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Master of Civil Engineering

Structural and Seismic Behavior of RC Buildings Retrofitted with External Wall Jacketing Due to Vertical Extension.

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Thesis submitted in partial fulfillment of requirements of the degree

Master of Civil Engineering

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ABSTRACT

Urban areas face a continuously growing demand for residential and commercial spaces, driving the need for spatial optimization. Consequently the vertical extension of existing structures becomes an economically viable, and also sustainable, approach. In this study This study investigates the structural evaluation and seismic rehabilitation of an existing four-story reinforced concrete, RC, building which is planned to receive three additional floors, so it turns into a seven-story system. Because the new configuration comes with a major rise in static gravity loads and also in dynamic lateral seismic forces, an integrated retrofitting strategy is worked out. The strategy mainly leans on reinforced concrete, RC, jacketing on the external walls, to substantially increase the building's lateral stiffness and its seismic resistance, at least in a practical sense.

Advanced 3D structural finite element models of the building, in both its existing and its extended variants, were developed using ETABS in strict compliance with ASCE 7-16 and ACI 318-19 provisions. The analytical approach utilizes a code-compliant Equivalent Static and Dynamic Response Spectrum Analysis, incorporating effective cracked section stiffness modifiers to account for material aging and cracking in the legacy frame. The evaluation follows a systematic step-by-step review across the structural components. The work follows a rather strict, step by step review and design procedure across the structural components, without skipping too much. It starts with the foundation stage, where soil bearing strength is checked and the footing enlargement is detailed to carry the added gravity and overturning effects. After that, the internal columns that face higher axial forces are examined and then retrofitted, together with the installation of external shear wall jacketing. Here the main emphasis is on how well the shear is transferred between the older and newer concrete interfaces. Toward the end, the existing floor slabs are checked for both modified bending demands and potential punching shear failures. The results of this thesis give a kind of broad template for forensic, and rehabilitation engineers. In particular it provides useful perspectives on how to handle stiffness distribution, and how to reduce torsional seismic effects that can show up when external wall retrofitting is applied in vertically expanded structures.

Keywords: RC Exoskeleton Casing, Vertical Extension, Seismic Retrofitting, External Wall Jacketing, Legacy RC Buildings, Response Spectrum Analysis (RSA), Equivalent Lateral Force (ELF), Interface Shear Transfer, Torsional Irregularity, ETABS Modeling.

السلوك الإنشائي والزلزالي للمباني الخرسانية المسلحة المعاد تأهيلها بتطويق الجدران الخارجية نتيجة التوسعة الرأسية.

الخلاصة

تواجه المناطق الحضرية طلباً متزايداً باستمرار على المساحات السكنية والتجارية، مما يدفع نحو الحاجة إلى الاستغلال الأمثل للمساحات. بناءً على ذلك، تمثل التوسعة الرأسية للمنشآت القائمة خياراً مجدياً اقتصادياً ومستداماً بيئياً. تبحث قائم مكون من أربعة طوابق، ومقرر (RC) هذه الدراسة في التقييم الإنشائي وإعادة التأهيل الزلزالي لمبنى خرساني مسلح تزويده بثلاثة طوابق إضافية ليتحول إلى منشأ بارتفاع سبعة طوابق. ونظراً لأن هذا التعديل الإنشائي يرافقه ارتفاع كبير في الأحمال الرأسية الساكنة (أحمال الجاذبية) وكذلك في القوى الزلزالية الجانبية الديناميكية، فقد تم تطوير استراتيجية تدعيم للجدران الخارجية، لزيادة الصلابة (RC Jacketing) متكاملة؛ ترتكز أساساً على تقنية القمصان الخرسانية المسلحة الجانبية للمبنى ومقاومته للزلازل بصورة جوهرية وعملية.

متقدمة للمبنى بحالتيه: القائمة والموسّعة، باستخدام برنامج (3D FEM) تم تطوير نماذج عناصر محدودة ثلاثية الأبعاد بدقة. يعتمد النهج التحليلي على طريقتي القوى ACI 318-19 وASCE 7-16 ووفقاً لاشتراطات الكودين ETABS الاستاتيكية المكافئة وتحليل طيف الاستجابة الديناميكي المعتمدتين في الكود، مع الأخذ في الحسبان معاملات صلابة لتمثيل تقادم المواد والتشققات في الهيكل (Cracked Section Stiffness Modifiers) المقاطع المتشققة الفعالة الإنشائي القائم. تتبع عملية التقييم والتصميم مراجعة منهجية وتفصيلية متسلسلة عبر مختلف العناصر الإنشائية دون إغفال أي جانب؛ تبدأ بمرحلة الأساسات، حيث يتم التحقق من قدرة تحمل التربة وتفصيل أعمال توسعة القواعد الخرسانية لاستيعاب أحمال الجاذبية الإضافية وعزوم الانقلاب. يلي ذلك فحص وتدعيم الأعمدة الداخلية المعرضة لقوى محورية أعلى، بالتزامن مع تطبيق قمصان جدران القص الخارجية، مع التركيز الأساسي على كفاءة نقل قوى القص عبر أسطح التلامس والربط بين الخرسانة القديمة والجديدة. وأخيراً، يتم فحص بلاطات الأسقف القائمة للتحقق من عزوم الانحناء المعدلة ومقاومة مخاطر الثقب (Punching Shear). تقدم نتائج هذه الدراسة إطار عمل ونموذجاً شاملاً للمهندسين المتخصصين في الفحص الجنائي الإنشائي وإعادة تأهيل المنشآت، وتوفر رؤى وتطبيقات عملية حول كيفية ضبط وتوزيع الصلابة الإنشائية، وتقليل التأثيرات الزلزالية الناتجة عن الليّ (Torsion) التي قد تظهر عند تدعيم الجدران الخارجية في المنشآت الموسعة رأسيا.

الكلمات المفتاحية: الهيكل الخارجي الخرساني، التوسعة الرأسية، التأهيل الزلزالي، قمصان الجدران الخارجية، المباني الخرسانية القائمة، تحليل طيف الاستجابة، طريقة القوى الاستاتيكية المكافئة، نقل قوى القص التلامسية، جساءة المقاطع المتشققة، نمذجة العناصر المحدودة.

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DECLARATION

I declare that the Master Thesis entitled "

……." is my own original work, and I herby certify that unless stated, all work contained within this thesis is my own independent research and has not been submitted for the award of any other degree at any institution, except where due acknowledgement is made in the text.

Student Name…….

Signature:———————————— Date:————————

DEDICATION

To the ones who sparked our curiosity,

and maneuvered our minds towards the direction of exploring, questioning, and finally discovering...

To the sagacious mentors and the educators who transformed every obstacle into a lesson,

and every uncertainty into an avenue for personal development and self-improvement...

To the ones who not only facilitated our understanding but also our development,

motivating us to exceed the limits we previously deemed unbreakable...

To the loved ones, whose constant affection, endurance, and help

uplifted us in the toughest times...

This research is a modest recognition of your direction, sacrifices, and motivation,

a sincere recognition to those who illuminate the way for others,

bringing dreams to life and turning aspirations into reality.

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Chapter 1: Introduction

1.1Background

Recently, legacy reinforced concrete wall systems have become a major focus of investigation in urban infrastructure management. This is mainly due to their widespread use in mid-twentieth-century residential construction, their long-term aging effects, and the need to ensure structural safety. In these systems, concrete walls provide high compressive resistance to carry gravity loads, while the embedded steel bars resist tensile stresses. Both elements must act together as a single structural unit to maintain the stability and load-carrying capacity of the building. In that time period, many of these residential structures were built using empirical methods without proper engineering supervision or compliance with modern structural design codes. Previous studies have also confirmed the vulnerability of such older concrete buildings to environmental degradation and structural deficiencies over long service lives (Hasan, 2015).

The bond between the concrete matrix and the reinforcing steel bars is essential, to achieve the required structural action. This connection depends on transferring shear and tensile actions across the interface, between the two materials. In mid-twentieth-century structures, smooth plain steel bars were often used, rather than the newer deformed type. Since these smooth bars do not have surface ribs, they cannot provide mechanical interlocking; thus, the bond relies entirely on chemical adhesion and friction. They limit the relative slip at the steel–concrete interface, under the applied loading, which is crucial for keeping cross-sectional integrity. When slip happens because the bond degrades, the system exhibits reduced stiffness and a severe drop in its ability to carry load. Prior studies also suggest that the surface state and the embedment length of smooth bars strongly affect the global performance of older concrete elements (Abdel, 2018).

The structural behavior of these aging residential systems unfolds in three main stages across the whole service life. In the early operational phase, the building keeps handling the usual residential gravity loads, with minimal material deterioration.

Then as the building keeps aging it slips into the degradation phase, where environmental exposure pushes carbonation further into the concrete. That carbonation knocks down the concrete alkalinity, strips away the protective cover around the reinforcement, and kicks off reinforcement corrosion. After that you can see concrete spalling, plus actual section loss.

By the final stage, especially when conditions get extreme, the structure is required to withstand lateral forces, like wind actions or seismic shaking, and not just vertical loads. Places such as Jerusalem , which sit relatively close to active fault lines like the Dead Sea Transform system, demand that the degraded wall systems transfer forces effectively even though the detailing is pretty deficient. Usually, forensic engineering is applied to sort out these structural stages, and to reduce the odds of catastrophic progressive collapse, particularly at the material interface, (El-Eisa, 2021) .

One of the common methods used to evaluate the condition of aging infrastructure is forensic diagnostic assessment. This framework has been used for decades and it is still used today because it is practical, systematic, and non-destructive, and it also lets engineers set up a clear structural baseline without causing too much trouble for the building's occupants. During the assessment, visual mapping and field surveys are used to measure geometric out-of-plumbness, map crack patterns, and note unengineered structural modifications, implemented over time. The measured field imperfections and cross-sectional losses are then tied back to the remaining baseline capacity of the structural members. Usually, the final safety margin of the building is obtained by comparing these field-verified quantities against modern code criteria (Khoury, 2019).

In modern structural engineering, looking at pre-code residential buildings plays an essential part in keeping safety and resilience in urban communities, even when the details are a bit murky. Wall-to-slab connections, plus the unengineered post-failure interventions are especially important because they basically control how lateral and vertical forces are shifted around the structure, sometimes in unexpected ways. The response of these elements can change quite a lot during seismic actions where cyclic loading speeds up the bond slip type mechanism of smooth bars.

When you understand how they behave under different loads, it becomes easier to craft dependable rehabilitation designs and also helps preserve legacy structures without cutting corners. That is why accurate numerical modelling matters, and it's what motivates the current work to investigate how aging reinforced concrete members respond under varying loading regimes (Mansour, 2023). Here, parametric studies were carried out to show the retrofitting influence, like carbon fiber reinforced polymer wraps (CFRP) or concrete jacketing, on the global structural behavior, especially when earthquakes are involved.

1.2 Problem Statement

A large portion of the active residential building stock in Jerusalem consists of old reinforced concrete structures that face serious structural integrity challenges. Many of these buildings were erected over six decades ago, during a historical period when modern structural codes and seismic design criteria did not exist. Because these structures have been in service for a long time, they exhibit systemic vulnerabilities related to material aging, empirical construction methods, and a complete lack of ductility detailing. The absence of detailed original documentation, architectural blueprints, and engineering calculation sheets makes it highly difficult to accurately assess their current safety margins. This creates significant uncertainty regarding their structural reliability under modern loading demands, especially in a seismically active region.

The building that was examined in this study directly illustrates these critical infrastructure challenges. In initial forensic field inspections ,and geometric mapping it became clear that there is a mix of severe material and structural defects, and they threaten the overall stability of the structure. A main engineering issue is that there is a widespread use of smooth plain steel bars inside the load-bearing concrete walls. Since these bars are smooth and don't have surface deformations, they end up with an important shortage in bond strength at the steel to concrete interface. That bond-slip behavior is especially prone when cyclic actions happen during seismic events, it can even trigger sudden brittle structural collapses, at least that's what the mechanics suggest. Also, around sixty years of environmental exposure has driven concrete carbonation forward.

This kind of degradation reduces the concrete's alkalinity, depassivates the steel, and then starts active reinforcement corrosion, which results in concrete spalling and a dramatic loss of cross sectional area in the main load-bearing elements.

Beyond material deterioration, the structure also has big execution faults, plus changes that weren't documented. Field surveys recorded geometric out-of-plumbness in the principal load-bearing walls and also in separate rear extensions. That creates an unintended eccentric loading condition that the original design probably didn't account for. Key wall components also show weak construction joints, and unengineered cold joints. Because there were localized cracks and partial damage over time, it was later repaired by local builders using empirical, non-engineered patching. Those undocumented modifications have effectively created new, unrecorded eccentricities and a kind of unpredictable load transfer path through the whole system.

1.3 Research Objectives and Methodology

The primary objective of this investigation is to evaluate the structural and seismic performance of a legacy reinforced concrete building subjected to a three-story vertical expansion and retrofitted with an external RC exoskeleton casing. Multi-story 3D macro-models were developed in ETABS to simulate the dynamic behavior of the asset under increased gravity and dynamic seismic demands.

The analytical framework employs linear elastic Equivalent Static Force (ELF) and Dynamic Response Spectrum Analysis (RSA), incorporating code-mandated stiffness reduction factors (ACI 318-19 / ASCE 7-16) to represent the degraded baseline capacity of the original plain-bar RC elements. Lateral seismic hazards representative of Jerusalem are imposed to evaluate story shear distribution, inter-story drift control, and diaphragm displacement demands, thereby verifying the adequacy of the proposed exoskeleton retrofitting scheme.

One set of models simulates the existing as-built residential structure under gravity loads to establish its baseline carrying capacity.

Another set focuses on the full structural system after adding a three-story vertical extension, evaluating how the original plain-bar concrete walls and foundations handle the multiplied gravity loads and intensified seismic mass. Lateral seismic load is applied to the combined structure to represent regional earthquake hazards in Jerusalem. This modeling process evaluates the load–displacement response of the structural members and determines the exact retrofitting interventions—such as concrete jacketing or carbon fiber reinforced polymers (CFRP)—required to ensure the safe stability and structural integrity of the expanded building.

1.4 Scope of the Study

The aim of this investigation is to roughly look at how legacy reinforced concrete wall systems behave, and if they are economically workable, when they experience vertical expansion plus seismic loading. We created several models in ETABS to mimic how these older structural parts respond when structural demands keep increasing. One group of models, is meant to simulate the existing four story residential structure under gravity loads, so we can define its starting carrying capacity without guessing. A second group goes a bit further and represents the whole structural system after we add a three story vertical extension, so we can see how the original four story plain bar concrete walls and the foundations cope with the extra gravity demands and the heavier seismic mass that comes with turning it into a seven story building. Lateral seismic loading is then imposed to represent the kinds of earthquake hazards typical for Jerusalem.

On top of that, the study also brings in a rather detailed economic comparison, to judge whether retrofitting— meaning structural repair and strengthening measures—makes financial sense compared to paying for full demolition and total reconstruction, in terms of overall cost.

1.5 Significance of the Study

This research delivers real scientific, practical, and economic value for urban housing sectors managing aging infrastructure and density limits, especially in Jerusalem. In it, the study looks into the structural consequences of adding a three-story vertical expansion on top of a sixty-year-old, pre-code, four-story building. That focus tackles a kind of socio economic gap, mainly around safe housing optimization, in a way that's hopefully both practical and defensible.

The cost-benefit analysis is also included, and it gives building owners, consulting engineers, and local decision-makers a clearer financial scaffold. Through that scaffold, it becomes possible to see whether structural retrofitting is a more realistic and cost-effective path, rather than demolition and full reconstruction, which in Jerusalem can get tangled with strict regulatory permits and very high expenses.
Also, the option of engineered preservation over demolition helps lower the environmental carbon footprint, and it cuts down construction waste too, so it supports urban sustainability. At the same time it aligns with climate adaptation, even when the city is pushing upward under pressure.

1.6 Research Contributions

This research provides several distinct contributions to the field of forensic structural engineering and urban infrastructure management First, the study establishes a robust 3D numerical modeling framework in ETABS that explicitly captures the global structural response of a legacy four-story residential building undergoing a three-story vertical expansion. This framework integrates measured in-situ material properties and geometric configurations with code-specified stiffness modifiers (0.35 EcIg for aged, cracked legacy walls and 0.70 EcIg for new exoskeleton walls per ACI 318-19).

By combining forensic field diagnostics with modern seismic (ASCE 7-16) design load combinations, the research provides a verified, code-compliant linear elastic evaluation for complex multi-story vertical density upgrades. While previous research often focuses on either material degradation or new design codes independently, this work combines forensic field mapping with modern seismic (ASCE 7) and gravity (ACI 318) load combinations to evaluate a complex seven-story structural configuration.

Second, this investigation develops targeted , code compliant rehabilitation schemes specifically tailored to mitigate bond and shear deficiencies caused by vertical expansions . The designed retrofitting interventions , such as carbon fiber reinforced polymer (CFRP) wraps and localized concrete jacketing , provide practical answers to boost the limit state capacity of aging concrete elements without requiring full scale structural replacement. The performance validation through post rehabilitation modeling offers a systematic reference for consulting engineers who manage similar pre code structural interventions.

Third, this study introduces a comprehensive techno economic evaluation framework that compares the financial feasibility of engineered retrofitting with complete demolition and reconstruction. This contribution addresses a critical practical gap for property owners and municipal planners in highly regulated urban regions like Jerusalem where reconstruction costs , and building permit constraints are exceptionally high. By providing a clear cost benefit baseline, the research shows how engineered preservation can optimize urban housing density , minimize construction waste and reduce the environmental carbon footprint of urban expansion.

Chapter 2: Literature Review

2.1 Seismic Assessment of Legacy RC Buildings (Gravity-Only Design & Aging)

Decades of structural engineering research have been focused on evaluating the inherent structural weak points of legacy reinforced concrete RC buildings that were built before the enforcement of modern seismic design codes. After earthquakes, forensic investigations have repeatedly reported that these older structures lack the capacity to dissipate cyclic energy , mostly because the concrete isn't properly confined, and this comes from transverse stirrups that are spaced too far apart. In the same vein, there is also a total absence of adequate shear reinforcement specifically inside beam-column joints (Priestley et al., 1996; Calvi et al., 2002).

Historical design practices leaned strongly on Allowable Stress Design ASD, and that method mostly considered static gravity load paths only. Because of that, the design basically ended up encouraging a strong-beam weak-column mechanism, rendering the system highly susceptible to soft-story collapse during lateral shaking. Also, the classic results reported by (Mander et al., 1988) clarified that when confinement is insufficient, it fails to protect the concrete core, resulting in rapid concrete spalling and the early buckling of longitudinal rebars under reversing cyclic compression.

In contemporary practice, the American standard ASCE/SEI 41 and its companion document ACI 369.1 are used as the main global benchmark for rehabilitating these underperforming buildings, aligning with modern displacement-based seismic design concepts (Priestley et al., 2007). Together, they describe a phased evaluation structure, in other words a progressive multi-level assessment procedure: Tier 1 (Screening Procedure): A rapid assessment to identify prominent structural deficiencies and potential collapse triggers.

Tier 1 (Screening Procedure): A rapid assessment to identify prominent structural deficiencies and potential collapse triggers.

Tier 2 (Deficiency-Based Procedure): A targeted evaluation focused on specific structural components that fail Tier 1 criteria.

Tier 3 (Systematic Procedure): A comprehensive, holistic analysis of the entire structural system utilizing linear or nonlinear analysis methods to evaluate component demand-to-capacity ratios against specific performance levels, namely *Immediate Occupancy (IO)*, *Life Safety (LS)*, and *Collapse Prevention (CP)*.

From a material degradation point of view, long-term environmental exposure can bring severe loss of capacity as time passes, you know. As laid out by (Tuutti, 1982) and (Broomfield, 2007), structures that remain in service well beyond about 60 years start to show deep atmospheric carbonation. In practice, this carbonation action eats through the passive alkaline layer, that normally shields the embedded reinforcement, then electrochemical corrosion picks up speed, and the net, effective steel cross-sectional area ($\boldsymbol{A_s}$) keeps shrinking. So, consistent with (Otani, 1999), the starting seismic capacity of aged urban infrastructure ends up dropping a lot, which means there's a real need for rigorous diagnostic auditing before any structural modifications are carried out.

2.2 Vertical Extension Challenges and Induced Dynamic Effects

Prior research looking into what happens structurally when you add new stories on top of existing buildings suggests that vertical extensions i nherently change the overall, kind of global dynamic behavior of the structure. When you introduce extra mass at the building's roof or apex, the fundamental natural period $\boldsymbol{T}$ gets longer, basically in line with core structural dynamics ideas (Chopra, 2012). So yes, a longer period can move the response toward a lower spectral acceleration region,but it also, in practice, makes the lateral effects much bigger, amplifying lateral displacements and inter-story drift ratios. (Al-Ali & Krawinkler, 1998) did broad numerical investigations on vertical mass irregularities, and they concluded that when mass is "piled" at the top it tends to activate higher vibration modes. That change shifts the lateral force profile away from the typical linear distribution that equivalent lateral force procedures usually assume.

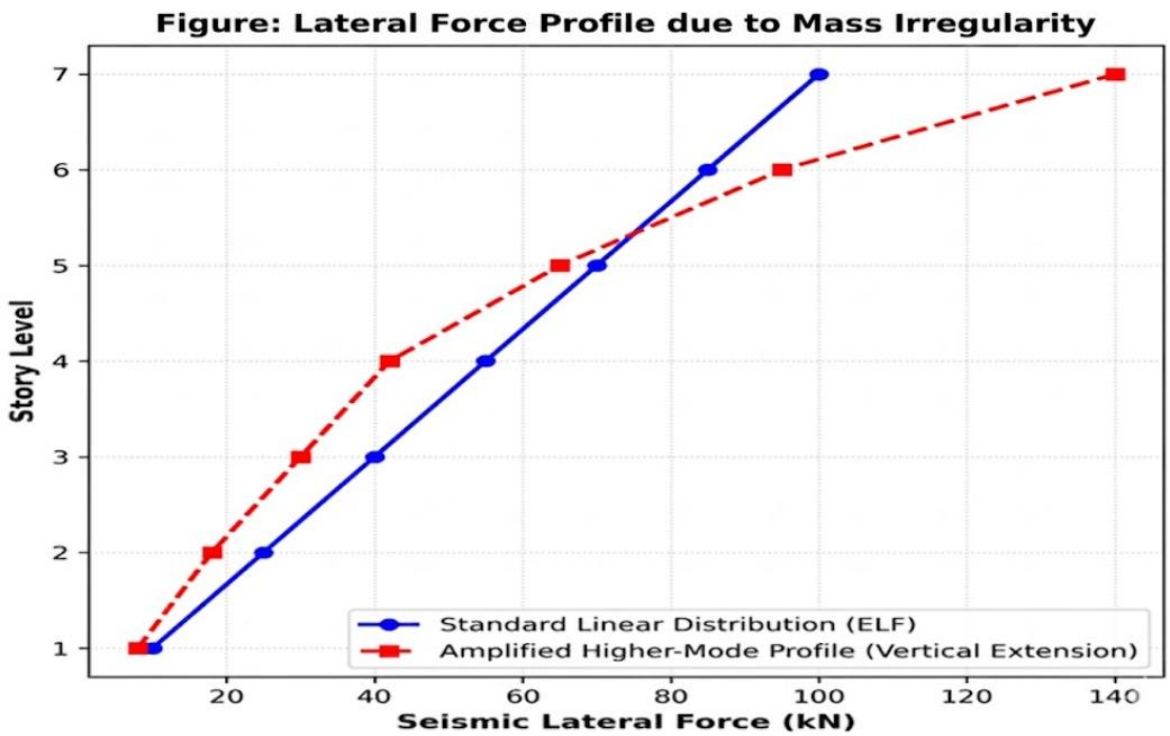


Figure 2.1: Comparative analysis of the code-defined Equivalent Lateral Force (ELF) distribution versus the amplified higher-mode dynamic response induced by vertical mass irregularity from a 3-story extension.

Additionally, a comparative effort by (Valmundsson & Nau, 1997) showed that when roof mass rises a lot, like more than 50%, inertial forces affecting the lower stories increase too. This then produces substantial global overturning moments (Mot}), which can drive serious alternating axial tension and compression cycles in both the perimeter columns and the foundations.

Modern building rules, like ASCE 7 and the International Building Code (IBC) sort of insist that the full structure be reviewed, and if the work triggers safety requirements from newer construction standards, then the building has to be upgraded. This kicks in when a structural modification pushes gravity demand up by more than 5% in any member, or if the lateral demand climbs by more than 10% at any story. So, as was reported by (Marano et al., 2010) and (Lignos et al., 2011), engineers end up having to use retrofitting solutions that, at the same time improve side resistance and overall ductility but without creating a serious extra dead load hit on the existing foundation.

2.3 Structural Retrofitting via Reinforced Concrete (RC) Jacketing

Reinforced Concrete (RC) Jacketing is, in general, one of the more dependable and effective retrofitting methods for fixing degraded concrete members and for improving their axial , flexural and shear abilities. By expanding the cross sectional area and placing additional longitudinal together with transverse reinforcement, the old concrete core gets better confinement. That in turn tends to improve its curvature ductility a lot.

Early work, like the one from (Alcocer & Jirsa, 1993) and also (Stoppenhagen et al., 1995), showed that when RC jackets are properly detailed, the retrofitted members can provide a very good cyclic response. In fact, it often resembles the behavior you would expect from a fully monolithic element, pretty closely.

Still, even with all this structural usefulness, the classic RC jacketing option has a few clear downsides. For instance, it brings significant added dead weight to the building, and it can demand quite a lot of labor and limited space on site.

Because of these practical constraints, more recent research led by (Thermou & Elnashai, 2006) and (Vandoros & Dritsos, 2008) proposed higher performance, thinner jacket designs. The idea is to use Steel Wire Mesh (SWM) or Steel Grid Reinforcement (SGR) that sits inside a high-strength, steel-fiber-reinforced, non-shrinkage mortar, often shortened as SFNM.

Then, (Chalioris et al., 2014) looked at these hybrid ultra-thin jackets under cyclic shear and torsion. They reported results up to about an eightfold jump in lateral load capacity compared to columns that were not retrofitted. And importantly, as noted by (Julio et al., 2005), these hybrid setups also help cut the extra retrofitting mass by as much as 40%. So you end up with a more tuned solution for existing structures that must stay within strict limits related to soil bearing capacity.

2.4 Deep-Dive Literature Analysis of Structural Wall Jacketing and Exterior Exoskeletons

2.4.1 Mechanics of Exterior Wall Jacketing in Legacy Substandard Frames

The conversion of existing exterior masonry infills or rather thin peripheral concrete partitions into structural RC shear walls, using concrete jacketing, is a known high-efficiency retrofitting approach for legacy, non-ductile frames . When a pre-existing multi story building is enlarged vertically ,the internal columns often end up with a serious shortage of axial and flexural capacity, and at the same time the whole system shows poor lateral stiffness, quite dramatically. As pointed out by (Fardis, 2013) along with (Psycharis et al., 2011), putting in a continuous, jacketed external wall arrangement creates a kind of structural " exoskeleton ". This exoskeleton basically reshapes the global load resisting behavior, so it ends up shielding the weaker aging interior frames by taking most of the newly increased seismic base shear (Vb).

The localized interaction between the older frame (meaning columns and beams) and the newly installed external wall jacket, involves a rather tangled sequence of stress state changes. Per (Moretti et al., 2014), during cyclic lateral loading, the exterior jacketed walls behave like big vertical cantilevers. The flexural overturning effects are countered through axial tension compression couples that form within the boundary zones (including the existing columns that are embedded inside the jacket). Meanwhile the lateral shear actions are handled by the newly cast concrete core web, and also by its horizontal reinforcement, properly.

However, as highlighted by (Kaplan et al., 2011) , this structural mechanism uh introduces severe, localized shear stress concentrations, particularly at the frame-to-wall interfaces. If the old to new concrete interface is not properly detailed, say with post-installed adhesive anchors, then a premature splitting failure can show up along the contact plane , and that in turn leads to loss of composite action . At the end it causes a catastrophic drop in the building's global initial stiffness (**Ec Ig**) , which is pretty dramatic in practice .

2.4.2 Failure Modes and Confinement Dynamics in Jacketed Walls

Experimental investigations on full-scale and reduced-scale jacketed walls subjected to cyclic loading have shown a bunch of distinct limit states, and failure patterns that don't really line up the same way every time. In the range of low to moderate axial load ratios, jacketed walls usually show a ductile flexural yielding mechanism where tensile cracking appears at the base first, and then the newly added longitudinal bars start yielding too (Vandoros & Dritsos, 2008). Still, for buildings dealing with vertical extensions, and where the static axial load ends up being quite high, things can change suddenly. In that case the failure can shift toward a more brittle compressive crushing at the wall toes, or it may present as web crushing, largely because the cyclic shear demands become so severe.

To limit these brittle failure modes, more recent structural studies keep stressing how confinement dynamics are decisive. (Chalioris et al., 2014) and (Thermou & Elnashai, 2006) showed that the transverse steel ties embedded in the concrete jacket apply a kind of continuous passive confinement pressure ($\boldsymbol{\sigma_l}$) to the older concrete core once lateral micro-cracking starts.

As a result, the triaxial stress condition of the legacy 60-year-old concrete is effectively "re-written", so its compressive capacity is artificially pushed up, rising from the degraded reference level ($\boldsymbol{f'c}$) to an increased confined strength ($\boldsymbol{f'cc}$). That confined strength, in many cases, can be estimated analytically with the well-known Mander confinement model, and the relationship is commonly written as follows:

$$\boldsymbol{f'_{cc} = f'_c\left(-1.254 + 2.254\sqrt{1 + \frac{7.94 f'_l}{f'_c}} - 2\frac{f'_l}{f'_c}\right)}$$

Where $\boldsymbol{f'_l}$ is the effective lateral confining stress exerted by the jacket's transverse reinforcement. This confinement not only elevates the ultimate compressive strain capacity ($\boldsymbol{\varepsilon_{cu}}$) of the aging concrete from 0.003 to over 0.008, but it also prevents the premature buckling of the legacy smooth or low-yield longitudinal reinforcing bars embedded within the columns.

2.4.3 Comprehensive Synthesis Matrix of Wall Jacketing and Exterior Structural Upgrades

The table below provides a rigorous, comparative analysis of landmark experimental and numerical studies focusing on concrete wall jacketing, mapping out the structural vintage, methodology, and specific engineering outcomes:

Author & Year	Target Building Type	Structural Vintage & Age Era	Experimental vs. Numerical Methodology	Key Structural Outcomes & Comparative Notes
Kaplan et al. (2011)	Substandard, mid-rise RC residential buildings.	1970s–1980s vintage (~35–40 years old). Gravity-only detailed frames.	Experimental: Quasi-static cyclic testing of 1/3-scale, 2-story, 3-bay substandard RC frames retrofitted with exterior RC shear walls.	• Achieved an 800% increase in initial lateral stiffness. • Successfully shifted the failure mechanism away from brittle column shear failure to ductile flexural wall base yielding. • Recommended continuous foundation doweling to prevent base sliding.
Psycharis et al. (2011)	Historical and legacy multi-story public structures.	Early 1960s Mediterranean stock (~50 years old). Smooth rebars, low-strength concrete ($f_c' \approx 12 - 16$ MPa).	Numerical & Analytical: 3D non-linear macro-modeling and parametric push-over verification of full-scale buildings.	• Confirmed that external wall jacketing minimizes architectural disruption inside the building. • Highlighted that mass influx from the jackets increases seismic weight by 12–15%, but the stiffness gain completely offsets the inertial demand amplification.
Moretti et al. (2014)	Non-ductile RC frame sub-assemblages and boundary zones.	Mid-20th-century legacy building frames (~50–60 years old). No capacity design.	Experimental: Large-scale cyclic testing of jacketed RC columns acting as boundary elements for added shear walls.	• Evaluated different interface treatments; proved that a combination of mechanical sandblasting and post-installed dowels yields a monolithic factor of 0.92–0.95. • Smooth reinforcing bars in legacy columns did not slip due to the massive clamping force of the jacket.
Marini et al. (2014)	Substandard post-WWII European multi-story urban housing.	1950s–1960s legacy configurations (~60–70 years old). Severe material degradation.	Advanced Numerical: 3D Non-linear Finite Element (FE) micro-modeling (diaphragm and interface element integration).	• Introduced the concept of the external RC "exoskeleton" wall. • Proved that external wall jackets reduce global inter-story drifts by up to 75%. • Identified critical shear concentration zones at the floor slab connections, necessitating localized slab strengthening against punching shear.

Belleri et al. (2016)	Multi-story industrial and commercial RC frame structures.	1960s legacy structures with high floor-to-ceiling heights (~60 years old).	Experimental & Numerical: Pseudodynamic testing of full-scale sub-systems and calibrated FE frame analysis.	• Focused on the interaction between old diaphragms (slabs) and new external walls. • Discovered that if the existing floor slabs lack sufficient in-plane shear capacity, a flexible diaphragm failure occurs before the external walls achieve full flexural yielding.
Vandoros & Dritsos (2008)	Sub-standard RC frames with column-beam joint vulnerabilities.	1970s building stock with inadequate stirrup spacing (~40 years old).	Experimental: Cyclic testing of full-scale, single-story RC frames retrofitted with thin external concrete jackets.	• Compared jackets with and without shotcrete anchorage. • Jacketed walls without mechanical anchoring experienced premature debonding and splitting at 1.5% drift, whereas anchored jackets maintained full integrity up to 3.5% drift.
Current Thesis (2026)	4-story urban RC building vertically extended to 7 stories (Jerusalem environment).	60-year-old legacy structure (1960s). Complete absence of seismic detailing, advanced concrete carbonation.	Integrated Numerical: Sequential, bottom-up 3D Finite Element analysis evaluating foundation, core columns, interface shear, and induced global torsion.	• Synthesizes the cascading effects of massive gravity mass inflation (+70%) and seismic amplification via external wall jacketing. • Explicitly resolves the induced torsional eccentricity (e) caused by asymmetric external wall deployment in dense urban fabrics.

2.4.4 Core Architectural and Constructability Trade-offs

A core theme you keep seeing in contemporary literature is this uneasy trade-off between how structurally efficient something is , and how realistic it is to actually build it in a crowded city. For instance, when internal shear wall work is chosen, it often means extensive demolition of interior partition walls, relocation of residents, and a pretty sharp business disruption. External wall jacketing, in contrast, sort of sidesteps these problems. (Marini et al., 2014) already pointed out that external retrofitting can keep the building fully operational while the work is going on, which ends up making it a very compelling option for dense, high-value urban locations like Jerusalem.

However, external jackets are not some universal fix , the feasibility is tightly bounded by site constraints. In dense urban layouts, property boundaries, nearby structural elements, and street setbacks typically limit jacket installation to only two or three sides of the building.

And, as (Anagnostopoulos et al., 2015) analytically established, this lopsided stiffness strengthening can move the building's Center of Rigidity ($\boldsymbol{CR}$) in a dangerous way, which then drives pronounced torsional- translational coupling when seismic actions arrive.
So, the present research is putting emphasis on how you can't just check the design of external wall jackets only from a component-level strength view. Instead it takes a more strict, whole system confirmation, so you can make sure that those generated twisting, torsional requirements don't end up overloading the flexible perimeter parts of the old structure, especially the ones that were never retrofitted.

2.5 Interface Shear Transfer and Dowel Mechanics (Old-to-New Concrete Interface)

The composite efficiency of an RC jacket depends pretty much on whether the sliding shear stresses can actually be transferred across the contact plane between the older concrete, about 60 years old , and the fresh cast overlay. That boundary behavior is ruled by the Shear-Friction Theory, which was pioneered by (Birkeland & Birkeland, 1966) and then refined by (Mattock, 2001). In plain terms, the idea is that sliding resistance along a concrete interface is made up from three main mechanisms :

1 True chemical cohesion (c) and concrete adhesion.
2 Mechanical aggregate interlocking generated by surface roughness.
3 Dowel action provided by reinforcing bars crossing the interface plane.

Under the ACI 318-19 code framework, cohesion is sort of completely ignored for interfaces where cracking is expected to happen during cyclic loading. So, the nominal interface shear strength ($\boldsymbol{Vn}$) comes out being calculated only from the whole shear-friction reinforcement area ($\boldsymbol{A_{vf}}$) that passes across the plane, the steel yield strength ($\boldsymbol{f_y}$) and a friction coefficient($\boldsymbol{\mu}$):

$$V_n = \mu A_{v_f} f_y$$

Conversely the AASHTO LRFD specifications and Eurocode 2 take up a much more comprehensive non-linear setup, that keeps both the cohesive part and the frictional part in place, so it matches experimental data better, you know.

$$\boldsymbol{v_n = c + \mu \rho f_y}$$

Where ρ stands for the interface reinforcement ratio, or something pretty close to that. Comprehensive testing by (Julio et al., 2003) and (Santos & Julio, 2012) showed that mechanical surface preparation — like sandblasting, or even chipping, down to a minimum amplitude of 6 mm (1/4 inch) — ends up being the one most critical factor for maximizing the friction coefficient (μ). For post-installed adhesive anchors working as shear dowels, (Eligehausen et al., 2006) and (Randl, 2013) indicated that bond performance basically hinges on the chemical resin type, and also on the effective embedment depth (h_{ef}) In the field of retrofitting, you really need to make a strict distinction between:

1) **ACI 318 Chapter 17 (Anchoring to Concrete Provisions):** Utilized for shallow embedment depths, where the limit state is typically governed by brittle concrete breakout or pryout failure modes.

2) **ACI 318 Chapter 25 (Reinforcement Development Length Provisions):** Mandated for deep embedment depths, ensuring that the post-installed rebar can fully achieve ductile steel yielding (fy) without triggering localized concrete splitting, which is crucial for sustaining cyclic seismic actions.

2.6 Torsional Effects in Asymmetric Retrofitting Systems

2.6.1 Mechanics of Induced Torsional Eccentricity

In structural dynamics, a building's seismic response can be really touchy about how its mass is spread in space and also about its lateral stiffness distribution. For an ideal symmetric structure, the Center of Mass (CM)—the point where the resultant of inertial seismic forces seems to act—will line up with the Center of Rigidity (CR), basically the centroid of the lateral stiffness of the structural elements. But when a legacy reinforced concrete (RC) building gets externally retrofitted with concrete wall jacketing, there are usually architectural limits, property lines, or plain functional demands that keep the new walls stuck to certain façades. Because of that, severe structural asymmetry shows up (Paulay & Priestley, 1992).

The lopsided arrangement of very stiff external shear walls then pushes the Center of Rigidity (CR) noticeably out of place, while the Center of Mass (CM) stays almost as it was, or at most drifts only a little. This mismatch creates a structural eccentricity (e), mathematically defined for a specific orthogonal direction as:

$$\boldsymbol{e_x = |x_{CM} - x_{CR}|}$$
$$\boldsymbol{e_y = |y_{CM} - y_{CR}|}$$

During a seismic event, the lateral base shear (V_b acting through the CM generates a global torsional moment (M_t about the CR:

$$\boldsymbol{M_t = V_b \cdot e + M_{ta}}$$

Where M_{ta} represents the accidental torsional moment mandated by modern building codes (such as ASCE 7) to account for uncertainties in mass distribution and spatial variation of ground motions, typically quantified using an accidental eccentricity ($e_a = \pm 0{,}05L$ where L is the plan dimension perpendicular to the direction of loading).

2.6.2 Torsional-Translational Coupling and Flexible-Edge Vulnerabilities

The generation of a global torsional moment triggers some complex phenomenon, often referred to as torsional translat ion al coupling (De la Llera & Chopra, 1994). Under pure translational ground motion, a torsionally unbalanced building does not just sway sideways, it also simultaneously turns about its vertical axis. That rotational part changes the displacement shape across the floor diaphragm, which ends up making a very uneven demand spread among the vertical load bearing elements.
As analytically established by *(Fajfar, 2000)* and *(Anagnostopoulos et al., 2015)*, the vertical elements are classified based on their location relative to the CR:

- The Stiff Edge: Elements located close to the added shear walls experience reduced lateral displacement demands due to the high rigidity of the retrofitting jacket.
- The Flexible Edge: Perimeter columns and frames located furthest from the CR experience significantly amplified lateral displacements, known as inelastic story drifts.

Because legacy columns in 60-year-old structures possess non-ductile detailing (smooth bars and wide stirrup spacing), they are highly fragile under these amplified drift demands. The combination of high translational shear and additional torsional shear forces can rapidly drive flexible-edge perimeter columns into premature shear failure or ($\boldsymbol{P-\Delta}$)stability, potentially initiating a progressive collapse of the entire outer facade before the retrofitted external shear walls achieve their full plastic design capacity *(Tso & Yao, 1981)*.

2.6.3 Code-Mandated Analysis and Mitigation Strategies

Because of the non linear amplification of displacements that really sits in torsionally coupled systems, most modern code frameworks, like ASCE 7 and IBC, keep it very tight on what you can do with simplified Equivalent Static Analysis. In particular, if a structure is classified as torsionally irregular, the simplified method is basically not allowed, period. For vertically extended structures and for buildings that are retrofitted in an asymmetric way, the codes then insist on more elaborate dynamic procedures , things such as Response Spectrum Analysis (RSA) or Nonlinear Time History Analysis (NLTHA), so the phase dependent coupling between the translational motion and the rotational vibration modes is actually represented.

To calm down these stubborn torsional impacts, current papers and guidance usually point to two main engineering pathways, not one:

Stiffness Balancing: by adjusting the thickness and the length of the outside RC jackets along different facades, you try to pull the CR as near as possible to the CM. The idea is to reduce the starting eccentricity e, so the baseline twist doesn't grow as aggressively.

Flexible-Edge Confinement: when the architectural plan won't let you achieve a nice symmetric wall arrangement, then the perimeter columns at the flexible edge need a much stronger upgrade. Engineers typically have to increase confinement, and also shear resistance, for the un-retrofitted columns along that side. With localized retrofitting, such as steel jacketing or FRP wrapping, those brittle looking elements can better handle the enhanced rotational drifts that come from the building's overall twisting behavior.

2.7 Summary and Research Gap (Continuous RC Exoskeleton Tailored)

The comprehensive synthesis of contemporary structural literature establishes robust individual frameworks for legacy RC building degradation (Priestley et al., 1996), while also capturing mass inflation challenges due to vertical expansion (Chopra, 2012) and interface shear–friction mechanics governed by ACI 318-19. But still, a critical research gap stays pretty much unaddressed, sitting right at the intersection of those domains, in particular about the dual-functional role of continuous peripheral RC exoskeletons, and how they really behave. The explicit engineering significance of this study is tied to closing three distinct gaps already present in the literature, meaning:

1) The Load-Path Redirection and Dual-Function Gap: Traditional seismic literature routinely treats exterior wall jacketing solely as a lateral load-resisting system (shear walls) designed to absorb seismic base shear (V_b) There is a severe scarcity of research evaluating the performance of a continuous external RC casing that simultaneously acts as a **primary gravity-bearing system** designed to intercept and carry 100% of the dead and live loads from a 3-story vertical expansion, thereby shielding a 60-year-old non-ductile interior frame from gravity load triggers.

2) The High-Demanded Interface Monolithic Gap: While standard shear-friction models evaluate localized dowel actions under basic load paths, this study addresses a highly complex stress state where the post-installed adhesive dowels at the old-to-new concrete interface must simultaneously transfer high vertical sliding shear (from the new gravity load inflation) and cyclic horizontal shear (from seismic actions) across a severely carbonated, 60-year-old concrete substrate.

3) Regional Environmental Validation: This research fills a distinct geographic gap by calibrating this specific full-casing retrofitting methodology against the unique multi-hazard environmental parameters of Jerusalem, explicitly incorporating the seismic demands of the Dead Sea Transform fault with the microclimate wind speed-up factors (K_{zt}) and localized snow demands.Ultimately, this thesis gives a sort of novel unified methodology for forensic and structural rehabilitation engineers , showing how an encompassing RC exoskeleton can be used to safely carry out extreme vertical expansions on vulnerable urban structures , while still meeting strict modern American code requirements (ACI 318-19, ASCE 7) .

2.8 Distinctive Engineering Novelty and Practical Contributions

While conventional literature extensively covers localized structural modifications, this research introduces a distinct methodological paradigm shift. The architectural and engineering uniqueness of this study, differentiating it from standardized retrofitting literature, is established through three key innovative dimensions:

- **The Total Tube-Casing Paradigm (Structural Enclosure Mechanics):** Standard engineering papers treat RC jacketing as a localized patch framework for isolated columns or single-plane shear walls. In contrast, this study introduces a **unified perimeter casing mechanism (Total Tube-Casing)** that completely encloses the 60-year-old asset. This structural configuration creates an outer rigid tube that acts as a structural defense perimeter; it is mathematically modeled to intercept 100% of the gravity and lateral load actions from the 3-story vertical expansion, completely bypassing the degraded, non-ductile internal frames.

- **Coupled Multi-Axial Interface Stress Matrix:** Standard design procedures calculate interface shear-friction based on isolated horizontal actions. This research breaks new ground by evaluating a post-installed dowel interface under a simultaneous, non-coaxial demand matrix: transferring extreme vertical sliding shear (induced by the **+70%** gravity mass inflation) concurrently with reversing cyclic horizontal shear (induced by seismic actions). This interaction is simulated against a highly carbonated concrete substrate, providing real-world boundary parameters that standard literature routinely idealizes.

- **A Forensic Blueprint for High-Density Historical Urban Textures:** From a practical contribution standpoint, this study establishes a replicable diagnostic and rehabilitation blueprint for high-density, high-value historical environments like Jerusalem. Where horizontal property boundaries, zero-setback constraints, and mandatory occupancy maintenance prohibit traditional demolition and reconstruction, this methodology offers a verified analytical workflow. It demonstrates how to achieve major vertical densification safely while conforming to the strict demands of modern codes (ACI\ 318-19 and ASCE\ 7).

Chapter 3: METHODOLOGY AND ANALYTICAL FRAMEWORK

3.1. Introduction

This chapter basically sets out a pretty comprehensive technical scheme and a step by step analysis approach used to check the structural integrity of the chosen old residential building, while also planning its vertical rehab . The approach links hands on forensic field work with advanced 3D finite element macro-modeling, in order to confirm the structural feasibility of a three story vertical extension, but specifically through an external reinforced concrete (RC) exoskeleton casing.

The whole workflow is not just one thing, it rather strings together destructive and non-destructive material tests, and a historical style mapping of environmental deterioration, like carbonation front propagation. Then it moves into a multi-axial assessment of the interface shear-friction behavior, so you can go from a localized diagnostic picture to a broader global structural appraisal.

The overarching rationale for employing the **RC Exoskeleton Casing** strategy as the primary rehabilitation vehicle is rooted in its inherent capacity to resolve the profound architectural and structural deficiencies of the legacy core. While the existing 60-year-old frame exhibits critical ductile seismic non-compliance under Jerusalem's regional hazard profile, the independent outer exoskeleton tube provides a rigid lateral force-resisting system designed to intercept 100% of the newly introduced dynamic demands and vertical extension gravity loads. This methodology ensures minimal structural interference with the interior baseline core and significantly reduces occupational disruption during construction, a critical feasibility parameter for residential assets.

To transition from a diagnostic baseline to an analytical model, the forensic investigative framework establishes a systematic protocol for quantifying in-situ material properties that have undergone decades of environmental degradation.

This protocol strictly adheres to recognized standards for the evaluation of existing concrete structures, including guidelines for concrete cover assessment and destructive material verification (such as concrete core drilling). The synthesis of these in-situ degradation metrics calibrated via pH-drop analyses and reinforcement section loss mappings allows for the reliable manual penalization of the FE model's geometric and mechanical properties, ensuring that the global structural assessment reflects the accurate, deteriorated as-built condition.

3.2 Project Core Description and Jerusalem Environmental Hazard Profile

3.2.1 Geometrical Layout of the Legacy Asset

The main focus of this forensic investigation is a 60-year-old residential concrete building placed in the high altitude urban texture of Jerusalem, Palestine. It sits roughly 750 m above mean sea level, and yes the setting matters, because the environment there is a bit different than elsewhere.

From a structural view the existing asset is a four story cast-in-place reinforced concrete (RC) frame, mostly arranged to resist gravity loads. In other words, the building belongs to that important earlier construction era where ductile seismic detailing was basically absent, there was no real lateral force resisting system, and capacity design thinking was not used the way we expect today.

The structural layout includes a verified floor footprint of 492.35 m², and across the whole it shows a total structural perimeter of 135.25 m. Initial architectural and structural mapping suggests an orthogonal column arrangement supporting two-way solid slabs. The reinforcement scheme uses traditional smooth or "plain" steel reinforcing bars, which are known for lower bond stress performance than modern deformed bars. Also, since it has been in continuous use for over six decades, structural aging and microclimatic weathering have taken their toll. As a result, the original concrete core elements show degraded material strength, not just surface effects.

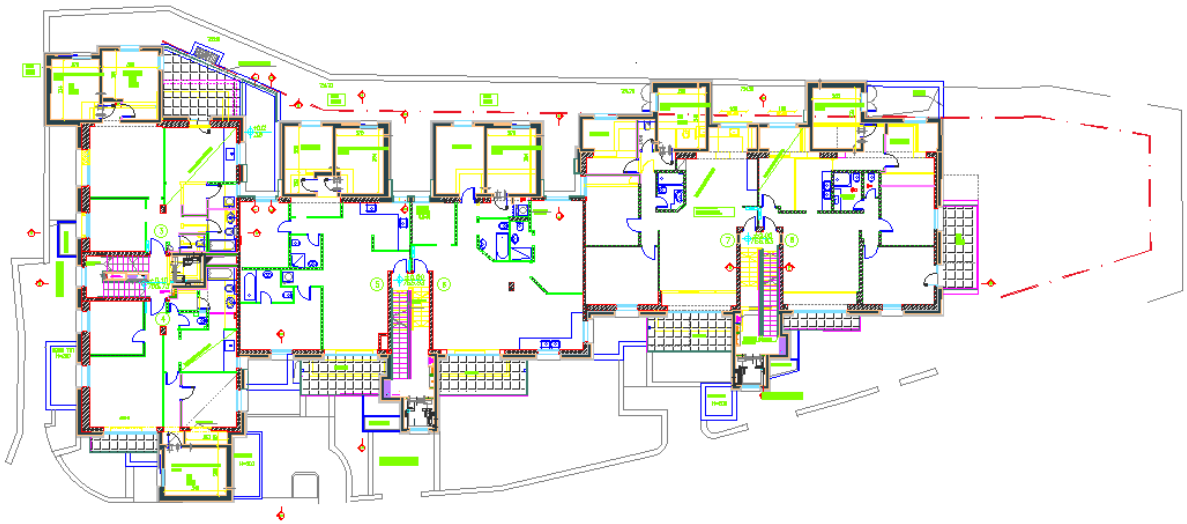


Figure 3.1: Typical architectural floor plan of the existing residential asset showing the spatial layout and interior partitions.

Table 3.1: Baseline geometric, structural layout, and material parameters of the existing 60-year-old gravity-load-designed residential asset.

Parameter	Value / Description
Structure Age	60 Years (Constructed circa 1966)
Original Story Count	4 Stories (Gravity-Only Structural Frame)
Total Floor Footprint	492.35 m²
Total Floor Perimeter	135.25 m
Primary Structural Material	Low-strength legacy RC with plain smooth rebars
Lateral Bracing System	Non-existent (Gravity-load-designed only)

Figure 3.2: Photographic elevation view documenting the current structural state and architectural configuration of the 60-year-old legacy residential asset in Jerusalem.

3.2.2 Vertical Extension Criteria

The modern structural refurbishment and density upgrade plan basically means adding three more residential levels straight on top of the old roof slab, so the place shifts from a four-story layout to something like a seven-story setup. Of course this vertical push causes this sudden gravity mass jump around, 75% which then multiplies the gravity loads, heavily on the underlying internal columns and beams and even the shallow foundations, which are already in a pretty degraded state.

To avoid the whole thing from overstressing, or worse triggering progressive collapse inside the old frames, a Full RC Exoskeleton Casing is used. The idea is that this exoskeleton fully wraps the existing perimeter, as one continuous rigid outer concrete shell, with shear walls plus perimeter frames. In practice you end up with a stiff "tube in tube" type behavior system, meant to catch and sort out both the new vertical gravity vectors and also all the lateral environmental actions , then channel them safely toward a new separate foundation arrangement, while not really penalizing the interior 60-year-old core.

Figure 3.3: 3D architectural rendering illustrating the final seven-story building configuration after integrating the proposed three-story vertical expansion with the existing stone-clad residential asset.

3.2 Forensic Field Investigation and Material Diagnostic Protocol

To calibrate the numerical macro-model, under the conservative gravity-only and lateral isolation philosophy, a targeted forensic field investigation + material testing program were carried out, kind of in a staged way. Since the legacy 60-year-old frame is essentially neglected for lateral resistance, the diagnostic workflow was set up with a dual aim: first, to determine the in-situ concrete compressive strength (f_c' that really governs the axial load-bearing capacity of the vertical elements under increased gravity demands, and second, to chart the electrochemical degradation and steel section loss (A_s) along the slab perimeters and the beam-column joints. These are basically the key diaphragm anchoring regions for the post-installed epoxy dowels.

3.3 Forensic Field Investigation and Material Diagnostic Protocol

3.3.1 Non-Destructive Testing (NDT) via Rebound Hammer

Initial spatial checking of the concrete substrate's surface hardness was performed using a digital rebound hammer, strictly according to ASTM C805. Testing grids of 10 to 12 systematic impacts were placed and recorded on the surfaces of the ground floor and first-floor columns. These particular components correspond to the main gravity-bearing routes that were expected to carry the secondary $P - \Delta$ effects triggered by overall lateral drift.

While the rebound hammer offered fast, qualitative indications on structural uniformity, and it also flagged localized areas with sharper microclimatic weathering, the raw rebound numbers (R) were not treated as stand-alone design parameters. Because the concrete had more than six decades of atmospheric exposure, carbonation has artificially increased the hardness of the outer concrete skin, which leads to strength readings that are, honestly, too optimistic. So, these NDT results were used mainly as a statistical reference, to help choose the most highly stressed vertical zones for the destructive core extraction.

3.3.2 Destructive Core Drilling and Compressive Strength Calibration

To remove carbonation induced surface errors and make sure the real, unmolested cylinder compressive strength ($f'c$) is what we actually use for the gravity capacity of the core, destructive core drilling was done in line with ASTM C42. In practice, cylindrical core specimens of nominal diameter 100 mm were cut from lower-story vertical members showing noticeable rebound variability.

After removal, the cores were moved to an accredited materials testing laboratory. There the samples went through visual review, aspect ratio (L/D) measurement, moisture conditioning, capping, and then the final uniaxial crushing test. The lab-checked compressive values were then modified with the code-specified correction factors so $(F_{L/D}, F_{dia}, F_r)$ the results match the equivalent standard cylinder compressive strength. This confirmed strength trend becomes the ultimate axial capacity reference inside the finite element macro-model. In other words, it helps guarantee that the older columns can still safely carry the vertical load vectors applied by the three-story extension.

3.3.3 Chemical Carbonation Depth and Diaphragm Interface Degradation

Because the existing solid slabs are utilized as rigid horizontal diaphragms to harvest and transfer 100% of the lateral inertial forces to the RC exoskeleton, verifying the chemical integrity of the perimeter concrete cover is paramount. Carbonation depth profiling was conducted immediately following core extraction by spraying freshly fractured or drilled concrete surfaces with a 1% phenolphthalein indicator solution dissolved in alcohol.

The chemical response of the concrete substrate was monitored based on colorimetric changes:

- Uncarbonated Concrete Core: Retained a highly alkaline state ($pH > 12.5$), reacting with the indicator to turn into a deep magenta/pink color.
- Carbonated Front: Exhibited a neutralized alkalinity state ($pH < 9$), remaining completely colorless upon contact with the solution.

3.4 Geotechnical Investigation and Soil Profile Characterization

To make sure the safe transfer of the massive gravity and the lateral load vectors collected by the external reinforced concrete RC exoskeleton, a kind of comprehensive geotechnical characterization of the site in Jerusalem was put into the analytical framework. The assessment of subgrade parameters matters a lot, because it helps with designing the independent foundation system for the exoskeleton , and also with setting up the soil surroundings that drives the seismic boundary conditions.

3.4.1 Subsurface Stratigraphy and Site Classification

Based on the site investigation and geological survey reports, the subsurface stratigraphy at the project location belongs to the regional "Bi'na" formation. The general rock mass composition is expected to consist of approximately 75% limestone (medium to hard) and 25% chalk. The unconfined compressive strength of individual limestone core samples ranges between 400 to 800 kg/cm², though the global mass strength is highly jointed and affected by localized karstic features. Additionally, the upper rock layers are covered by a highly expansive clay stratum with a thickness varying from 0.5 to 2.0 meters. This clay pocket layer exhibits a remarkably high swelling potential with a quantified

potential swelling pressure reaching up to 5.0 kg/cm² (500 kPa). In strict accordance with regional seismic criteria and soil classification parameters, the site substrate is designated as Site Class B (Competent Rock). This classification serves as the geological baseline that governs the short-period $(F_a)$and long-period (F_v)site amplification coefficients under standard seismic provisions. The explicit numerical values of these site coefficients, along with their direct impact on the design response spectrum, are comprehensively detailed and applied within the structural loading combinations in Section 3.5.

3.4.2 Foundation Design Parameters and Continuous Strip Footing Mechanics

The existing four story building is essentially supported by shallow foundations. In some places certain peripheral walls sit straight on the rock substrate, while others are held up by reinforced concrete continuous footings or something similar.

To fully isolate the legacy core from the newly introduced structural loads, the RC exoskeleton is set up with its own independent continuous perimeter strip footing, and it basically follows the same structural perimeter line as the new shear walls,not a whole different layout.

As per the governing geotechnical criteria, the allowable net bearing capacity, noted as q_{all},of the competent limestone bedrock is set at 4.0 kg/cm² (400 kPa) for normal static gravity combinations. For transitory extreme events, meaning wind actions and seismic design combinations, the code allows a stress amplification at the contact surface. This gives an increased allowable dynamic bearing capacity of 6.0 kg/cm² (600 kPa).

To avoid any chance of differential settlement and also to fight against the rather high swelling pressure coming from the clay pockets, the design protocol requires the continuous strip footing to have a minimum width of 60 cm. The construction execution framework then requires the complete removal of the upper expansive soil layer, so the footings must go at least 50 cm into the un-weathered hard rock mass. The final foundation depth should end up somewhere between 1.0 and 3.0 meters. The main geotechnical parameters taken from the lab report are summarized in Table 3.2.

Geotechnical Parameter	Value / Metric	Reference Code / Standard
Soil Profile / Site Class	Site Class B (Competent Rock / Bi'na Formation)	ASCE 7-16 Chapter 20 / IS 413
Site Amplification Coefficients (F_a, F_v)	Evaluated in Section 3.5	ASCE 7-16 Chapter 11
Allowable Bearing Capacity ($q_{all} - Static$)	400 kPa (4.0 kg/cm²)	ASTM D1194 / Geotechnical Report
Allowable Bearing Capacity (q_{all}-Seismic)	600 kPa (6.0 kg/cm²) (50% Amplification Allowed)	ASCE 7-16 Combinations / Report
Limestone Unconfined Strength	400 – 800 kg/cm²	Laboratory Uniaxial Testing
Potential Clay Swelling Pressure	500 kPa (5.0 kg/cm²)	ASTM D4546 / High Swelling Profile
Minimum Strip Footing Width	60 cm	Code Design Standard Criteria
Minimum Embedment in Hard Rock	50 cm	Strata Penetration Requirement
Final Foundation Depth Range	1.0 – 3.0 meters	Geotechnical Recommendation
Groundwater Table Depth	Not Encountered (Deep stable bedrock)	Field Borehole Log Documentation

Table 3.2: Governing geotechnical design parameters and soil profile specifications for the Jerusalem project site.

3.4.3 Seismic Liquefaction and Geotechnical Construction Safety

Because of how the project is placed, and within a validated seismic hazard region, there has to be a screening step for possible soil liquefaction. From what is written in the subsurface record, the profile shows competent, very high-strength limestone bedrock tied to the “Bi'na” formation, so the liquefaction triggering likelihood at the location is judged as non existent. In other words, the hard-rock structural backbone effectively blocks any pore-water pressure increase, or a shear-strength drop, during design-basis shaking events.

Also, since the new independent, continuous strip footings are going to be excavated right near the 60 year old shallow foundations , and maybe even down to a lower bearing elevation, we apply strict structural mitigation rules. The digging for the perimeter RC exoskeleton casing will be carried out in a tightly controlled sequential fashion, using staged excavation blocks, so we don’t cause undermining or stress-relief type failures under the old gravity frames. As required by the geotechnical guidance, the excavation for the main corner footings and the first casting phases of the foundation will be monitored by direct on-site geotechnical engineering oversight, just to confirm structural safety and also ongoing stability of the rock mass.

3.5 Load Calculation

Structural loads computations are among the preliminary and most important activities in structural design. Hence, the loads will be identified for conducting a good analysis and checking the safety and stability of the structure. Given that the building is already there, the load calculations will reflect the real situation of the structure as it is today.

3.5.1 Dead Loads (DL):

The dead load components account for the permanent gravitational weights of all structural and non-structural elements. In accordance with the Jordanian National Code for Loads and Forces, the dead loading framework is split into two primary segments:

Self-Weight of Structural Members: The dead weight of the reinforced concrete core and the perimeter exoskeleton casing (beams, columns, and solid slabs) is automatically computed by the finite element analysis software (ETABS) based on a designated material unit weight of 25 kN/m^3 for reinforced concrete.Superimposed Dead Loads

(SDL): These permanent actions incorporate architectural finishes, floor tiling, screed, internal plastering, and mechanical/electrical installations.The uniform Superimposed Dead Load SDL applied across the typical residential floor diaphragms is analytically derived based on the specific material layers and nominal densities prescribed by the Jordanian Building Code, as detailed in Table 3.3.

Table 3.3: Material specification, thickness breakdown, and structural load calibration for typical floor Superimposed Dead Loads (SDL) and internal partitions.

No.	Material	Calculation
1	Tile	0.03*23*1= 0.69 kN/m^2
2	mortar	0.03*22*1= 0.66 kN/m^2
3	Coarse sand	0.07*17*1= 1.19 kN/m^2
4	Ceiling Plaster	0.02*22*1= 0.44 kN/m^2
5	Mechanical&Electrical Services (MEP)	0.2 kN/m^2
6	Interior partitions	$(\gamma_{\text{block}} \cdot t_{\text{wall}} + 2 \cdot \gamma_{\text{plaster}} \cdot t_{\text{plaster}}) \cdot h_{\text{clear}}$
Sum		3.18 kN/m^2

3.5.2 Live Loads (LL) — Jordanian Code Framework

Occupational live loads are basically transient forces that show up because people occupy a space, place furniture around, move domestic equipment a bit here and there, and also because of temporary actions during routine maintenance. Different from permanent dead loads, these live loads tend to shift in both magnitude and location over time. So, in strict compliance with the structural occupancy provisions in the Jordanian National Building Code (Loads and Forces Code), the nominal unreduced design live loads, noted as (L_o),that are assigned over the various functional zones of that seven-story residential building are standardized in Table 3.4.

Table 3.4: Nominal unreduced live load design values specified by the Jordanian National Building Code for residential structural assets.

Item No.	Occupancy / Functional Zone Classification	Nominal Live Load (Lo) (kN/m2)
1	Residential Apartments & Private Rooms	2.00
2	Corridors and Public Lobbies	3.00
3	Staircases and Primary Escape Routes	3.00
4	Balconies and Protruding Terraces	3.00
5	Non-Accessible Roofs (Maintenance Actions Only)	1.00

For the horizontal floor slabs, and the localized beam elements, the full nominal intensity (L_o) is explicitly assigned inside the finite element macro-model so it can help structural serviceability, and verify local punching as well as the flexural limit states . In a way, it's meant to be sure the whole thing behaves properly, not just roughly, but at the specific zones where those beam elements sit.

3.5.3 Rain and Drainage Pooling Loads

Rain loading represents transient hydrostatic forces generated by catastrophic water accumulation (ponding) on roof boundaries, typically triggered by primary drainage clogging. For the retrofitted seven-story structure, direct numerical rain profiling is safely neglected within the structural combination framework.

This justification is strictly governed by the architectural layout of the terminal roof, which enforces an active gravity drainage run-off slope integrated with secondary safety overflow scuppers (openings in the parapet wall). In strict compliance with ASCE 7-16 (Chapter 8), because ponding accumulation is geometrically prevented via verified overflow paths, the structural risk of hydraulic consolidation is eliminated, making the implementation of separate rain load combinations redundant.

3.5.4 Snow Loads (ASCE 7-16 Framework)

Unlike rain actions, snow accumulation constitutes a primary environmental risk that cannot be neglected within the high-altitude topography of the Jerusalem mountains. In accordance with regional hazard data compiled for an absolute site elevation of 750 m above sea level, the basic ground snow load (p_g) is established as $0.875\ kN/m$

Following the analytical protocol of ASCE 7-16 (Chapter 7), the design flat roof snow load (p_s) applied uniformly over the horizontal shell elements of the newly added seventh floor is formulated as:

$$\boldsymbol{p_s = 0.7 \cdot C_e \cdot C_t \cdot I_s \cdot p_g}$$

Evaluating the baseline indices for a standard residential exposure (Exposure Category B, $C_e = 1.0, C_t = 1.0, I_s = 1.0$) yields a mandatory uniform design action of 0.613 kN/m^2. This environmental force is explicitly structuralized within the ETABS global matrix to verify the ultimate limit state of the upper slab and its direct transfer paths to the perimeter exoskeleton shear walls.

3.5.5.2 Basic Design Wind Speed (V)

According to ASCE 7-16 (Chapter 26), the basic design wind speed (V) must be specified as a 3-second gust velocity measured at 10 m above ground level within an un-obstructed Exposure Category C environment. Because the project site in the Jerusalem highlands lies outside the direct geographic mapping grid of the standard ASCE 7-16 charts, local meteorological historical data and regional engineering standards are cross-referenced to establish compliance.

For a standard residential asset classified under Risk Category II, the kodiak recurrence interval for wind hazard mapping corresponds to a 700-year mean return period. Based on localized microclimate data and regional technical consensus, the basic design wind speed adopted for the safety evaluation of the Main Wind-Force Resisting System (MWFRS) is designated as:

$$V = 44 \text{ m/s} \quad (\approx 100 \text{ mph})$$

This baseline wind velocity represents the ultimate un-factored environmental action driving the dynamic pressure equations, ensuring that the lateral windward and leeward impact envelopes on the RC exoskeleton casing are conservatively captured.

3.5.6 SEISMIC ANALYSIS

Prepared for 7-Story Reinforced Concrete Building with Exoskeleton Casing – Height = 20.18 m

This chapter explains the seismic design parameters and structural analysis methodologies utilized to evaluate the proposed retrofitted multi-story reinforced concrete structure. The seismic design is conducted in strict compliance with the stipulations of ASCE 7-16, employing the Equivalent Lateral Force (ELF) Procedure for the structural assessment as per the code provisions.

The total seismic height of the seven-story reinforced concrete building is: h_n= 20.18 m
The structure is taken to be **Risk Category II** while the Importance Factor is:
Ie=1.0

Table 1.5-2 Importance Factors by Risk Category of Buildings and Other Structures for Snow, Ice, and Earthquake Loads

Risk Category from Table 1.5-1	Snow Importance Factor, I_s	Ice Importance Factor—Thickness, I_i	Ice Importance Factor—Wind, I_w	Seismic Importance Factor, I_e
I	0.80	0.80	1.00	1.00
II	1.00	1.00	1.00	1.00
III	1.10	1.15	1.00	1.25
IV	1.20	1.25	1.00	1.50

Note: The component importance factor, I_p, applicable to earthquake loads, is not included in this table because it depends on the importance of the individual component rather than that of the building as a whole, or its occupancy. Refer to Section 13.1.3.

Table 3.4 ASCE 7 tabulated importance factors based on the building's risk category.

3.5.6.2 Seismic Hazard Parameters

The site has the following mapped spectral accelerations:

$$\boldsymbol{S_s = 0.56g}$$

$$\boldsymbol{S_1 = 0.28g}$$

And it is placed in **Site Class B** according to the geotechnical conditions that are supposed.

3.5.6.3. Site Coefficients and Design Spectrum

According to ASCE 7-16 Tables 11.4-1 and 11.4-2:

$$F_a = 0.9$$

$$F_v = 0.8$$

Table 11.4-1 Short-Period Site Coefficient, F_a

Site Class	Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at Short Period					
	$S_S \le 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S = 1.25$	$S_S \ge 1.5$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.9	0.9	0.9	0.9	0.9	0.9
C	1.3	1.3	1.2	1.2	1.2	1.2
D	1.6	1.4	1.2	1.1	1.0	1.0
E	2.4	1.7	1.3	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8
F	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8

Note: Use straight-line interpolation for intermediate values of S_s.

Table 11.4-2 Long-Period Site Coefficient, F_v

Site Class	Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at 1-s Period					
	$S_1 \le 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 = 0.5$	$S_1 \ge 0.6$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.8	0.8	0.8	0.8	0.8	0.8
C	1.5	1.5	1.5	1.5	1.5	1.4
D	2.4	2.2[a]	2.0[a]	1.9[a]	1.8[a]	1.7[a]
E	4.2	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8
F	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8

Note: Use straight-line interpolation for intermediate values of S_1.
[a]Also, see requirements for site-specific ground motions in Section 11.4.8.

Table 3.5 Determination of the short-period (Fa) and long-period (Fv) seismic site coefficients based on ASCE 7 provisions, highlighting the values corresponding to Site Class B.

MCER Spectral Response Accelerations

$$S_{MS} = F_a S_s = 0.9 \times 0.56 = 0.504$$

$$S_{M1} = F_v S_1 = 0.8 \times 0.28 = 0.224$$

Design Spectral Response Accelerations

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 0.504 = 0.336$$

$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 0.224 = 0.149$$

Seismic Design Category

TABLE 11.6-1 Seismic Design Category Based on Short-Period Response Acceleration Parameter

Value of S_{DS}	Risk Category I or II or III	Risk Category IV
$S_{DS} < 0.167$	A	A
$0.167 \le S_{DS} < 0.33$	B	C
$0.33 \le S_{DS} < 0.50$	C	D
$0.50 \le S_{DS}$	D	D

TABLE 11.6-2 Seismic Design Category Based on 1-s Period Response Acceleration Parameter

Value of S_{D1}	Risk Category I or II or III	Risk Category IV
$S_{D1} < 0.067$	A	A
$0.067 < S_{D1} < 0.133$	B	C
$0.133 \le S_{D1} < 0.20$	C	D
$0.20 \le S_{D1}$	D	D

Table 3.6 Seismic Design Category (SDC) determination tables

Seismic Design Category $= \boldsymbol{C}$

3.5.6.4. Seismic Force-Resisting System (SFRS)

The lateral seismic force-resisting system (SFRS) of the retrofitted structure is classified as an ASCE-compliant **Bearing Wall System featuring Special Reinforced Concrete Shear Walls**. Geometrically, this system is engineered as a continuous, three-dimensional **Monolithic Rigid Box Exoskeleton Casing** wrapping the peripheral boundary of the legacy core. By interlocking the transverse and longitudinal shear walls at the exterior corners, the structure acts as a rigid hollow box, maximizing torsional resistance and eliminating severe stress concentrations.

According to the design specifications of **ASCE 7-16 (Table 12.2-1)**, the structural coefficients assigned to govern the numerical modeling and cross-sectional design of this box-type configuration are:

- **Response Modification Coefficient (R):** 5.0
- **System Overstrength Factor** (Ω_0):2.5
- **Deflection Amplification Factor (C_d):** 5.0

Justification of the Rigid Box Mechanism: Designing the exoskeleton under a Bearing Wall System criteria with a continuous box layout ($R = 5.0$) ensures that the peripheral shell resists 100% of the horizontal seismic shear while safely sustaining the gravity load demands of the newly added upper floors.

The closed-loop box configuration provides exceptional structural redundancy; when lateral loads strike any principal direction, the parallel walls act as primary web shear frames, while the orthogonal walls act as rigid flanges. This cellular box-type behavior prevents independent multi-story drift, safely shielding the non-ductile old core within an un-yielding structural envelope.

3.5.6.5. Fundamental Period and Dynamic Modal Analysis

Rather than relying on empirical, height-dependent approximations, the fundamental natural period $(T)$of the retrofitted multi-story asset is rigorously evaluated using Dynamic Modal Analysis (Eigenvalue Procedure) inside ETABS. This numerical approach solves the characteristic undamped free-vibration eigenvalue problem based on the global stiffness matrix $([K])$and the compiled mass source matrix $([M])$of the monolithic rigid box exoskeleton, as formulated in Equation 3.8:

$$\det([K] - \omega^2[M]) = 0$$

Where $\boldsymbol{\omega}$ represents the natural circular frequencies of the structure (rad/s).This dynamic procedure strictly complies with the Modal Response Spectrum Analysis provisions of ASCE 7-16 (Chapter 12.9), ensuring that the mass-stiffness distribution of the continuous reinforced concrete shear wall shell is precisely simulated.

3.5.6.5.1 Modal Mass Participation and Target Design Spectrum

The dynamic response of the continuous exoskeleton box structure is evaluated by extracting the natural periods from the dominant modes of vibration. The numerical model is calibrated to satisfy the code requirement where the cumulative modal mass participation exceeds 90% of the actual structural mass in both principal orthogonal directions ($X$and Y).

The fundamental elastic period (T) governing the dynamic lateral demand is determined directly from the first translational mode shapes:

- Mode 1 (Primary Translation): Establishes the governing fundamental period (T) along the critical structural axis.

- Response Spectrum Scaling: The extracted dynamic periods are mapped directly against the calibrated ASCE 7-16 elastic design response spectrum (defined by $S_{DS} = 0.336g$ and S_{D1}= $0.1493g$) to extract the exact dynamic spectral accelerations and distribute the inertial seismic forces across the vertical wall piers.

3.5.6.6 Equivalent Lateral Force (ELF) Baseline for Dynamic Scaling

In accordance with **ASCE 7-16 (Section 12.9.4.1)**, the lateral base shear forces obtained via the Dynamic Response Spectrum Analysis (V_{dynamic}) must be cross-checked against the baseline Equivalent Lateral Force (ELF) procedure. If the dynamic base shear is less than 100% of the calculated ELF base shear (V_{ELF}), a scaling factor must be applied to the dynamic results inside ETABS.

For the **Special Reinforced Concrete Shear Wall Box System** (R = 5.0, I_e= 1.0), the benchmark seismic response coefficient (C_s) is formulated using ASCE 7-16 Equation 12.8-2:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \frac{0.373}{\left(\frac{5.0}{1.0}\right)} = 0.0746$$

This coefficient satisfies the upper bound ($C_{s,\max} = 0.0804$) calculated using the empirical period ($T_a = 0.465$ s), and the absolute minimum lower bound ($C_{s,\min} = 0.0164$). Since the acceleration factor complies with the code boundaries:

$$C_{s,\min}\,(0.0164) \leq C_s\,(0.0746) \leq C_{s,\max}\,(0.0804)$$

The governing baseline seismic response coefficient settles precisely at **0.0746**.

3.5.6.7 Minimum Target Base Shear for Scaling

The baseline nominal static base shear (V_{ELF}) serving as the absolute lower-bound threshold for the dynamic design is expressed as:

$$V_{\text{ELF}} = C_s \cdot W$$

$$V_{\text{ELF}} = 0.0746 \cdot W$$

Where W is the total effective seismic weight of the seven-story structure. If the dynamic modal analysis outputs a base shear lower than this value ($V_{\text{dynamic}} < V_{\text{ELF}}$), the ETABS response spectrum scale factor is linearly adjusted by a ratio of $\left(\frac{V_{\text{ELF}}}{V_{\text{dynamic}}}\right)$ to ensure strict code compliance.

CHAPTER4: Structural Assessment and Design Verification.

4.1 Introduction

This chapter presents the structural assessment and design verification of the proposed seven-story reinforced concrete building strengthened by an external reinforced concrete exoskeleton casing. The primary objective is to evaluate the adequacy of the proposed retrofitting system under gravity and seismic loading conditions, and to verify that the strengthened configuration satisfies the strength, stiffness, stability, and serviceability requirements specified in the adopted design codes.

The analytical investigation is based on the three-dimensional numerical model developed in ETABS (Computers and Structures, Inc., 2023) in the previous chapter. The model incorporates the existing four-story reinforced concrete structure, the proposed three-story vertical extension, and the external RC exoskeleton casing, which is intended to support the additional gravity loads and resist the induced lateral seismic actions. The verification process focuses on the overall seismic response of the structure, including modal behavior, base shear scaling, story drift control, lateral displacement limits, force transfer mechanisms, and the adequacy of the main structural components.

The assessment procedure follows a sequential verification approach. First, the dynamic modal behavior of the structure is evaluated to ensure that the selected number of vibration modes captures the required percentage of participating mass in both principal horizontal directions. Second, the response spectrum base shear is compared with the equivalent lateral force base shear in order to determine the required scaling factors. Third, the lateral displacement and inter-story drift responses are examined to confirm that the exoskeleton casing provides sufficient lateral stiffness and limits excessive deformation. Finally, the primary force-resisting components, including the external shear walls, diaphragm transfer mechanism, and supporting foundation system, are checked to ensure safe, continuous, and reliable load transfer throughout the retrofitted structural system.

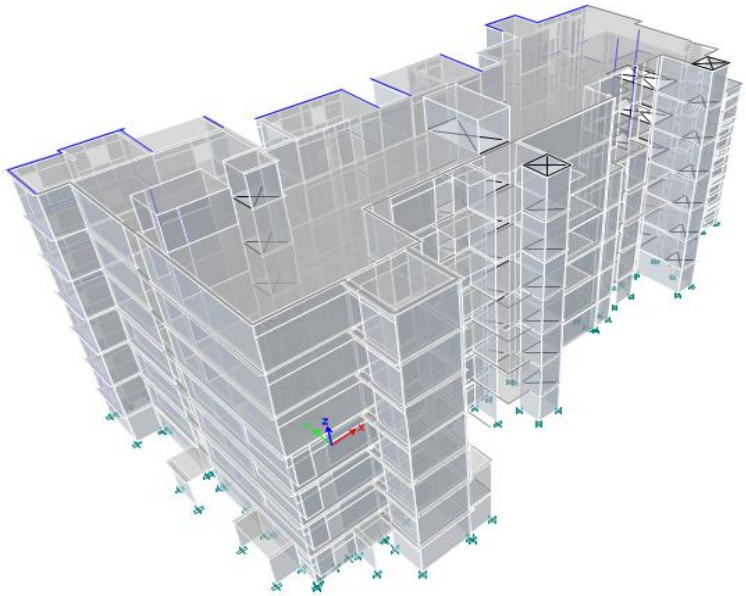


Figure 4.1: Three-dimensional analytical ETABS model of the seven-story retrofitted building.

4.2 Existing Building Condition Assessment

Prior to the development of the analytical model and the implementation of the proposed rehabilitation strategy, a comprehensive field assessment was conducted to evaluate the current structural condition of the existing building. The investigated structure is a residential reinforced concrete building approximately sixty years old, located in Jerusalem, Palestine. The building consists of four stories constructed using reinforced concrete structural walls and solid slabs, representing a construction period that predates the adoption of modern seismic design provisions and ductile detailing requirements.

During the field investigation, we did a kind of visual look over the accessible structural elements , also we noted the deterioration patterns, we identified structural deficiencies, and we assessed the overall state of the load-bearing system. We paid close attention to

the external walls , the slab edges, the wall-to-slab join areas, and the zones that seem to get hit the most by long-term environmental impacts.

From the inspection it was clear that, even with its age, the building still appears to be in generally good structural condition. We did not notice any major structural instability, no obvious excessive deformation , and no critical distress mechanisms during the on-site survey. Still, we did find a few localized signs of deterioration that match what you'd expect after long-term weathering. For example, there was limited cracking in the concrete , carbonation on exposed concrete surfaces, localized moisture-related effects, minor loss in concrete cover, and a few isolated indications pointing toward reinforcement corrosion.

Even though these observations are pretty typical for reinforced concrete structures from that kind of age, they do not seem to weaken the overall structural integrity of the main load bearing system in a meaningful way. Still, the deterioration mechanisms that were identified were written down , and they were taken into account during the structural assessment process. this was done so that the proposed rehabilitation strategy properly tackles both current, as well as future performance needs.

The findings obtained from the field assessment established the basis for the subsequent material testing program and numerical modeling procedures. The collected information was incorporated into the analytical evaluation to ensure that the numerical model accurately represents the actual condition of the existing structure.

Figure 4.2 presents the general condition of the existing building prior to rehabilitation.

Figure 4.3: Localized cracking, moisture-related deterioration, and minor reinforcement corrosion observed in a reinforced concrete connection region during the field assessment.

Figure 4.4: Deterioration of reinforced concrete characterized by reinforcement corrosion, cracking, and loss of concrete cover in a weather-exposed structural element.

Figure 4.5: Localized reinforcement corrosion and concrete cover deterioration observed in an existing reinforced concrete slab during the field inspection.

Figure 4.6: Representative condition of the existing structure showing adequate structural integrity despite localized age-related deterioration.

4.2.1 Identified Structural Deficiencies

The visual inspection identified several deterioration mechanisms commonly encountered in aging reinforced concrete buildings. Although the severity of these defects was generally limited, their potential influence on long-term durability and structural performance was evaluated as part of the assessment process.

Table 4.1 summarizes the primary deficiencies identified during the field investigation together with their potential structural implications.

Table 4.1: Summary of observed structural deficiencies and their engineering implications.

Structural Deficiency	Observed Condition	Potential Structural Implication
Localized Concrete Cracking	Limited isolated cracking	Local reduction in stiffness and durability
Carbonation of Concrete Cover	Observed in exposed locations	Increased risk of reinforcement corrosion
Reinforcement Corrosion	Minor localized corrosion signs	Potential reduction in reinforcement cross-section over time
Moisture Penetration	Local dampness and weather exposure	Accelerated deterioration processes
Surface Concrete Deterioration	Minor cover damage in isolated areas	Reduced protection of embedded reinforcement

Even with all those deterioration mechanisms around, the building overall condition was judged, as kind of satisfactory, despite everything. The defects that were seen in practice were mainly pretty localized and they did not really suggest any big decline in the global stability of the structure. Still, once you take into account the proposed three-story vertical expansion, and the needs coming from current seismic design codes, strengthening and rehab measures were considered necessary, to make sure the long term structural safety, and the serviceability, stay in good shape.

4.2.2 Summary of Material Condition Assessment

To verify the actual mechanical properties of the existing structure, a material testing program was carried out. The investigation included non-destructive testing, field observations, and laboratory evaluation of concrete samples extracted from representative structural elements.

The laboratory results indicated that the reinforced concrete structural walls achieved an average compressive strength of approximately 22 MPa, while the reinforced concrete bond beams and ring beams achieved an average compressive strength of approximately 18 MPa. These values demonstrate that the primary structural elements retain adequate residual strength despite prolonged environmental exposure and structural aging.

The obtained results confirm that the existing structure remains suitable for rehabilitation and strengthening. Although localized deterioration mechanisms were identified during the field investigation, the measured material properties indicate that the building maintains sufficient structural capacity to support the proposed rehabilitation strategy when combined with the external reinforced concrete exoskeleton casing.

Accordingly, the measured material properties were adopted during the calibration of the ETABS analytical model presented in the following sections, ensuring that the numerical simulations accurately represent the actual condition of the existing structure.

4.3 Description of the Analytical Verification Model

The structural verification was done with a three- dimensional finite element model built in ETABS, or, well, basically that's where everything started. In the model you can find the existing reinforced concrete core along with the proposed vertical extension, plus the external RC exoskeleton casing. The current building, as it stands before any change, has four stories. After the retrofitting, the final layout is seven stories, since three extra upper floors were added on top of the original structure.

For the external RC exoskeleton casing, it was modeled as a sort of continuous perimeter wall system that wraps around the existing building. In this setup, the casing is supposed to behave like the main lateral force resisting system, but also like a way to transfer gravity loads for the newly added floors. The idea behind the modeling strategy is that the new gravity and seismic actions are mostly intercepted by the outer casing, rather than being forced straight into the deteriorated internal legacy frame.

As for the floor slabs, they were represented as diaphragm elements, meaning they can pass the inertial forces caused by earthquakes from the floor mass toward the external wall system. The mass used for the dynamic analysis came from several components, such as structural self weight, superimposed dead loads, and also the suitable share of live loads, all taken according to the seismic mass definition that was used in the analysis model

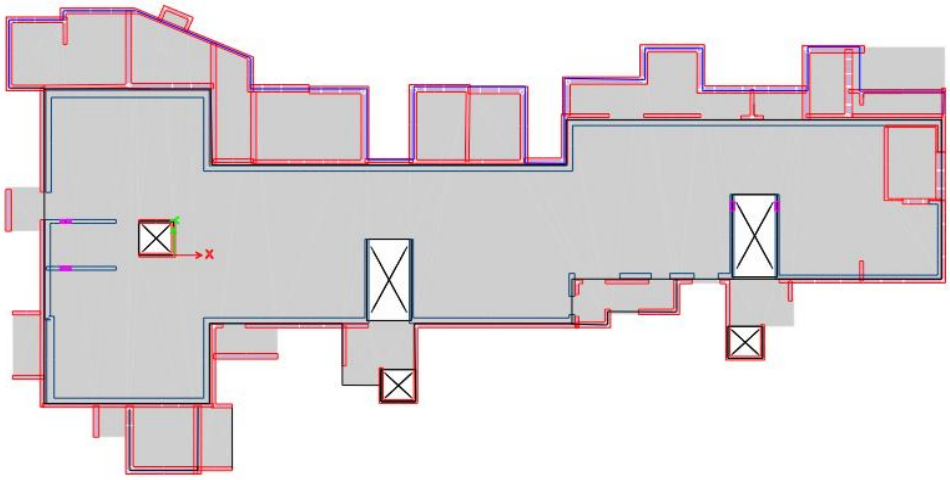


Figure 4.7: Plan view of the ETABS analytical model showing the existing structural walls in blue and the newly added structural walls in red.

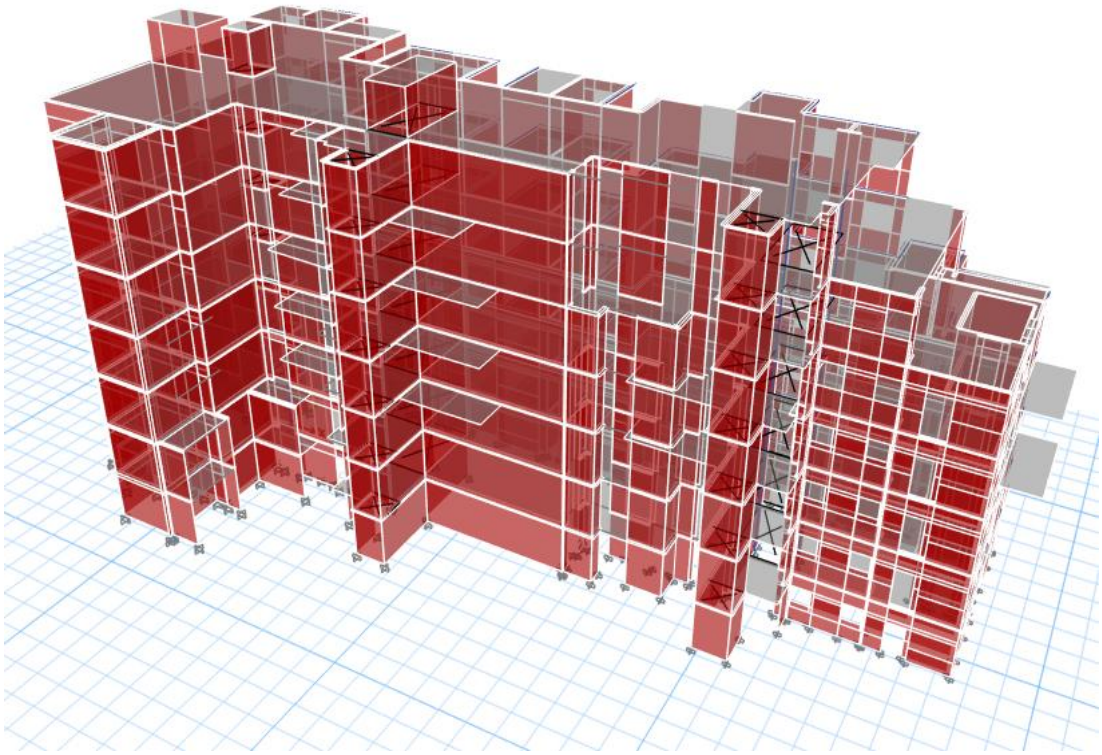


Figure 4.8: Three-dimensional view of the ETABS analytical model showing the existing structure, the proposed vertical extension, and the external reinforced concrete casing.

4.3.1 General ETABS Model Basis

The analytical model was worked out by using the ETABS definitions of story levels, material properties, shell section properties, stiffness modifiers, load patterns , mass source, seismic parameters, and also response spectrum load cases. These items were then checked, to make sure the model is representing what is actually going on, meaning the difference between the old aged reinforced concrete structure and the newer attached external RC exoskeleton casing.

4.3.2 Story Definitions and Vertical Geometry

The vertical geometry of the model was defined according to the ETABS story data, as summarized in Table 4.2.

Table 4.2: Summary of story height definitions adopted in the ETABS model.

ETABS story level	Story height, m
Story8	2.44
Story7	3.33
Story6	3.22
Story5	3.26
Story4	3.16
Story3	3.12
Story2	3.17
Story1	2.87

4.3.3 Material Definitions

Separate material definitions were used to distinguish between the existing and newly added reinforced concrete components. Concrete materials were defined as isotropic materials, while reinforcing steel was defined as a uniaxial rebar material.

Table 4.3: Summary of material definitions adopted in the ETABS analytical model.

Material name	Material type	Unit weight, kN/m³	f'c, MPa	fy, MPa	Ec, MPa	Adopted use in the model
B 180	Concrete	24.0	14.4	—	18025	Existing or lower-strength RC elements, as assigned in the model
B 220	Concrete	24.0	17.6	—	19927.4	Existing RC walls or structural elements, as assigned in the model
B 300	Concrete	24.0	24	—	23270.15	New RC elements or strengthening components, as assigned in the model
B 400	Concrete	24.0	32	—	26870.06	New RC joint regions and local nodal strengthening zones
FY-420	Rebar	78	—	420	200000	New reinforcing steel used for the RC exoskeleton and strengthening system
FY-230	Rebar	78	—	230	200000	Existing reinforcement steel in legacy RC elements

The concrete material definitions were assigned according to the structural role and condition of each element group. Lower-strength concrete materials were used for the existing legacy reinforced concrete elements, while B 300 was assigned to the new reinforced concrete strengthening components. The B 400 concrete material was used only for the new RC joint regions and local nodal strengthening zones where higher local strength was required. The reinforcement materials were also defined separately, with FY-230 representing the existing reinforcement steel and FY-420 representing the new reinforcing steel used in the external RC exoskeleton and strengthening system.

4.3.4 Shell Section Definitions

The slab and wall elements were modeled using shell-thin area elements. Different shell thicknesses were assigned to represent the existing slabs, new slab regions, existing walls, and the newly added external RC exoskeleton walls.

Table 4.4: Summary of shell section definitions adopted in the ETABS model.

Section category	Section name in ETABS	Material	Thickness, mm	Adopted use in the model
Slab	Solid Slab 200 B180	B180	200	Existing slab elements
Slab	Slab 200 B300	B400	200	New or strengthened slab elements
Slab	Solid Slab 300 B400	B400	300	New thickened slab regions
Slab	Solid Slab 320 B400	B400	320	New thickened slab or stiffened regions
Slab	Stiff1	B400	320	Local stiffened slab region
Wall	Wall 100 B300	B300	100	Thin RC wall elements
Wall	Wall 200 B220	B220	200	Existing RC wall elements
Wall	Wall 200 B300	B300	200	New RC wall elements
Wall	Wall 300 B220	B220	300	Existing thicker RC wall elements
Wall	Wall 300 B300	B300	300	New RC wall elements
Wall	Wall 400 B220	B220	400	Existing thick RC wall elements
Wall	Wall 400 B300	B300	400	New RC exoskeleton wall elements
Wall	Wall 500 B300	B300	500	Main external RC exoskeleton wall elements

4.3.5 Stiffness Modifiers and Mass Assumptions

To account for the different stiffness contributions of the existing and newly added reinforced concrete elements, separate stiffness modification factors were assigned in the ETABS model. For the existing legacy RC walls, membrane and bending stiffness modifiers of 0.35 were applied to represent the cracked and aged condition of the old wall system. This conservative assumption was adopted to avoid overestimating the lateral stiffness contribution of the existing structure.

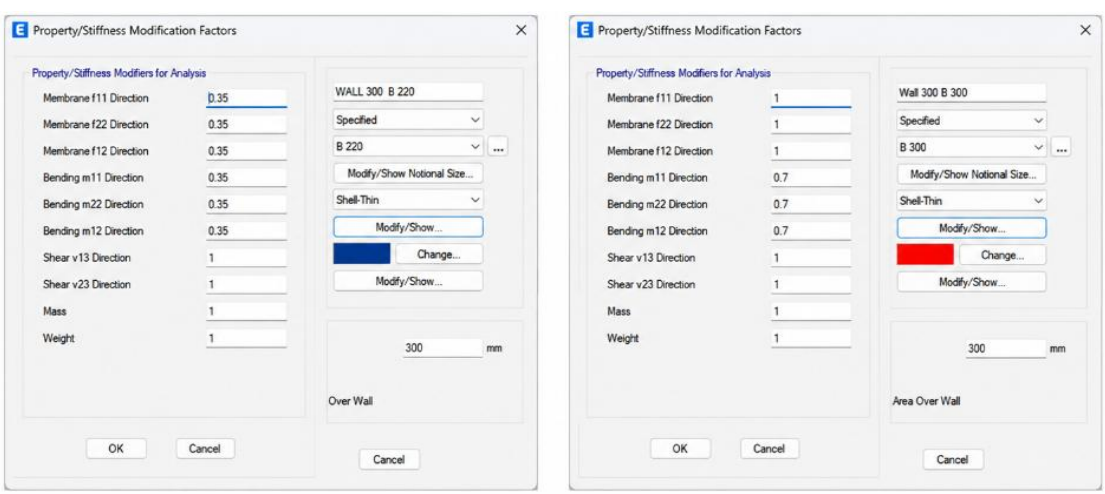
For the newly added external RC exoskeleton walls, the membrane stiffness modifiers were kept equal to 1.0, while the bending stiffness modifiers were reduced to 0.70. This assumption reflects the higher reliability and stiffness of the new RC wall system while accounting for effective cracked bending behavior. For the slab elements, the membrane stiffness modifiers were kept equal to 1.0 to maintain diaphragm action, while the bending stiffness modifiers were reduced to 0.25 to represent cracked slab bending behavior. The shear, mass, and weight modifiers were kept equal to 1.0 to preserve the actual gravity load and seismic mass of the building.

Table 4.5: Summary of stiffness and mass modification assumptions.

Element group	Membrane stiffness modifier	Bending stiffness modifier	Shear modifier	Mass modifier	Weight modifier	Purpose
Existing RC walls	0.35	0.35	1.0	1.0	1.0	Represents cracked and aged legacy walls
New external RC exoskeleton walls	1.00	0.70	1.0	1.0	1.0	Represents new RC walls with reduced effective bending stiffness
Slabs elements	1.0	0.25	1.0	1.0	1.0	Represents cracked slab bending stiffness while maintaining diaphragm action and actual seismic mass

The ETABS model included the gravity and seismic load patterns required for the structural assessment of the retrofitted building. The Dead load pattern was assigned a self-weight multiplier of 1.0 so that the structural self-weight was automatically included in the analysis. The SD load pattern was used to represent superimposed dead loads, including finishes and permanent non-structural loads, while the Live load pattern was used to represent occupancy-related live loads. The equivalent static seismic load patterns, EQX and EQY, were defined according to ASCE 7-16 and were used as reference cases for scaling and comparing the response spectrum analysis results.

The assigned stiffness modifiers were verified through the ETABS property/stiffness modification definitions, as shown in Figure 4.9.



(a) Existing RC wall section (b) New RC wall section

Figure 4.9: ETABS property/stiffness modification factors adopted for wall elements: (a) existing RC walls with stiffness modifiers of 0.35; (b) new external RC exoskeleton walls with stiffness modifiers of 0.70.

4.3.6 Load Patterns and Mass Source Definition

Table 4.6: Summary of load pattern definitions adopted in the ETABS model.

Load pattern	Load type	Self-weight multiplier	Adopted use in the model
Dead	Dead	1.0	Structural self-weight and permanent gravity load
SD	Super Dead	0.0	Superimposed dead loads, including finishes and permanent non-structural loads
Live	Live	0.0	Occupancy live load
EQX	Seismic	0.0	Equivalent static seismic load in the X direction
EQY	Seismic	0.0	Equivalent static seismic load in the Y direction

4.3.7 Seismic Parameters and Response Spectrum Cases

The seismic loading parameters were defined according to ASCE 7-16. The structure was classified as Risk Category II, with a seismic importance factor of Ie = 1.0. The mapped spectral acceleration parameters were taken as Ss = 0.56 and S1 = 0.28, with Site Class B. The corresponding site coefficients were Fa = 0.9 and Fv = 0.8, resulting in design spectral response accelerations of SDS = 0.336 and SD1 = 0.1493. The adopted response modification factor was R = 5.0, the overstrength factor was Ω = 2.5, and the deflection amplification factor was Cd = 5.

Table 4.7: Summary of seismic parameters adopted in the ETABS model.

Parameter	Adopted value
Seismic code basis	ASCE 7-16
Risk category	II
Seismic importance factor, Ie	1.0
Site class	B
Ss	0.56
S1	0.28
Fa	0.9
Fv	0.8
SDS	0.336
SD1	0.1493
Response modification factor, R	5.0
Overstrength factor, Ω	2.5
Deflection amplification factor, Cd	5

The ASCE 7-16 design response spectrum function was defined in ETABS using the adopted seismic site parameters and design spectral accelerations, as shown in Figure 4.10.

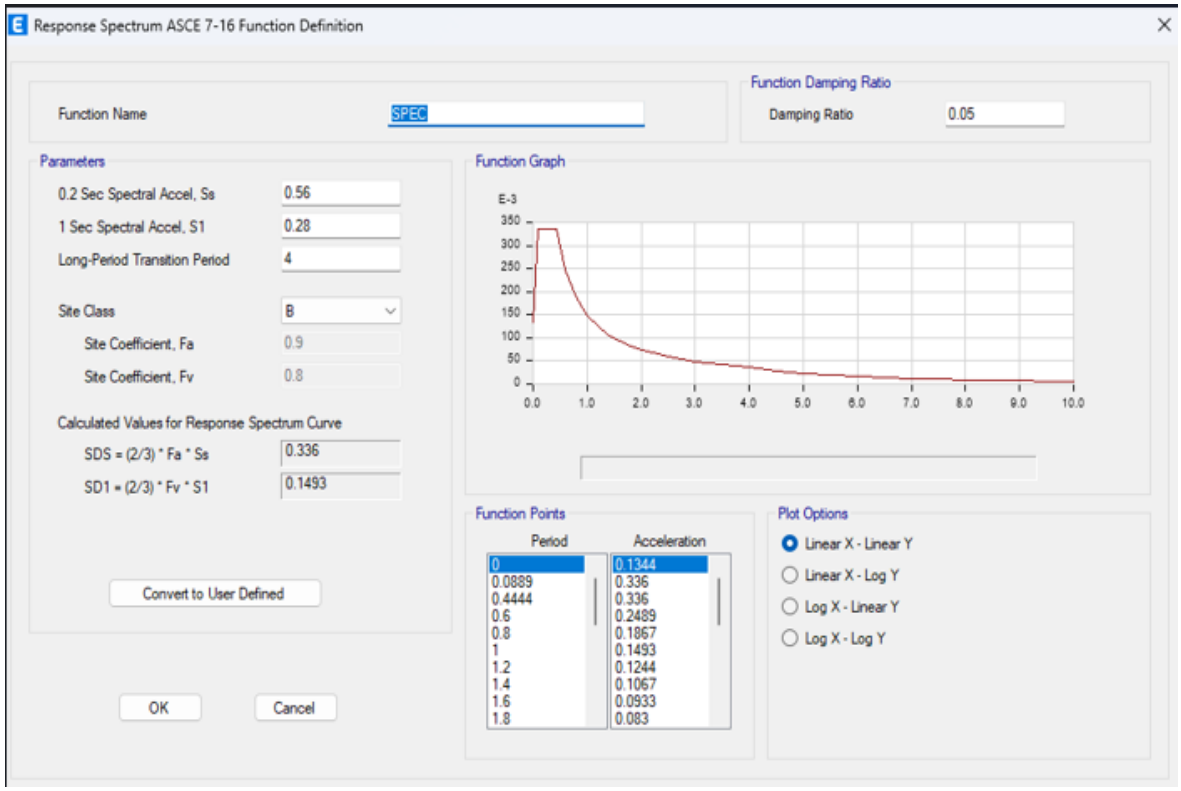


Figure 4.10: ASCE 7-16 response spectrum function definition adopted in ETABS for the dynamic seismic analysis, showing the selected site parameters, damping ratio, and calculated design spectral acceleration values.

The response spectrum analysis was performed in the two principal horizontal directions using the RSAX and RSAY load cases. The RSAX case was applied in the U1 direction, while the RSAY case was applied in the U2 direction. These response spectrum cases were later scaled to satisfy the equivalent static base shear requirement before being used for story shear, story drift, displacement, and design verification checks.

Table 4.8: Summary of response spectrum load cases adopted in the ETABS model.

Load case	Analysis type	Direction	Spectrum function	Combination method	Adopted use
RSAX	Response spectrum	U1 / X direction	ASCE 7-16 spectrum	CQC modal combination	Dynamic seismic analysis in X direction
RSAY	Response spectrum	U2 / Y direction	ASCE 7-16 spectrum	CQC modal combination	Dynamic seismic analysis in Y direction

Accordingly, the adopted ETABS model definitions and modeling assumptions provide a consistent analytical basis for evaluating the seismic behavior and structural performance of the retrofitted building. These assumptions ensure that the contribution of the aged existing structure is not overestimated, while the external RC exoskeleton casing is properly represented as the primary strengthening and lateral force-resisting system.

4.4 Modal Analysis and Mass Participation Verification

A modal eigenvalue analysis was performed to evaluate the dynamic characteristics of the retrofitted seven-story reinforced concrete structure. This analysis represents a fundamental preliminary stage for the response spectrum procedure because it defines the natural periods, mode shapes, and cumulative modal mass participation ratios that govern the seismic response of the building.

In the early test runs, it turned out that using only a small set of modes was not enough to reach the required participating mass in the two main horizontal directions. That shortfall was mostly linked to the stiffness focusing created by the outside reinforced concrete exoskeleton casing, plus the heavier seismic mass introduced by the proposed three-story vertical extension. For that reason, the maximum number of modes was raised up to 150 modes, so as to get a more complete picture of how the structure behaves dynamically, overall.

Modal Participating Mass Ratios

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Units: As Noted Hidden Columns: Yes Sort: None Modal Participating Mass Ratio

Filter: None

	Case	Mode	Period sec	UX	UY	SumUX	SumUY
	Modal	106	0.037	0.0007	0.0008	0.8953	0.8761
	Modal	107	0.037	0.0002	0.0477	0.8955	0.9238
	Modal	108	0.035	8.671E-06	0.0001	0.8955	0.9239
	Modal	109	0.035	0	0	0.8955	0.9239
	Modal	110	0.031	0.0388	0.0018	0.9343	0.9257
	Modal	111	0.029	0.0003	0.0112	0.9346	0.9369
	Modal	112	0.028	3.306E-06	2.666E-05	0.9346	0.9369
	Modal	113	0.028	0	0	0.9346	0.9369
	Modal	114	0.027	0.014	0.0028	0.9486	0.9397
	Modal	115	0.025	0.0004	0.0001	0.9489	0.9399
	Modal	116	0.025	0.0004	0.0103	0.9493	0.9502
	Modal	117	0.023	0.0008	0.0146	0.950099999...	0.9647
	Modal	118	0.021	0.0238	0.0013	0.974	0.966
	Modal	119	0.02	0.0003	3.523E-06	0.9742	0.966
▶	Modal	120	0.02	0.0001	0.0001	0.9743	0.9661
	Modal	121	0.019	0.0008	0.0094	0.9751	0.9755

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Figure 4.11: ETABS modal participating mass ratios showing that the cumulative modal mass participation exceeds 90% in both principal horizontal directions after increasing the number of extracted eigen modes.

Table 4.9: Summary of modal mass participation verification.

Direction / Component	Mode at which 90% was exceeded	Cumulative participation at that mode	Cumulative participation at Mode 120	Verification status
UX	110	0.9343	0.9743	Satisfactory
UY	107	0.9238	0.9661	Satisfactory

The results presented in Table 4.2 indicate that the adopted modal basis is sufficient for the dynamic analysis of the retrofitted structure. The cumulative participating mass in the Y direction exceeded the 90% threshold at Mode 107, reaching 0.9238. The X direction exceeded the same requirement at Mode 110, reaching 0.9343. Moreover, by Mode 120, the cumulative mass participation increased to 0.9743 in the X direction and 0.9661 in the Y direction. These values confirm that the response of the structure is adequately represented by the extracted eigen modes.

The need for a relatively high number of modes is attributed to the structural configuration of the retrofitted building. The external RC exoskeleton casing significantly increases the lateral stiffness of the system, while the three-story vertical extension increases the upper-level seismic mass. This combination causes higher modes to contribute noticeably to the total dynamic response. Therefore, limiting the analysis to a smaller number of modes would have underestimated the effective participating mass and could have led to inaccurate response spectrum results.

Although some of the early extracted modes showed very long periods and small individual participation ratios, their contribution to the effective lateral mass was negligible. The dominant translational mass participation was captured in the later modes, where the cumulative participation ratios increased rapidly and exceeded the required 90% limit in both principal horizontal directions.

Accordingly, the modal analysis was considered satisfactory, and the model was accepted for subsequent response spectrum analysis, base shear scaling, story drift verification, and structural design assessment of the external reinforced concrete exoskeleton casing.

4.5 Equivalent Static and Dynamic Base Shear Comparison

The seismic behavior of the retrofitted building was evaluated using both the Equivalent Lateral Force (ELF) procedure and Response Spectrum Analysis (RSA). The equivalent static seismic cases were used as the reference baseline for verifying the minimum required dynamic base shear in accordance with the adopted seismic design framework.
The ETABS story force output indicates that the story shear increases gradually from the upper levels toward the base level, which is the expected trend for a multi-story lateral force-resisting system.

The ETABS base reaction outputs showed that the unscaled response spectrum base shear was lower than the equivalent static base shear in both principal horizontal directions. Therefore, response spectrum scaling was required to ensure that the dynamic seismic actions used for

design are not less than the equivalent static baseline.

The comparison between the equivalent static base shear and the unscaled response spectrum base shear is summarized in Table 4.3.

Table 4.10: Comparison between equivalent static and response spectrum base shear.

Direction	Equivalent Static Case	Static Base Shear, kN	Response Spectrum Case	Dynamic Base Shear, kN	Required Scale Ratio
X-direction	EQX	6667.86	RSAX	4333.898	1.5385
Y-direction	EQY	6667.86	RSAY	4427.667	1.5059

Base Reactions

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Units: As Noted Hidden Columns: Yes Sort: None Base Reactions

Filter: ([Output Case] = 'EQX1' OR [Output Case] = 'EQY' OR [Output Case] = 'RSAX' OR [Output Case] = 'RSAY')

Output Case	FX kN	FY kN	FZ kN	MX kN-m	MY kN-m	MZ kN-m
EQX1	-6667.8645	-2.713E-05	1.092E-05	0.0007	-98502.5978	17290.5102
EQY	0.0001	-6667.8646	-8.077E-06	98502.604	-0.0001	-84015.9405
RSAX	6663.2415	651.2277	4.017E-05	9598.1924	98291.047	66544.156
RSAY	637.7169	6666.054	1.204E-05	102152.9881	8453.1408	98802.1121

Figure 4.12: ETABS base reaction results after response spectrum scaling, showing that the dynamic base shear in both principal directions is approximately equal to the equivalent static seismic base shear.

The scaling factors are calculated as follows:

$$Scale_X = \frac{V_{EQX}}{V_{RSAX}} = \frac{6667.86}{4333.898} = 1.5385$$

$$Scale_Y = \frac{V_{EQY}}{V_{RSAY}} = \frac{6667.86}{4427.667} = 1.5059$$

After applying the required response spectrum scaling factors, the dynamic analysis results become consistent with the equivalent static seismic base shear requirement. The scaled dynamic results are then used for the design and verification of the lateral force-resisting system.

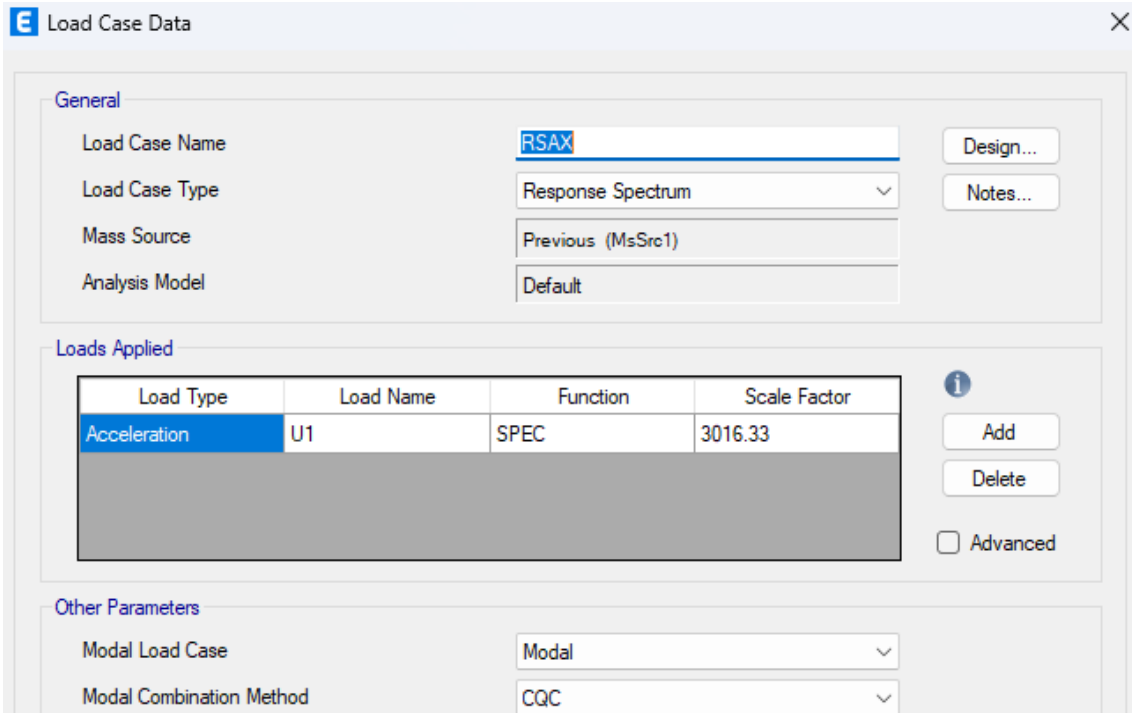

Figure 4.13: RSAX response spectrum load case definition in ETABS after applying the required scale factor in the U1 direction.

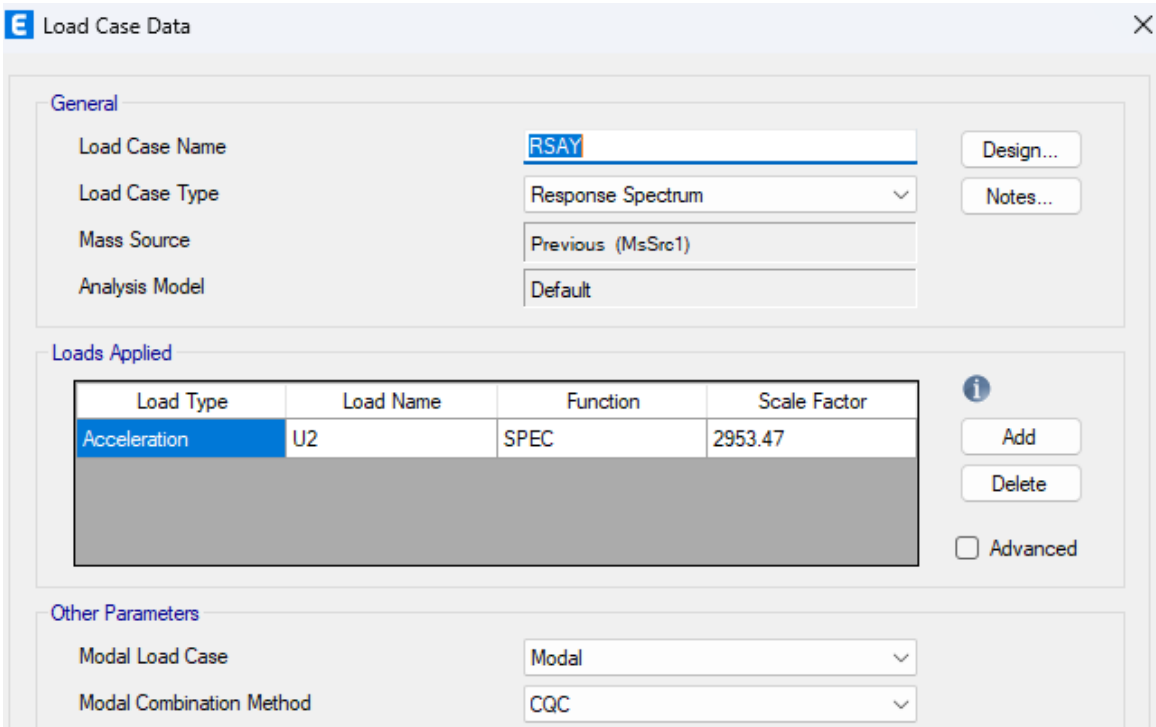

Figure 4.14: RSAY response spectrum load case definition in ETABS after applying the required scale factor in the U2 direction.

4.6 Story Shear Distribution and Lateral Force Transfer

The story shear distribution was reviewed to evaluate how the lateral seismic forces are accumulated and transferred through the height of the retrofitted seven-story reinforced concrete structure. This verification is important because the adopted rehabilitation strategy depends on the ability of the external reinforced concrete exoskeleton casing to collect the diaphragm inertia forces and transfer them safely toward the foundation level.

The scaled response spectrum cases, RSAX and RSAY, were used for this assessment. These cases represent the dynamic seismic response after applying the required base shear scaling factors discussed in Section 4.5. The ETABS story force output shows that the story shear increases gradually from the upper levels toward the base level, which is the expected trend for a multi-story lateral force-resisting system.

For the X-direction response spectrum case, RSAX, the story shear in the X direction increased from 60.10 kN at Story8 to 6663.24 kN at Story1. Similarly, for the Y-direction response spectrum case, RSAY, the story shear in the Y direction increased from 58.44 kN at Story8 to 6666.05 kN at Story1. The base-level story shear values are consistent with the scaled base reactions presented earlier, confirming that the lateral seismic demand is properly accumulated through the height of the structure and transferred to the base.

Table 4.11: Story shear distribution under the scaled response spectrum cases.

Story	RSAX shear in X-direction, kN	RSAY shear in Y-direction, kN
Story8	60.10	58.44
Story7	1355.90	1585.38
Story6	2679.55	3004.46
Story5	3914.86	4229.88
Story4	4992.10	5243.63
Story3	5861.67	6013.58
Story2	6426.14	6484.13
Story1	6663.24	6666.05

Story Forces

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Units: As Noted Hidden Columns: Yes Sort: None Story Forces

Filter: ([Story] = 'Story1' OR [Story] = 'Story2' OR [Story] = 'Story3' OR [Story] = 'Story4' OR [Story] = 'Story5' OR [Story] = 'Story6' O
Case] = 'RSAX') AND ([Case Type] = 'LinRespSpec')

Story	Output Case	Case Type	VX kN	VY kN	MX kN-m	MY kN-m
Story8	RSAX	LinRespSpec	60.1012	19.7191	48.1146	146.6469
Story7	RSAX	LinRespSpec	1355.8952	239.3204	836.7142	4633.1796
Story6	RSAX	LinRespSpec	2679.5499	419.0837	2175.5492	13220.8773
Story5	RSAX	LinRespSpec	3914.8644	466.2581	3649.222	25874.4151
Story4	RSAX	LinRespSpec	4992.0961	486.8529	5041.2769	41452.1449
Story3	RSAX	LinRespSpec	5861.6651	543.6891	6449.3618	59445.7454
Story2	RSAX	LinRespSpec	6426.1352	612.9078	8023.9749	79463.618
Story1	RSAX	LinRespSpec	6663.2415	651.2277	9598.1924	98291.047

Figure 4.15 (a): ETABS story force output under the scaled response spectrum load case RSAX, showing the accumulation of lateral seismic shear in the X direction along the building height.

Story Forces

File Edit Format-Filter-Sort Select Options

Units: As Noted Hidden Columns: Yes Sort: None Story Forces

Filter: ([Story] = 'Story1' OR [Story] = 'Story2' OR [Story] = 'Story3' OR [Story] = 'Story4' OR [Story] = 'Story5' OR [Story] = 'Story6' O
Case] = 'RSAY') AND ([Case Type] = 'LinRespSpec')

Story	Output Case	Case Type	VX kN	VY kN	MX kN-m	MY kN-m
Story8	RSAY	LinRespSpec	15.574	58.4404	142.5946	38.0005
Story7	RSAY	LinRespSpec	188.9904	1585.3823	5410.0328	658.9889
Story6	RSAY	LinRespSpec	309.9736	3004.4579	15048.8532	1644.2266
Story5	RSAY	LinRespSpec	378.4743	4229.8812	28698.0642	2822.6971
Story4	RSAY	LinRespSpec	434.8308	5243.6297	44990.641	4044.7376
Story3	RSAY	LinRespSpec	513.6812	6013.5836	63341.824	5363.6478
Story2	RSAY	LinRespSpec	593.8625	6484.1316	83415.4942	6895.6571
Story1	RSAY	LinRespSpec	637.7169	6666.054	102152.9881	8453.1408

Figure 4.15(b): ETABS story force output under the scaled response spectrum load case RSAY, showing the accumulation of lateral seismic shear in the Y direction along the building height.

The results confirm that the external reinforced concrete exoskeleton casing participates effectively in resisting and transferring the lateral seismic forces. The gradual increase in story shear toward the lower stories indicates a stable and continuous load path from the floor diaphragms to the vertical resisting elements and finally to the foundation

system. Therefore, the obtained story shear distribution supports the adequacy of the proposed strengthening configuration in providing a reliable lateral force-resisting mechanism for the retrofitted structure.

4.7 Story Drift Verification

The inter-story drift response of the retrofitted seven-story reinforced concrete structure was evaluated under the scaled response spectrum load cases. Story drift is a key seismic performance indicator because it represents the relative lateral displacement between two consecutive floor levels. Excessive drift may lead to damage in structural elements, non-structural components, façade systems, and diaphragm-to-wall connections.

Drift verification was performed of performed with the scaled response spectrum cases RSAX and RSAY. In ETABS the drift output reports the elastic story drift ratios for each story level, and for each principal horizontal direction too. Now because the response spectrum cases were previously scaled to satisfy the equivalent static base shear requirement , the drift values that were obtained are basically the controlling dynamic deformation response of the retrofitted structure.

For the X direction response spectrum case, RSAX , the maximum elastic story drift happened at Story 6, with a drift ratio of 0.002082. For the Y direction response spectrum case, RSAY, the maximum elastic story drift also happened at Story6, with a drift ratio of 0.001164. The clustering of the maximum drift around the upper middle portion of the structure, seems consistent with the combined effect of the added three-story vertical extension and the stiffness distribution supplied by the external reinforced concrete exoskeleton casing.

Table 4.5 summarizes the governing drift values obtained from the response spectrum analysis

.

Table 4.12: Maximum story drift verification under the scaled response spectrum cases.

Direction	Governing response spectrum case	Governing story	Elastic drift ratio	Amplified drift ratio	Allowable drift ratio	Verification status
X-direction	RSAX	Story6	0.002082	0.01041	0.020	Satisfactory
Y-direction	RSAY	Story6	0.001164	0.00582	0.020	Satisfactory

The results presented in **Table 4.9** indicate that the adopted modal basis is sufficient for the dynamic analysis.

For design-level drift verification, the elastic drift ratios were amplified using the deflection amplification factor. Based on the adopted seismic force-resisting system shown in the ASCE 7-16 seismic loading definition, the deflection amplification factor was taken as Cd = 5, while the seismic importance factor was taken as Ie = 1.0. Therefore, the amplified drift ratio in the X direction was calculated as 0.00937, and the amplified drift ratio in the Y direction was calculated as 0.00524.

Both amplified drift ratios are lower than the adopted allowable drift limit of 0.020. This confirms that the retrofitted structure satisfies the inter-story drift requirement in both principal horizontal directions.

Both amplified drift ratios are below the adopted allowable drift limit 0.020 . So yes it kinda confirms that the retrofitted structure meets the inter-story drift requirement in both main horizontal directions. The results also seem to show that the external RC exoskeleton casing gives enough lateral rigidity for controlling the seismic deformation demand , that is driven by the proposed vertical extension.

Story Drifts

File Edit Format-Filter-Sort Select Options

Units: As Noted Hidden Columns: Yes Sort: None Story Drifts

Filter: ([Output Case] = 'RSAX') AND ([Direction] = 'X')

Story	Output Case	Direction	Drift	X m	Y m
Story8	RSAX	X	4.2E-05	12.728	-3.5879
Story7	RSAX	X	0.001216	32.3032	2.694
Story6	RSAX	X	0.002082	32.3032	2.694
Story5	RSAX	X	0.001443	29.9204	2.694
Story4	RSAX	X	0.000298	32.3032	2.694
Story3	RSAX	X	0.001681	32.3032	2.694
Story2	RSAX	X	0.002046	32.3032	2.694
Story1	RSAX	X	0.001054	29.9204	2.694

Figure 4.16: ETABS story drift results under the scaled RSAX response spectrum case, showing the maximum X-direction drift demand at Story6.

Story Drifts

File Edit Format-Filter-Sort Select Options

Units: As Noted Hidden Columns: Yes Sort: None Story Drifts

Filter: ([Output Case] = 'RSAY') AND ([Direction] = 'Y')

Story	Output Case	Direction	Drift	X m	Y m
Story8	RSAY	Y	0.000203	-5.7697	1.8535
Story7	RSAY	Y	0.00077	32.3032	2.694
Story6	RSAY	Y	0.001164	32.3032	2.694
Story5	RSAY	Y	0.000884	29.9204	2.694
Story4	RSAY	Y	0.000459	32.3032	2.694
Story3	RSAY	Y	0.000893	32.3032	2.694
Story2	RSAY	Y	0.001138	29.9204	2.694
Story1	RSAY	Y	0.000587	32.3032	2.694

Figure 4.17: ETABS story drift results under the scaled RSAY response spectrum case, showing the maximum Y-direction drift demand at Story6.

The story drift verification confirms the effectiveness of the proposed external reinforced concrete exoskeleton casing in limiting lateral deformation. Although the vertical extension increases the seismic mass of the building, the added perimeter wall system provides adequate stiffness and maintains the drift response within the adopted code-based limit. Accordingly, the drift performance of the retrofitted structure is considered satisfactory.

4.8 Diaphragm Displacement and Torsional Response Verification

The diaphragm displacement response of the retrofitted seven-story reinforced concrete structure was reviewed to evaluate the global lateral deformation and torsional behavior of the analytical model. This verification is important because the proposed strengthening system depends on the ability of the floor diaphragms to collect seismic inertia forces and transfer them to the external reinforced concrete exoskeleton casing. The center of mass displacement results for the diaphragm were pulled from the scaled response spectrum scenarios RSAX and RSAY. Here , RSAX is the dynamic seismic response, specifically for the X direction. In the same way , RSAY is the dynamic seismic response for the Y direction. The displacement readings were checked at the diaphragm center of mass for each floor level so it could support the overall deformation trend and also spot any notable torsional kind of behavior.

For the RSAX response spectrum scenario , the highest diaphragm displacement showed up at the top story. That is, a horizontal displacement of 0.791 mm was observed in the X direction at Story8. The linked transverse displacement came out to 0.310 mm , and the rotational response about the vertical axis was 3.8×10^{-5} rad. Meanwhile for RSAY , the maximum diaphragm displacement also appeared at the upper story level. The horizontal displacement in the Y direction at Story8 was 2.157 mm. The associated transverse displacement was 0.221 mm, and the rotational response about the vertical axis measured 2.9×10^{-5} rad.

Table 4.13: Summary of diaphragm center of mass displacement response under response spectrum cases.

Response spectrum case	Governing story	UX, mm	UY, mm	RZ, rad	Governing displacement direction
RSAX	Story8	0.791	0.310	3.8×10^{-5}	X-direction
RSAY	Story8	0.221	2.157	2.9×10^{-5}	Y-direction

The results show that the diaphragm displacement response increases gradually toward the upper levels, which is the expected behavior for a multi-story building subjected to lateral seismic excitation. The maximum displacements occurred at the top level of the structure, while the lower stories had smaller displacement values due to their proximity to the base and the higher restraint provided by the vertical lateral force-resisting system.

The rotational response values about the vertical axis were fairly small compared with the translational displacement response. This suggests that the external RC exoskeleton casing gives a stable and ongoing lateral load resisting mechanism and it helps to curb excessive diaphragm rotation. The results also confirm that the diaphragms can channel the lateral inertia forces into the external RC wall system without triggering excessive global torsional deformation .

Accordingly, the diaphragm displacement and torsional response verification supports the adequacy of the proposed external RC exoskeleton casing. The strengthened system provides sufficient lateral stiffness and maintains the global displacement response within a controlled range under the scaled response spectrum load cases.

4.9 Summary of Analytical Findings

This chapter presented the analytical verification of the retrofitted reinforced concrete building after the proposed vertical extension. The verification was carried out using a three-dimensional ETABS model that included the existing four-story reinforced concrete structure, the proposed three-story vertical extension, and the external reinforced concrete exoskeleton casing. The main objective of the analysis was to evaluate whether the proposed strengthening system provides a reliable gravity and lateral load-resisting mechanism for the upgraded seven-story configuration.

The analytical model definitions were first reviewed, including story levels, material properties, shell section assignments, stiffness modifiers, load patterns, mass source definition, seismic parameters, and response spectrum load cases.

These definitions confirmed that the model distinguishes between the aged existing reinforced concrete elements and the newly added external RC exoskeleton system. Reduced stiffness modifiers were assigned to the existing RC walls to avoid overestimating their contribution, while the new external RC walls were modeled to represent the main strengthening and lateral force-resisting system.

The modal analysis results confirmed that the adopted number of modes was sufficient to capture the dynamic behavior of the structure. The cumulative modal mass participation exceeded the commonly accepted 90% requirement in the principal horizontal directions, confirming that the response spectrum analysis was based on an adequate representation of the building dynamic response.

The equivalent static and dynamic base shear comparison showed that the response spectrum cases were properly scaled to satisfy the equivalent static base shear requirement. After scaling, the RSAX and RSAY base shear values became consistent with the corresponding EQX and EQY reference values. This confirms that the dynamic seismic demand used in the subsequent verification checks was not underestimated.
The story shear distribution showed a gradual increase in lateral shear from the upper stories toward the base level. This trend confirms the expected accumulation of seismic forces through the height of the building and indicates that the external RC exoskeleton casing provides a continuous and stable lateral load path. The base-level story shears obtained from the scaled response spectrum cases were consistent with the scaled base reactions.

The story drift verification showed that the maximum elastic drift ratios occurred at Story6 under both principal response spectrum directions. After applying the deflection amplification factor, the amplified drift ratios remained below the adopted allowable drift limit of 0.020. This confirms that the retrofitted structure satisfies the inter-story drift requirement and that the proposed external RC exoskeleton casing provides sufficient lateral stiffness to control seismic deformation.

The diaphragm displacement and torsional response verification also showed acceptable behavior. The maximum diaphragm displacements occurred at the top story level, which is expected for a multi-story building under lateral seismic excitation. The rotational response about the vertical axis remained small compared with the translational displacement response, indicating that excessive global torsional deformation was not observed in the analytical model.

Overall, the analytical findings confirm the effectiveness of the proposed external reinforced concrete exoskeleton casing in improving the seismic performance of the vertically extended building. The strengthening system provides an alternative and continuous load path for gravity and lateral demands, limits the deformation response, and reduces the demand imposed directly on the deteriorated internal legacy structure. Therefore, the proposed retrofitting strategy is considered structurally adequate for supporting the planned vertical extension within the scope of the linear elastic ETABS analysis performed in this study.

4.10 Techno-Economic Feasibility and Cost-Benefit Analysis

To provide a comprehensive justification for the proposed rehabilitation strategy, a techno-economic feasibility study was conducted, comparing structural retrofitting via an external reinforced concrete exoskeleton casing against complete demolition and full reconstruction. In highly constrained, historic urban fabrics such as Jerusalem, structural decisions are strongly governed by economic constraints, stringent municipal permitting procedures, and the socioeconomic impact on existing residents.

4.10.1 Comparison Framework and Baseline Parameters

The feasibility analysis evaluates two primary intervention scenarios for the 60-year-old residential asset containing 21 existing apartment units with a baseline area of 1,600 m^2:

- **Option A (Demolition and Full Reconstruction):** Complete demolition of the existing 4-story structure, relocation of the current residents to temporary accommodation for a projected 36-month construction cycle, and the construction of a modern 7-story reinforced concrete building with a total gross area of 3,625 m^2.

- **Option B (Structural Retrofitting with Residents in Place):** Retaining and strengthening the existing 4-story building using the perimeter RC exoskeleton casing, integrating a 25 m² lateral expansion per existing apartment (totaling 525 m² of integrated floor extensions), and adding three new vertical stories (1,500 m²) to yield the identical final building area of 3,625 m². All construction interventions are executed externally, eliminating the need to vacate the occupants.

The economic calculations are based on prevailing market rates in Jerusalem, evaluated on a conversion rate of USD 1 = NIS 3.066.

4.10.2 Cost Breakdown and Comparative Evaluation

The itemized capital expenditures, logistics costs, and preliminary total investments for both development pathways are summarized in Table 4.14.

Table 4.14: Economic feasibility comparison between demolition/reconstruction and external retrofitting.

Cost Component	Option A: Demolition & Reconstruction	Option B: Structural Retrofitting with Residents in Place
Existing Building Area	1,600 m²	1,600 m²
Number of Existing Apartments	21	21
Additional Area to Existing Apartments (21 x 25 m²)	525 m²	525 m²
New Residential Construction Area	1,500 m²	1,500 m²
Final Building Area	3,625 m²	3,625 m²
Demolition of Existing Building	$489,237	-
Temporary Housing - 21 Apartments x 36 Months x $3,262/month	$2,465,753	-
Residents' Relocation Cost - 21 x $1,957	$41,096	-
Residents' Return Cost - 21 x $1,957	$41,096	-
Permits and Regulatory Fees	$163,079	$163,079*
Reconstruction / New Construction - 3,625 m² x $2,283/m²	$8,276,256	-
Retrofitting & 25 m² Extension per Existing Apartment - 525 m² x $2,609/m²	-	$1,369,863
Construction of New Apartments - 1,500 m² x $2,283/m²	-	$3,424,658
Total Estimated Project Cost	**$11,476,517**	**$4,957,599***
Estimated Cost Difference	-	$6,518,917
Estimated Cost Reduction Relative to Option A		56.8%

*Note: Preliminary retrofit estimate. If the rate of $2,609/m² accounts strictly for the 525 m² perimeter extension and excludes global seismic retrofitting of the original 1,600 m² core, localized structural repair costs should be integrated into the final ledger.

4.10.3 Discussion of Economic and Socioeconomic Outcomes

The financial assessment demonstrates clear economic advantages for the structural retrofitting approach:

1. **Capital Expenditure Optimization:** Option B achieves a direct cost reduction of approximately 56.8%, generating an estimated financial saving of $6,518,917 compared to complete replacement.
2. **Elimination of Relocation Overhead:** In the reconstruction scenario (Option A), non-structural ancillary expenses—comprising temporary rent subsidies ($2,465,753) and dual relocation logistics ($82,192)—constitute over 22% of the total budget. The exterior exoskeleton methodology bypasses these secondary costs entirely by maintaining full residential occupancy throughout the project lifespan.
3. **Regulatory and Environmental Sustainability:** Beyond direct monetary savings, preserving the existing concrete substructure minimizes municipal rezoning disputes and drastically shortens construction schedules. Furthermore, retrofitting averts the disposal of thousands of metric tons of demolition debris, significantly lowering the embodied carbon footprint of the urban vertical densification process.

Chapter 5: Conclusions and Recommendations

5.1 Introduction

This chapter presents the main conclusions and recommendations derived from the analytical investigation carried out in this study. The research focused on evaluating the structural and seismic behavior of an existing reinforced concrete building proposed for vertical extension and retrofitted using an external reinforced concrete exoskeleton casing. The original building consists of four stories, while the proposed upgraded configuration includes three additional upper stories, resulting in a seven-story structure.

The study was worked out inside a code oriented structural engineering framework, more or less. The seismic loading and the response spectrum verification were taken from the provisions of ASCE/SEI 7-16, and the reinforced concrete design plus detailing way of thinking also followed ACI 318-19. Since this project concerns an existing reinforced concrete building, the general ideas behind seismic evaluation, rehabilitation, and the assessment of older concrete structures were interpreted too, with reference to ASCE/SEI 41-17, and ACI 562-19.

The main goal was not just about verifying whether the vertically extended building is structurally good, or adequate. Instead, it tried to check if the proposed external reinforced concrete exoskeleton casing could act as a trustworthy alternate load path for gravity and seismic actions , at the same time lowering the direct demand placed on the deteriorated internal legacy structure. That aim matters a lot for existing buildings where the original system might not have been conceived for any future vertical extension, and it likely wasn't designed for the seismic demand tied to the extra mass and height.

The conclusions in this chapter come from that three dimensional ETABS analytical model, the seismic verification outcomes, and the more refined structural drawings made for the proposed strengthening system.

The analytical check included modal analysis, response spectrum analysis, an equivalent static versus dynamic base shear comparison , story shear distribution, the story drift verification, diaphragm displacement response , and an appraisal of the torsional behavior.

5.2 Summary of the Study

The investigated structure kind a shows up a lot in real reinforced concrete buildings, where you need vertical extension but the original structural system was never tuned to take the added gravity and seismic demands from the extra stories. In this kind of situation, if you just push the new loads straight into the existing columns, beams, and walls it can cause severe overstressing in the old system, particularly if you have material aging and stiffness degradation, and also limited reinforcement detailing, plus possible deterioration that was kind of ignored earlier.

So, to cope with that, an external reinforced concrete exoskeleton casing was chosen as the main retrofitting and strengthening concept. The proposed arrangement basically wraps the building from the outside, and it behaves as a new external structural system that is linked to the floor diaphragms. With this setup the floor diaphragms can gather the seismic inertia forces, then channel them to the external RC wall system, and from there the forces go down toward the foundation level.

Because of that, the study looked at two key points. First, it did an analytical checking of the retrofitted structure under both gravity and seismic effects, using ETABS. Second, it produced the detailed structural drawings for the strengthening system that was proposed. In the end, the work did not only stay in the conceptual or analytical realm, it also turned the retrofitting approach into a workable, practical structural design solution.

5.3 Summary of the Analytical Verification

The analytical model got built up as a three-dimensional finite element model within ETABS. In there, it covered the existing four-story reinforced concrete structure, plus the planned three-story vertical extension and also the external reinforced concrete exoskeleton casing, kind of all together. The modeling setup then defined story levels, material properties, shell section assignments, stiffness modifiers, load patterns, mass source definition, seismic parameters, and those response spectrum load cases too .

A key modeling consideration, was the distinction between the aged existing reinforced concrete elements and the recently added strengthening components. Lower stiffness modifiers were assigned to the existing reinforced concrete walls , to reflect cracking, aging and maybe a bit of stiffness degradation. Meanwhile , the newly added external reinforced concrete walls were modeled with greater effective stiffness values, and even then they still had to consider cracked flexural behavior. This way of doing things matches the broader intent, of not overestimating how much the existing structural elements actually contribute in rehabilitation studies .

The seismic analysis was then conducted using both equivalent static and response spectrum methods. The equivalent static procedure was treated like a reference for base shear comparison, and the response spectrum analysis was used to check the dynamic behavior of the structure in the two main horizontal directions. The response spectrum cases were scaled so they could meet the equivalent static base shear requirement , meaning the dynamic seismic demand used for design verification would not end up unconservatively low .

In addition to analytical verification, the study included the preparation of detailed structural design drawings for the proposed external reinforced concrete exoskeleton casing and related strengthening components. As a result, the study did not remain at the conceptual or numerical modeling stage. Instead, the proposed strengthening strategy was developed into a practical design solution supported by analytical evidence and construction-oriented drawings.

5.4 Summary of Analytical Findings

he modal analysis results confirmed that the adopted number of modes was sufficient to represent the dynamic behavior of the retrofitted structure. The cumulative modal mass participation exceeded the commonly accepted 90% requirement in the principal horizontal directions. This indicates that the response spectrum analysis was based on an adequate representation of the building dynamic characteristics.

The comparison between equivalent static and response spectrum base shear results confirmed that the dynamic response spectrum cases were properly scaled. After scaling, the response spectrum base shear values became consistent with the equivalent static seismic base shear demand. This step was essential because it ensured that the dynamic seismic forces used in story shear, drift, displacement, and design verification were not underestimated.

So basically the story shear distribution, showed this gradual increase in lateral shear from the upper stories down to the base, like it kinda piles up over the height. Theres a reason for it, its consistent with the way seismic forces are expected to accumulate in a multi story building, yes. The base level story shears that came from the scaled response spectrum cases were about in line with the scaled base reactions. That kind of match, confirms that the lateral demand was properly built up through the height and then handed off toward the foundation level.

Then the story drift verification part, it showed that the maximum elastic drift ratios showed up around Story6 under the response spectrum load cases. After applying the deflection amplification factor, the amplified drift ratios stayed below the adopted allowable drift ratio of 0.020 in both principal directions, so nothing crazy happened there. The maximum amplified drift ratio was roughly 0.00937 in the X direction and 0.00524 in the Y direction. Overall, these numbers suggest the external reinforced concrete exoskeleton casing delivered enough lateral stiffness to manage the seismic deformation demand.

The diaphragm displacement and torsional response verification also showed acceptable behavior. The maximum diaphragm displacements occurred at the top story level, which is expected for a multi-story structure subjected to lateral seismic excitation. The maximum center of mass displacement was 0.791 mm in the X direction under RSAX and 2.157 mm in the Y direction under RSAY. The rotational response about the vertical axis remained small compared with the translational response, indicating that excessive global torsional deformation was not observed in the analytical model.

5.5 Engineering Interpretation of the Structural Behavior

The analytical findings show that the biggest contribution of the proposed strengthening system is really about making a new external load path. In the unstrengthened situation, the existing internal structural arrangement would have basically had to take up the extra gravity and seismic requirements that come from the vertical extension. But once the external reinforced concrete exoskeleton casing is added, the whole global response of the building shifts, because a new stiff perimeter system gets introduced, and this system ends up taking part in resisting both the lateral effects and the gravity demands.

That external casing helps the seismic response in a few practical ways. To start, it boosts the overall lateral stiffness of the building, which in turn helps lower the inter-story drift. Next, it offers a continuous vertical load transfer route, so that lateral forces coming from the floor diaphragms can move all the way to the foundation level through the new external reinforced concrete wall system. Finally, it reduces the reliance on the deteriorated internal legacy structure, and that point matters a lot, since the original building might not have enough strength, stiffness, or ductile detailing to safely handle the increased seismic demand.

The diaphragm action is a critical component of the proposed strengthening concept, like it really does most of the work in a way. The floor slabs act as horizontal collectors and they transfer earthquake-induced inertia forces from the floor masses toward the external reinforced concrete exoskeleton walls.

So in the end, the effectiveness of the strengthening system depends not only on how strong and how stiff those new external walls are, but also on whether the connection and force transfer mechanism between the diaphragms and the external casing is actually adequate.

The controlled drift and displacement results suggest that the proposed exoskeleton system provides an effective balance between stiffness and deformation control, even if it sounds almost too neat. Sometimes excessive stiffness concentration can lead to undesirable force attraction, and also local demand concentration. Still, the obtained response indicates that the system provides adequate stiffness without producing excessive global torsional deformation. This, in turn, supports the suitability of the proposed external reinforced concrete casing as a global strengthening solution for the vertically extended structure.

5.6 Detailed Design and Structural Drawings

In addition to the analytical verification, the study also included the preparation of detailed structural design drawings for the proposed external reinforced concrete exoskeleton casing and a few related strengthening components . So the result of this study is not limited to simply evaluating the seismic behavior of the retrofitted building . It goes further, and it covers the development of a practical structural design solution that can be used as a starting point for implementation, after final engineering review approval and site coordination.

The prepared drawings, they show the structural representation of the proposed strengthening system, including the arrangement of the external reinforced concrete wall system , its connection with the existing building, the strengthening layout, and the intended load transfer mechanism between the diaphragms, the external casing, and the supporting foundation system . In other words , these drawings convert the analytical assumptions into construction-oriented details, and they offer a workable framework for applying the proposed retrofitting strategy.

Because the analytical verification is integrated with the detailed design , the practical value of the study becomes stronger . The external reinforced concrete exoskeleton casing was not only checked numerically through the ETABS model, but it was also expanded into a detailed design package . This basically confirms that the proposed retrofitting concept is supported by both analytical evidence and design documentation.

5.7 Conclusions

Based on the analytical investigation and detailed design work performed in this study, the following conclusions can be drawn:

1. The proposed external reinforced concrete exoskeleton casing is an effective strengthening strategy for supporting the vertical extension of the investigated reinforced concrete building.
2. The three-dimensional ETABS model provided a consistent analytical representation of the existing structure, the proposed upper extension, and the external reinforced concrete exoskeleton casing.
3. The use of different stiffness modifiers for the existing and newly added reinforced concrete elements was necessary to represent the difference between the aged legacy structure and the new strengthening system.
4. The modal analysis results confirmed that the cumulative modal mass participation exceeded the required level in the principal horizontal directions, indicating that the dynamic behavior of the structure was adequately captured.

   The response spectrum base shear values were successfully scaled to satisfy the equivalent static base shear requirement, ensuring that the seismic demand used in the verification checks was not underestimated.
5. The story shear distribution confirmed the presence of a continuous and logical lateral force transfer mechanism from the upper stories toward the base level.
6. The story drift verification showed that the amplified drift ratios in both principal directions remained below the adopted allowable drift limit of 0.020.
7. The diaphragm displacement and torsional response verification indicated that the global displacement response was controlled and that excessive torsional deformation

was not observed.

8. The external reinforced concrete exoskeleton casing reduces the demand imposed directly on the old internal structural system by providing an alternative and stiffer load-resisting path.
9. The preparation of detailed structural drawings demonstrates that the proposed strengthening system was developed beyond the analytical stage and translated into a practical design solution.
10. Within the scope of the linear elastic ETABS analysis and the adopted design assumptions, the proposed retrofitting system can be considered structurally adequate for supporting the planned vertical extension.

5.8 Recommendations for Practical Implementation

Even if the analytical and design results actually do indicate that the proposed external reinforced concrete exoskeleton casing is adequate , there are still a few practical items that really should be handled a lot more carefully before any construction starts.

First , the detailed structural drawings that were prepared here must go through a final engineering review , plus coordination and formal approval, before the work can begin. That means revisiting the compatibility between the analytical assumptions and the reinforcement detailing, also the connection details , the actual site conditions, and the construction constraints. If anything doesn't line up—like the existing building condition compared to the design drawings—then it should be fixed and clarified before construction takes place.

Also, special care is needed for the connection between the existing structure and the new external reinforced concrete casing. These connections are what enable transfer of diaphragm forces , and they help establish interaction between the old structural system and the new one. Anchors, dowels, collector elements , and the interface shear transfer mechanisms should be carefully verified both during the design review , and again during construction, even if the documents look correct at first glance.

The foundation system that is essentially supporting the external reinforced concrete exoskeleton casing ought to be checked , for the extra gravity loads as well as seismic base shear , overturning moments sliding behavior , bearing pressure , and eventual settlement. Since the outside casing tends to pick up a large chunk of the lateral demand , the foundation system becomes a key, sorta decisive component of the entire strengthening scheme.

Also the construction sequence needs to be laid out ahead of time , in a very deliberate way. The placement of the external reinforced concrete casing , the connection tasks , and then the building of the extra upper stories can , for a while , modify the load path and the stiffness distribution of the structure. So, temporary support needs, the staged construction effects, and safe load transfer during construction should all be reviewed , before any work goes too far.

The prepared drawings should also be coordinated with architectural, mechanical, electrical, and site constraints. Since the external exoskeleton surrounds the existing building, coordination is required to avoid conflicts with openings, façades, services, access routes, and existing structural elements.

Quality control during construction should be strictly applied to ensure that the constructed system matches the design intent shown in the detailed drawings. This includes verifying reinforcement placement, anchorage lengths, concrete strength, surface preparation, curing conditions, and inspection of the interface between existing and new reinforced concrete elements.

5.9 Limitations of the Study

Although the study included analytical verification and detailed structural design drawings, the findings should be interpreted within the limitations of the adopted modeling and verification approach.

The global structural behavior was evaluated mainly using a linear elastic ETABS model. This approach is suitable for code-based design verification; however, it does not fully capture nonlinear material behavior, progressive cracking, reinforcement yielding, connection slip, local damage mechanisms, or post-elastic redistribution of internal forces.

The study did not really include a detailed non linear pushover analysis, or a nonlinear time history analysis, not in any deep way. However, these kinds of analyses could give more insight into inelastic behavior and the overall yielding progression, plus how plastic hinges actually form, what ductility demand looks like, and how well collapse prevention works for the retrofitted structure.

Also, the way the existing building connects with the external reinforced concrete casing was captured in a simplified form, in the global analytical model. In real use, connection stiffness, anchorage performance, interface shear transfer, and even possible slip between the old and the new concrete faces, could end up changing the load path , and how forces really move from one to the other.

Finally, soil structure interaction was not modeled explicitly with much detail. Foundation flexibility, soil stiffness, differential settlement, and nonlinear soil behavior could influence the global seismic response, especially since the external reinforced concrete exoskeleton carries major gravity and lateral actions toward the foundation system.

In the global analysis, the construction stage effects were sort of not fully modeled , yeah. Like the real behavior of the building while strengthening is ongoing , during connection installation , and also when the vertical extension is being added , may be a bit different from how things end up in the final completed structural configuration. So, the construction sequencing should be reviewed separately before any actual implementation happens.

The detailed drawings were prepared using the structural information that was available and the analytical assumptions that were adopted. Still though, even after that, the final on-site work should be preceded by a field check of the existing building dimensions , material properties , reinforcement conditions , and foundation details.

5.10 Suggestions for Future Work

Future research may extend the present study by performing nonlinear pushover analysis to evaluate the expected inelastic behavior of the retrofitted structure and identify potential weak zones. This type of analysis would help assess the sequence of yielding and the global ductility capacity of the strengthened building.

Nonlinear time-history analysis could be done using selected ground motion records that are compatible with the seismicity of the project area. In that way you get a more nuanced evaluation of the seismic response when the structure is really subjected to earthquake excitation, and it also makes it possible to compare the results with the response spectrum approach.

A more focused numerical investigation of the connection system between the existing structure and the external reinforced concrete exoskeleton casing is recommended, basically it should cover the whole arrangement of anchors , dowels , collector elements, plus the interface shear transfer and any potential slip between the old and the new concrete surfaces.

Future work may also extend toward a detailed foundation and soil–structure interaction study for the external reinforced concrete exoskeleton system, which matters because the proposed strengthening scheme brings extra vertical loads and lateral actions right at the foundation level.

A comparative study between various retrofitting strategies might be useful as well, in a sort of “apples vs oranges” way, even if the details overlap a bit. The proposed external reinforced concrete exoskeleton casing could be weighed against other strengthening techniques like internal shear walls, steel bracing systems, reinforced concrete jacketing, FRP strengthening, or even local member strengthening. Doing that kind of comparison would help judge the relative efficiency, overall cost, constructability , how much it changes the architecture, and the seismic performance you can expect from each option, sort of in one coherent picture.

Finally , once everything is implemented, it is recommended to run field monitoring. The goal is to check whether the analytical assumptions actually hold up and to confirm what the strengthened building really does under service conditions. Instrumentation can offer valuable data on the displacement response, how loads get transferred, long-term performance, and the interaction between the existing structure and this new external reinforced concrete exoskeleton casing, rather than relying only on the models.

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