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Comparative Seismic Performance of OMF Vs. IMF For Reinforced Concrete Residential Multi-storey Buildings in Tripoli, Libya

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***Abstract*— Current construction practices in Tripoli, Libya, still depend on Ordinary Moment Frames (OMF) integrated with ribbed slab systems based on the traditional assumption of low seismicity in the region, however, recent local seismic hazard studies have classified the region Seismic Design Category (SDC) as C or D, which necessitates a critical assessment under the current structural norms. The seismic adequacy of an eight-story reinforced concrete (RC) building was investigated under three different configurations based on American Concrete Institute (ACI 318-19) and American Society of Civil Engineers (ASCE 7-16): conventional OMF with ribbed slabs, Intermediate Moment Frames (IMF) with ribbed slabs, and IMF with solid slabs. The standard OMF-ribbed configuration was analyzed and found to have 23.4% of the beams failing due to critical shear-torsion failures, which is fundamentally not adequate for current demands, while upgrading to an IMF-ribbed system reduced the failure rate to 10.5% (55% improvement) that is still not compliant with the code, and the IMF with a solid slab system achieved 100% structural adequacy with no beam failures, inter-story drift of 0.57%, and column demand-capacity (P-M) ratios within a safe threshold of 0.68.** These localized findings suggest that current construction practices may underestimate seismic vulnerability, and consequently, implementing an IMF integrated with a solid slab system provides an enhanced structural configuration to satisfy modern code provisions and warrants consideration in future revisions of local construction regulations**.**

Keywords*— *Seismic Vulnerability, Reinforced Concrete Frames, Response Spectral Analysis, Structural Stability

I. INTRODUCTION

The structural building systems used in Tripoli, Libya have transitioned dramatically throughout the last century, from traditional masonry construction to the modern reinforced concrete systems. Most of the urban buildings were built using unreinforced masonry (URM) load-bearing walls in the early 1900s [1]. By the mid-20th century, the construction industry gradually moved to reinforced concrete multistory frame buildings [2], [3]. Despite major advances in international seismic design standards, these structural systems still use the conventional ordinary moment resisting frames (OMF) originally designed to resist gravity loads. This is generally attributable to the long-standing perception that Libya has a relatively low level of seismic risk [1].

This structural design approach has resulted in inadequate ductile detailing in beam-column joints, reducing their ability to form stable plastic hinges and dissipate seismic energy effectively [2]. Therefore, these joints usually have a lack of confinement and anchorage, making the existing buildings more susceptible to inelastic deformation under seismic loads [4]. Lack of these essential detailing requirements for joints may lead to permanent inelastic deformation, strength degradation, and increased risk of brittle failure [5], [6]. Therefore, such beam-column joint systems are generally characterized by low ductility and limited energy dissipation capacity during ground motions, which causes considerable structural vulnerability under modern seismic hazard assessments [7], [8].

Libya is often considered a region with relatively low seismic activity and free from large ground shakes. However, its earthquake history indicates that seismic risk should not be disregarded. The ground activities are largely driven by the complex tectonic interaction between the African and Eurasian plates and a local geological activity in rift zones such as the Hun Graben and the Sabratah Basin [1]. Historically, Libya has experienced several considerable earthquakes, including the catastrophic 1963 Al Marj earthquake (body wave magnitude 5.6) and the 1977 Hun Graben earthquake (Mw 5.9–6.0). Both of these events caused serious damage to masonry buildings and poorly reinforced concrete structures [1], [9]. These earthquake events demonstrate the need for appropriate seismic design to mitigate the serious risk to life and property.

In North Africa, including northern Libya, ordinary moment frames (OMFs) remain the dominant building system. These frames were originally designed mainly to support gravity loads [2], [4]. However, this design approach prevailed despite the region having a documented history of seismic activity and increasing human and property loss risk. Consequently, the seismic capacity of these buildings may not be sufficient to meet current design standards [4].

Most local maps and data are either outdated or from broad regional enterprises such as the Global Seismic Hazard Assessment Program (GSHAP). These sources usually miss important local effects, such as site amplification effects associated with Tripoli's complex coastal geology [10]. However, recent site-specific probabilistic seismic hazard assessments (PSHA) suggest that the design spectral accelerations for Tripoli are significantly higher than previously estimated. Consequently, Tripoli may require a higher seismic classification —possibly Seismic Design

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Category (SDC) C or D under the ACI 318-19 [18] building code [2], [11], [12].

Although neighboring North African countries (Algeria, Egypt, and Morocco) have updated their seismic codes after major earthquakes, still the Draft of the Suggested Libyan Standard (DSLS-1977) is the only currently national seismic design code in Libya. Specifically, this draft is outdated, inconsistent with modern standards, and even poorly enforced [4], [13]. The lack of updated national seismic code has continued the use of outdated design standards that rely on gravity loads instead of ductile lateral-force-resisting systems, resulting in a large gap between the actual seismic demand and the available lateral capacity [1], [13]. Furthermore, Libya has not adopted an international seismic design code as a standard basis, which has led structural engineers to rely on various international standards that are often not calibrated to local seismic conditions.

Recent investigations have increasingly focused on the critical vulnerabilities of Mediterranean building stocks to realistic seismic demands. Elsouri and Harajli (2015) [14] experimentally evaluated as-built beam-column joints designed for gravity loads, they revealed that the lack of transverse reinforcement within the joint core causes brittle shear failure, even at very low lateral drift levels. This deficiency in core confinement leads to an early loss of load-carrying capacity and severely reduces overall deformation capacity of the frame during cyclic seismic loading. Separately, Meslem and Moussa (2021) [15] showed that moving from non-seismic to seismic design standards produces a major shift in structural fragility curves and their analytical results were based primarily on solid slab models. following the 2023 Kahramanmaraş earthquake sequence, two post-event studies by Demir et al. (2024) [16] and Avcil et al. (2024) [17] investigated buildings using concrete ribbed slab systems without supplemental lateral-force-resisting elements. They showed that these configurations possess significantly lower in-plane diaphragm stiffness and amplified P-Delta effects compared to traditional solid slab systems. Especially, the reduction in lateral load distribution, coupled with increased second-order instability, substantially elevates the risk of soft-story collapse mechanisms during strong ground motion.

Despite growing evidence of higher seismic risk, there is a critical gap in both academic research and professional practice regarding the performance-based assessment of IMFs as a minimum seismic-resisting system for Libyan buildings. While international provisions, like ACI 318-19 [18] and ASCE 7-16 [19], require enhanced ductile detailing for structures in SDC C and D, the implications of these rules on building stock in Tripoli have not been adequately quantified, in terms of fundamental response parameters such as inter-story drift, lateral stiffness and overall deformation capacity [2] , [7], [20].

To address this gap, the present study performs a comparative parametric evaluation of OMF and IMF systems designed according to ACI 318-19 [18] and ASCE 7-16 [19]. The results provide a technical basis for updating seismic design practices in Tripoli and, more broadly, for guiding code development across North Africa.

II. METHODOLOGY

A three-dimensional structural model was created using ETABS v20, employing a lateral moment-resisting frame system with semi-rigid or rigid diaphragms—depending on the slab-beam system used—to ensure proper distribution of lateral forces to the frames. Effective stiffness modifiers based on recommendations from ACI 318-19 [18] were applied to account for cracking behavior of reinforced concrete under sustained gravity and cyclic seismic loading. Three models were developed and analyzed sequentially across four distinct cases, as summarized below:

A. Case 1-OMF-ribbed slab model (Gravity Validation):

In this case, the model—denoted model A—was subjected to gravity loads alone (DL + LL combinations as per ACI 318-19, chapter 5) [18]. This step served to validate that all structural member satisfied strength requirements under non-seismic conditions, establishing baseline structural integrity prior to seismic evaluation.

B. Case 2-OMF-ribbed slab model (Seismic Evaluation):

In this case, the model A was subjected to seismic excitation using hazard parameters (Ss and S1) derived from local Libyan studies. Response modification factor (R), overstrength factor (Ωo), and displacement amplification factor (Cd) relevant to OMF systems were adopted as per ASCE 7-16 (Table 12.2-1) [19].

C. Case 3-IMF-ribbed slab Model:

In this case, the moment-resisting system was upgraded to an Intermediate Moment Frames (IMF) while retaining the ribbed slab with concealed beams—denoted as Model B. The moment-resisting frame system parameters (R, Ωo, and Cd) were updated accordingly as per ASCE 7-16 [19].

D. Case 4-IMF-solid slab model:

In this case, the slab system was upgraded to a solid slab with dropped beams while maintaining the IMF system—denoted Model C. The model was subjected to the same seismic hazard parameters as Case 3, with gravity loads adjusted for the superimposed dead load values relevant to the solid slab system.

The seismic behavior of the three models (Models A, B, and C) was evaluated using both equivalent lateral force (ELF) and response spectrum analysis (RSA) procedures. Key seismic response parameters—including inter-story drifts, base shear distribution, and modal properties including fundamental periods and modal participation factors—were compared to identify the extent of improvement achieved by utilizing different moment-resisting detailing and slab systems.

III. PROTOTYPE MODELING

The reference structure model—denoted model A—represents a typical 8-story residential building consistent with the prevailing architectural and structural configuration practices in Tripoli, utilizing an OMF as the primary lateral force-resisting system in both principle directions. The total plan dimensions of 22 m ×15 m is divided into four bays along the X-direction with four spans of 6 m, 5 m,5 m, and 6 m, and three equal spans of 5 m each along the Y-direction as shown in Figure 1.

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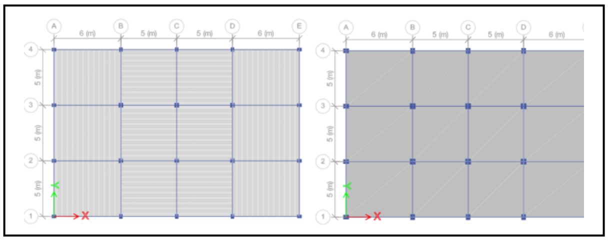

Fig. 1. Typical floor plans of the two slab systems: (a) ribbed slab illustrating rib orientation (Models A and B); (b) solid slab with dropped beams (Model C).

The ground floor height is taken as 3.5 m, followed by seven typical floors of 3.0 m each, resulting in a total building height of 24.50 m. These proportions are broadly representative of mid-rise residential construction where a higher elevation at the ground level is often adopted for commercial use as shown in Figure 2.

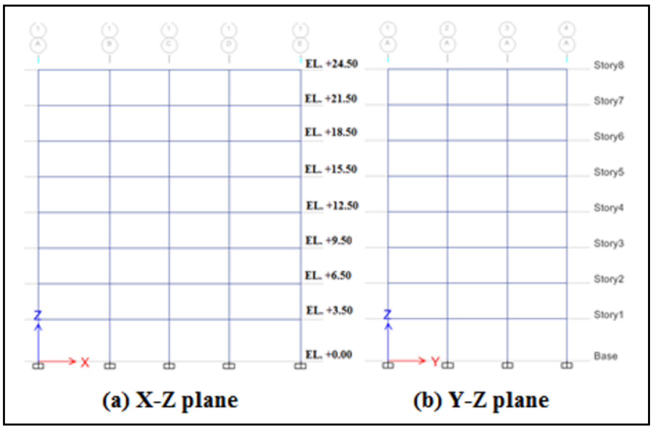

Fig. 2. Elevation views of the building: (a) X-Z plane; (b) Y-Z plane.

A 25 cm ribbed slab is adopted for typical floors and the roof. The ribbed slab consists of hollow clay bricks of size 20×17×40 cm arranged in the slab plan such that rectangular ribs of 12 cm width are formed between the bricks, with a concrete topping of 8 cm thickness, shown in Figure 3.

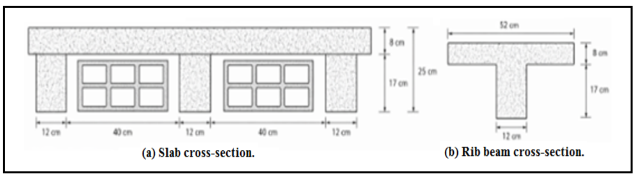

Fig. 3. Rib slab details: (a) Slab cross-section; (b) Rib beam cross-section.

A concrete of compressive strength (f'c) of 30 MPa, reinforcement yield strength of 400 MPa, and stirrups yield strength (fyt) of 300 MPa are adopted in accordance of local Libyan market standards. Gravity loading consists of a superimposed dead load (SDL) of 4.5 kN/m² and a live load (LL) of 2 kN/m² applied to typical floors, whereas an SDL of 3 kN/m² and an LL of 1 kN/m² are applied to the roof slab for Models A and B. For the solid slab system (Model C), an SDL of 2.5 kN/m² is assigned for typical floors and 2 kN/m² for the roof slab, while the live load remains unchanged. These loads are applied in addition to the self-weight of structural elements, which is calculated automatically by the program (ETABS v20) for all three models.

The modeling approach incorporates several critical analytical assumptions to ensure code compliance with ACI 318-19 [18] and ASCE 7-22 [19]. Specifically, cracked section properties are applied using stiffness modifiers of 0.35Ig for beams, 0.25Ig for slabs, and 0.70Ig for columns to reflect realistic seismic response under service-level and design-level drifts according to ACI 318-19, as per Table 6.6.3.1(a). Furthermore, second-order effects (P-Delta) are included in the analysis to account for geometric nonlinearities amplified by the taller ground floor height. In addition, the seismic mass is derived from load components using a mass source definition of 1.0D + 0.25L, assuming that 25% of live load is sustained during the design-level earthquake for residential occupancy, ASCE 7-16, Section 12.7.2. For the comparative analysis, the OMF model is assigned a response modification factor (R) of 3 and a deflection amplification factor (C_d) of 2.5, while the IMF model utilizes R=5 and C_d=4.5 with mandatory Chapter 18 detailing requirements enabled and per ASCE 7-16 (Table 12.2-1) [19].

IV. RESULTS AND DISCUSSION

This section presents and discusses the results obtained from modal and strength analysis of the three structural configurations. Significant seismic response parameters—including fundamental periods, inter-story drifts, beam failure rates, and column demand-capacity ratios—are examined to evaluate performance relative to ACI 318-19 [18] and ASCE 7-16 [19] code requirements.

A. Model A: Gravity-Based Baseline (Case 1)

Model A was developed as a benchmark representative of current regional practice, designed for gravity load combinations in accordance with ACI 318-19. The model demonstrated reliable performance as all frame elements satisfied the code's ultimate strength requirements, maintaining demand-capacity ratios (**D/C**) below 1.0. However, a more careful investigation revealed a local vulnerability in the axial load distribution. Specifically, the interior columns on the lower levels and the perimeter columns at the roof line showed axial stress ratios of 0.44. This value is in agreement with the upper value of 0.55 determined by ACI 318-19 for tied columns under gravity-dominant loading, however, this signals a potential seismic vulnerability. This stress level is considerably exceeding the 0.35 limit suggested by [21]), to ensure a sufficient flexural ductility capacity against substantial lateral displacement anticipated during seismic excitations. While Model A is stable under gravity loads, a pre-existing stiffness deficit is already present, hence, the structure seismic the seismic analysis phase with limited capacity for ductile energy dissipation.

B. Seismic Response of the OMF-Ribbed System (Case 2)

Case 2 was exemplified by subjecting model A to seismic excitation defined with seismic hazard spectral parameters of S_s=0.32g and S_1=0.05g for Tripoli harbor zone as suggested by Ngab (2023). A response modification factor (R) of 3 and a deflection amplification factor (C_d) of 2.5 are adopted for the case of OMF frames. A modal analysis revealed that, in the X-direction, a fundamental period of 2.447 sec and cumulative

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modal mass participation ratio of 90.45 % by mode 4 were observed. While, in the Y-direction, a fundamental period of 2.454 sec and cumulative modal mass participation ratio of 90.61 % by mode 5 were observed. Although, the mass participation ratios—in orthogonal directions—exceed the 90% limit prescribed by the ASCE 7-16 (section 12.9.1) [19], the computed period found to be approximately 4.8 times larger than the code's approximate period of Ta=0.514 sec in accordance with ASCE 7-16-Equation 12.8.7. The large difference reflects the high flexibility of the ribbed slab. Base shear was evaluated using equivalent lateral force (ELF) and response spectrum analysis (RSA) methods. A value of 499.51 kN was obtained in both principle directions. This proves that V_{RSA}/V_{ELF} =100% >85% satisfying the requirements of ASCE 7-16 (section12.9.4).

Story drifts were evaluated using the load combination 1.3D+L+EQ/RS applied separately in both orthogonal directions. A similar drift behavior was observed. The maximum elastic inter-story drift ratios were found as 0.268% in the X-direction and 0.285% in the Y-direction. Both values occurred the 5th floor indicating that the upper mid-height of the structure is subjected to the largest lateral deformation under the seismic loading. Inelastic behavior was estimated by amplifying the elastic drift values using Cd=2.5. the values obtained were 0.67% in the X-direction and 0.71% in the Y-direction. Both values are below the allowable limit of 2% specified in ASCE 7-16 Table 12.12-1. This suggests that the story drift requirements are satisfied and no excessive deformations are expected under the design-level earthquake. A torsional irregularity check was performed using the ratio of Max-story drift over Aver-story drift. The maximum ratio was calculated as 1.06 at the 5th floor, which is lower than the code threshold of 1.2 according to ASCE 7-16 section 12.3.2.1. this means that the structure has negligible torsional irregularity and the seismic response is dominated by transition rather than rotation.

Despite satisfying drift and torsional requirements, a critical deficiency was identified in the strength design checks in orthogonal directions. A total of 58 beam elements failed under the combined shear-torsion interaction check, evaluated in accordance with ACI 318-19 (section 22.7.7.1). The failed beams were distributed between the first and seventh floors, with no failures recorded at floor 8. In floors 1 through 5 f, failures occurred in both edge and interior beams, with D/C ratios ranging from 1.02 to 1.18, reflecting the accumulated effect of simultaneous shear and torsion's demands in the lower portion of the structure. floors 6 through 7, failures were concentrated in edge beams, with D/C ratios of 1.02 to 1.05, indicating that torsional demand becomes the governing stress component as height increases and direct shear diminishes.

This height-dependent shift in failure mode carries an important structural interpretation, lower floors accumulate both shear and torsional demands from the full height of the structure above, making them susceptible to combined stress exceedance across a wider set of elements. Upper floors, by contrast, carry lower shear forces but remain vulnerable to torsional effects, which selectively overstress edge beams due to their peripheral position in the floor plan. This behavior is consistent with semi-rigid diaphragm action of the ribbed slab system which amplifies torsional effects as lateral loads propagates upward through the structure, concentrating torsional demand at the perimeter framing rather than distributing it uniformly across all beams.

Torsional shear stress was found to govern over direct shear stress in all flagged elements. Direct shear produces a diagonal stress field across the beam web, whereas torsion generates a circulating stress patter around the section perimeter, inducing higher combined stress intensities even at locations where direct shear alone remains within permissible limits. This distinction confirms that the root cause of failure in this configuration is torsional in nature, and that section adequacy cannot be restored through stirrups redesign alone without first addressing the source of torsional demand.

Column PM ratios were evaluated for critical load combinations and were found to satisfy the ACI code strength requirements of the limit of 1.0, however, a detailed review reveals a variant pattern in how seismic demands affect the columns response. On the first floor, interior columns reached a PM ratio of 0.82 under the load combination 1.3D+L+EQx, reflects high axial stress combined with seismic demands in the X-direction. At the floors 2 to 3, interior columns remained critical where values of 0.62 to 0.72 were observed under the load combination 1.3D+L+RSx. From floors 4 to 8, edge and corner columns became critical under the load combination 1.3D+L+EQy, with PM ratios ranging from 0.65 on the 4th floor up to 0.87 on the 8th floor. This increase at the upper-floor edge columns is likely attributed to force redistribution from failed edge beams at different floors along the structure height.

Although all PM ratios remain below 1.0, values approaching 0.9 indicate a substantial reduced reserve capacity. This condition may become critical under increased seismic demands or sudden changes in the structure load path. Furthermore, such higher ratios are critical in columns because as they must maintain stability during lateral seismic excitation to prevent global collapse.

C. Seismic Response of the IMF-Ribbed System (Case 3)

In case 3, the moment-resisting system was upgraded from OMF to IMF while retaining the ribbed slab system. Measurable improvements in seismic response were observed, although some deficiencies remained despite the upgrade. The base shear calculated via RSA dropped to 489 kN in Case 3 compared to 499.51 kN in Case 2. This 2.1% reduction is a regulatory compliance feature stemming from the deployment of a higher response modification coefficient (R = 5 for IMF vs. R = 3 for OMF) prescribed by ASCE 7-16. Crucially, this reduction in design forces does not represent a decrease in physical ground motion demand. Instead, it reflects a code-permitted allowance that relies explicitly on the superior ductile detailing, continuous load paths, and cyclic energy dissipation capacity inherent to IMF as required by ACI 318-19 Chapter 18. The dynamic response of the structure remained very similar between cases 2 and 3, specifically the fundamental period was calculated as 2.45 sec and modal mass participation ratio exceeded the required code limit of 90% by mode 4 in the X-direction and mode 5 in the Y-direction. These results demonstrate that while the structural detailing changed, the overall flexibility ribbed slab system dominates the structure's vibration characteristics.

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The maximum elastic inter-story drift in the Y-direction was obtained as 0.27% at the 5th floor under the load combination 1.3D+L+EQy and inelastic drift was 1.24% using the implication factor of Cd=4.5. this value below the threshold 2% prescribed by the code. Additionally, torsional irregularities ratios were calculated as 1.06 in the Y-direction and 1.02 X-direction. both values are well lower than threshold 1.2 defined by the code confirming that negligible torsional response. These results confirm case 3 representing an improvement over case 2 in terms of drift control.

The most noticeable improvement was observed in the number of beam failures—evaluated through the combined shear-torsion D/C ratio per ACI 318-19 (section 22.7.7.1)—which dropped from 58 in case 2 to 29 in case 3, a reduction of approximately 50%. The maximum D/C ratio decreased from 1.18 to 1.13 in the upper floors, while lower floors retained rations in the range of 1.02 to 1.18, confirming that the frame upgrade produced an obvious relief in the upper portion of the structure compared to the lower floors. The beam failure distribution pattern along the structure height remained broadly consistent with case 2, though with a reduced intensity. In the upper mid-storeys –floors 6 to 7, failures were mainly concentrated in edge beams, with D/C ratios of 1.02 to 1.13, indicating that torsional effects continue to influence perimeter elements due to the inherent flexibility of the ribbed slab diaphragm. In the lower floors –1 through 5—both edge and interior beams recorded D/C ratios up to 1.18, where accumulated shear and torsion demands continue to govern, this partial improvement is attributed to the IMF detailing requirements specified in ACI 318-19 (chapter 18), where closer spacing of transverse reinforcement enhanced the shear resistance of individual elements. However, since the ribbed slab was retained, the underlying torsional eccentricity dividing the combined stress demand was not fully resolved, limiting the overall effectiveness of the frame upgrade alone.

Moreover, column demand-capacity ratios demonstrate a remarkable enhancement, i.e., the highest ratio dropped from 0.87 in case 2—observed at 8 floor-edge columns—down to 0.83 in case 3—found at 1 first floor-interior column. All columns ratios continue below the ACI code limit of 1.0 meaning no column failed. Additionally, the controlled load combination changed from 1.3D+L±EQy at the upper floors in case 2 to 1.3D+L±EQx at the lower floors in case 3. This shift indicates that the upgraded frame system moves the structure demands toward the lower levels and highlight the response in the X-direction.

D. Seismic Response of the IMF-Solid Slab System (Case 4)

Case 4 employed an IMF with a solid slab configuration. Modal analysis was performed and an approximately similar fundamental period of 1.60 sec were obtained for orthogonal directions. These values are approximately shorter than those observed in the cases of ribbed slab configuration with OMF and IMF frames, reflecting a much stiffer structural system response. The max-elastic inter-story drift was 0.127% at the 5th floor under the load combination 1.3D+L+EQy, which was amplified to 0.572%. this value stays well below the code limit pf 2% and hence, drift is not a concern. Torsional irregularity ratio was calculated as 1.09 which falls below the threshold of 1.2 and indicates that torsional irregularities are negligible. The most importantly that all beam shear-torsion failures were completely resolved. Also, base shear was calculated as 425 kN which is approximately 13% lower than base shear in case 3 (489 kN) and 15% lower than base shear in case 2 (499 kN). This reduction occurs due to the shorter fundamental period of the stiffer solid slab configuration. Additionally, a clear improvement was observed in column PM ratios, where recorded peak values across the structure were decreased, for instance, the maximum PM ratio of 0.83 previously recorded in cases 2 & 3 for an interior column on the first floor was reduced to 0.68 in case 4. This reduction shows lower axial and flexural demands on the column broadly. As a result, a greater reserve capacity is provided for the columns and ensure more safety against global failure. To compare the seismic behavior of the studied cases, the obtained results are presented using response curves demonstrated in Figures 4 & 5 and tabulated in Table 1. The curves visualize the variation in structural response along the building height. This comparison helps identify the effect of each structural configuration.

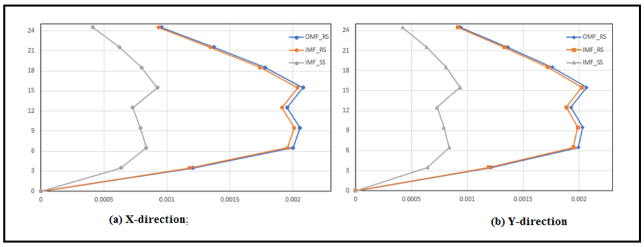

Fig. 4. Story drift response: (a) X-direction, (b) Y-direction.

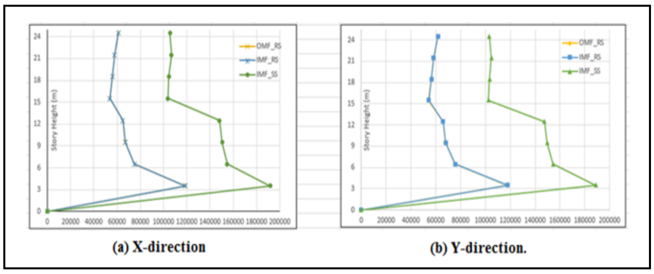

Fig. 5. Story Stiffness profile in (kN/m): (a) X-direction, (b) Y-direction.

V. CONCLUSION

This study examined the seismic behavior of a typical eight-story reinforced concrete residential building in Tripoli, Libya, through a comparative parametric evaluation of four structural configurations designed in accordance with ACI 318-19 and ASCE 7-16. The investigated configurations progressed from a conventional ordinary moment frame with a ribbed slab system—representative of current regional practice—through to an intermediate moment frame coupled with a solid slab system, allowing a systematic assessment of the performance gains achievable at each upgrade level. Given the assumptions regarding the analyzed prototype, the findings indicate that current construction practices in Tripoli significantly underestimate seismic vulnerabilities. Furthermore, the integration of intermediate ductile detailing with solid floor systems presents a highly effective and compliant structural strategy. This approach provides a

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valuable technical benchmark for future updates to regional building provisions.

VI. RECOMMENDATIONS

The results of this research indicate that structural engineers, designers, and relevant municipal authorities in Libya are encouraged to transition from conventional ordinary moment frames combined with a one-way ribbed slab system toward code-compliant lateral force-resisting systems for mid-rise residential construction in regions of elevated seismicity. Consequently, updating the regional building regulations to align with modern codes represents an important step toward improving the built environment in the country. However, the present investigation is limited to linear elastic analysis of one prototype of an eight-floor building, and therefore, future work should extend this research into the nonlinear behavior domain using advanced assessment methods to provide a more realistic characterization of collapse capacity and ductility reserve for conventional structural systems in Tripoli, Libya.

TABLE I. MODAL AND RESPONSE SUMMARY FOR STUDIED CASES

Performance Metric	**Case 1: Baseline (Gravity Only)**	**Case 2: OMF + Ribbed Slab**	**Case 3: IMF + Ribbed Slab**	**Case 4: IMF + Solid Slab**
Frame System	OMF	OMF	IMF	**IMF**
Slab/Diaphragm	Ribbed (Membrane)	Ribbed (Semi-Rigid)	Ribbed (Semi-Rigid)	**Solid (Rigid)**
Fundamental Period (sec)	N/A	2.477	2.454	**1.596**
Global Stiffness (story1) (kN/m)	High (Gravity)	~115,000	~115,000	**~190,000**
Max Elastic Drift	N/A	0.00285	0.00276	**0.00127**
Max Inelastic Drift	N/A	0.713%	1.242%	**0.572%**
Design Base Shear (kN)	N/A	499.51	489.26	**425.00**
Beam Failures (O/S 45)	0	58	26	**0**
Max Column P-M Ratio	0.44 (Axial)	0.87	0.83	**0.68**
Compliance Status	**Serviceability Fail**	**Strength Fail**	**Strength Fail**	**FULLY PASS**

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