. Zahid, "Pushover analysis of RC frames with varying ductility levels," *Eng. Struct.*, vol. 260, Art. no. 114230, Jun. 2022.
- [14] A. M. Mwafy and A. S. Elnashai, "Calibration of force reduction factors of RC buildings," *J. Earthq. Eng.*, vol. 6, no. 2, pp. 239–273, 2002.

- [15] T. Almusallam, S. Elsanadedy, H. Abbas, Y. Al-Salloum, and T. Ngo, "Seismic assessment of RC buildings with different floor systems in a moderate seismic zone," *Eng. Struct.*, vol. 244, Art. no. 112773, Oct. 2021.
- [16] A. Hassan and M. Mohamad, "Nonlinear seismic response comparison of RC frames with ribbed and solid slabs in moderate seismicity zones," *Structures*, vol. 47, pp. 1943–1958, Jan. 2023.
- [17] H. Alwashali, Y. Torihata, K. Jin, and M. Maeda, "Experimental observations on the in-plane behaviour of masonry wall infilled RC frames," *Bull. Earthq. Eng.*, vol. 17, no. 7, pp. 3961–3987, 2019.
- [18] A. Demir *et al.*, "Field observations on the seismic performance of RC buildings with joist floors in the February 2023 Turkey earthquakes," *J. Build. Eng.*, vol. 82, Art. no. 108276, Apr. 2024.
- [19] F. Avcil *et al.*, "Structural performance of RC buildings with ribbed slabs during the 2023 Kahramanmaraş earthquakes," *Eng. Fail. Anal.*, vol. 150, Art. no. 107312, Aug. 2023.
- [20] Federal Emergency Management Agency, *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*, FEMA 356, Washington, DC, USA, 2000.
- [21] Applied Technology Council, *Seismic Evaluation and Retrofit of Concrete Buildings*, ATC-40, Redwood City, CA, USA, 1996.
- [22] American Society of Civil Engineers, *Seismic Evaluation and Retrofit of Existing Buildings*, ASCE/SEI 41-17, Reston, VA, USA, 2017.
- [23] H. Bekkar, A. Tiliouine, and M. Chemrouk, "Pushover analysis of RC frame buildings in Algeria," *Asian J. Civ. Eng.*, vol. 19, no. 4, pp. 451–464, 2018.
- [24] H. Chaulagain, H. Rodrigues, E. Spacone, and H. Varum, "Seismic response of current RC buildings in Kathmandu Valley, Nepal," *Structures*, vol. 6, pp. 1–15, May 2016.
- [25] S. Antoniou and R. Pinho, "Advantages and limitations of adaptive and non-adaptive force-based pushover procedures," *J. Earthq. Eng.*, vol. 8, no. 4, pp. 497–522, 2004.