STRUCTURE

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CONCRETE

Renovating the Egyptian Theater

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Image courtesy of Mike Hume at historictheatrephotos.com.

The seismic retrofit of the Egyptian Theatre in Los Angeles serves as a model for balancing modern engineering demands with the preservation of architectural heritage.

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The historic Egyptian Theatre by Sid Grauman was designed as one of the great movie palaces in a Revival-Egyptian style in 1921 and opened to the public in late 1922 by hosting the world's first red carpet movie premiere. The building's original structure consisted of reinforced concrete frames with hollow clay tile infills at all interior and exterior walls. Prior earthquakes have resulted in the collapse of similar building types. The City of Los Angeles passed an ordinance requiring the demolition or analysis and possible retrofit of concrete frame buildings designed prior to January 1977. The design team, led by architects Studio 440 and structural engineers Structural Focus eagerly got to work to retrofit this landmark. Work on the Egyptian Theatre, which is listed on the National Register of Historic Places, was performed according to the City's ordinance and the California Historical Building Code. During construction, unforeseen conditions required the design and construction teams to collaborate towards flexible solutions to accomplish an effective retrofit and rehabilitation. The Theatre re-opened to the public in late 2023.

The Theatre is a mostly single-story building with an attached three-story portion at the southwest corner. The building is approximately 200 feet long, 130 feet wide, and 64 feet tall. The concrete roof structure is multi-tiered and supported by a series of concrete trusses to create an unobstructed interior with minimal interior columns. Concrete beam and column frames form the perimeter walls and all columns are generally supported on spread footings (Fig. 1).

The Theatre received several renovations and changes over the years, including modification of the proscenium (the dividing line between the stage and auditorium) structure to increase the viewable stage width for a wide projection screen and a renovation and voluntary seismic retrofit in 1997/1998 following significant damage from the 1994 Northridge earthquake.

PROJECT TEAM

Owner: Netflix, Los Angeles, CA
Structural Engineer of Record: Structural Focus, Gardena, CA
Architect (General): Studio 440, Hollywood, CA
Preservation Architect: Historic Resources Group, Pasadena, CA
Mechanical, Electrical, Plumbing Engineer: Syska Hennessy, Los Angeles, CA
Geotechnical Engineer: Geotechnologies, Inc., Glendale, CA
General Contractor: Whiting-Turner, Irvine, CA
Construction Manager: Lincoln Property Company, El Segundo, CA

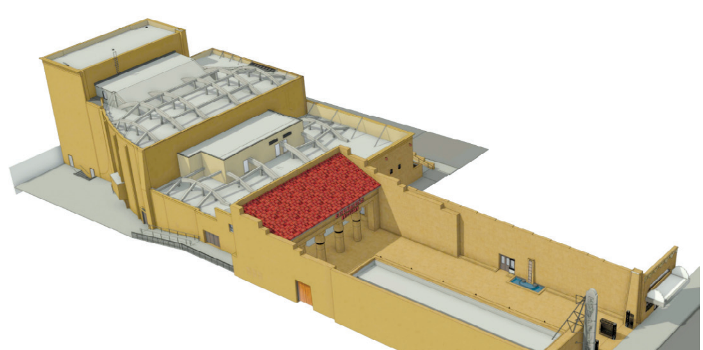

Fig. 1. The Egyptian Theater has a sloping mid-level roof over an exterior portico entry, a low flat roof over the interior lobby area, a mid-level flat roof over the auditorium, and a high flat roof over the stage.

City of Los Angeles Retrofit Ordinance

Strong earthquakes have caused severe damage to concrete buildings designed and constructed without ductile detailing requirements implemented in the 1976 Los Angeles City Building Code, or similar codes from that period. The City of Los Angeles implemented an ordinance that required the compliance, retrofit, or demolition of concrete buildings permitted before January 1977 within city limits, with exceptions for detached single-family dwellings or detached duplexes. The prior 1997/1998 voluntary seismic retrofit of the Theatre was determined to be inadequate for the requirements of this ordinance. The team proceeded with an ASCE 41 approach to meet the ordinance requirements.

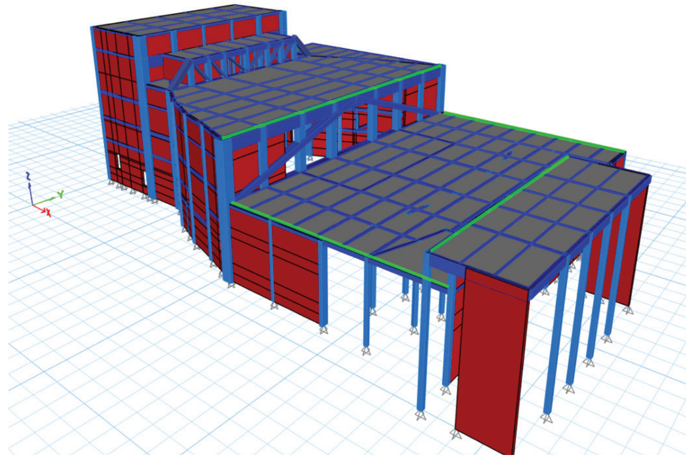

Fig. 2. As shown in this Egyptian Theatre ETABS Model, original concrete roof slabs are in gray; new or previous retrofit concrete shear walls are in red; original non-ductile concrete frame columns are in light blue; original non-ductile concrete frame beams are in dark blue; and new collectors are green.

Material Testing and Condition Assessment

The design team located detailed original architectural and structural drawings of the Theatre and the structural drawings of the previous voluntary retrofit which were invaluable. However, despite having the detailed drawings, a major challenge faced at the project's start was the lack of material properties in the original documents. A material testing program was prepared based on the ASCE 41-17 Comprehensive Data Collection Requirements and according to the LADBS Information Bulletin P/BC 2020-153 to obtain the concrete and reinforcing strength of the original construction. As part of the material testing, the height and spacing of the deformations on the reinforcing bars were recorded to calculate the rebar development and splice lengths using guidance by the Concrete Reinforcing Steel Institute. A total of 72 concrete cores and 15 rebar samples were tested to determine the expected strength, lower bound strength, and applicable knowledge factor based on the coefficient of variation for each member type according to ASCE 41-17. These final values were used in the analysis and design. Foundation testing was delayed until construction due to inaccessibility and an assumed foundation material strength from ASCE 41-17 based on the building's age was used during design and confirmed by testing during construction.

ASCE 41-17 and the LADBS ordinance required a Visual Condition Assessment of the building when performing a Tier 3 Risk Category III evaluation because the original and previous voluntary retrofit drawings of the Theatre contain detailed information about the member sizes, configurations, and reinforcing. To comply with the required Visual Condition Assessment quantities, all locations of material testing and an additional 40 original beams and 10 original columns were specified to be visually inspected for damage, degradation, and plumbness. The condition assessment verified the accuracy of the construction documents and found limited concrete damage and cracking which was not unexpected for a 100-year-old structure.

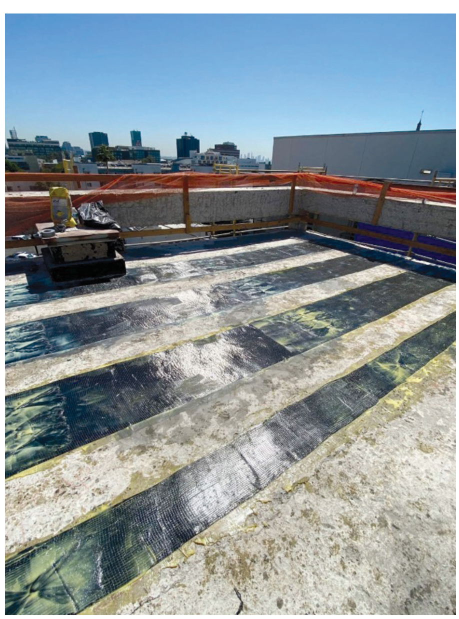

Fig. 3. Early coordination between Structural Focus and various FRP suppliers was required to ensure the diaphragm FRP strengthening design requirements specified on the structural drawings were feasible.

Project Retrofit Scope

Structural Focus' experience with seismic retrofits of similar non-ductile concrete buildings informed the decision to avoid using any of the existing non-ductile concrete frame system as a vertical lateral force resisting system. A proposed seismic retrofit system of new concrete shear walls was developed, and the behavior of the building with the proposed seismic retrofit upgrades was analyzed in ETABS using a linear dynamic response-spectrum-based modal analysis procedure (Fig. 2). To confirm a linear dynamic analysis was acceptable, ASCE 41-17 section 7.3.1.1 was followed and required the new lateral force resisting system did not have in-plane and out-of-plane discontinuities. Section 7.3.1.1 allows a linear dynamic analysis with prescribed limitations if a weak-story or torsional strength irregularity is present. The layout of the new lateral force resisting system avoided a weak-story irregularity, but a torsional strength irregularity was confirmed to exist in the east-west direction; however, the DCRs in that direction were confirmed to be below the prescribed limitations defined that allows Linear Dynamic Analysis to be utilized.

To create diaphragm continuity across the various roof elevations, new concrete diaphragm transfer walls were designed as deformation-controlled elements to avoid adding interior shear walls that would interrupt the auditorium area. Utilizing the analysis results from the linear dynamic model, the shear and flexural strengths of the existing concrete diaphragms were analyzed per ASCE 41 as deformation-controlled actions. While the flexural strengths were sufficient, the existing shear strength of the lightly reinforced 3-inch-thick diaphragms was deficient adjacent to various shear wall lines. Fiber-reinforced polymer (FRP) diaphragm shear strengthening demands were determined per ASCE 41 as deformation-controlled actions and specified on the structural drawings for deferred submittal design (Fig. 3).

The diaphragm connections to the shear walls were then analyzed per ASCE 41 as force-controlled actions and found to be deficient. To mitigate the deficiencies, new collectors were designed to increase the shear transfer length with the addition of FRP between the diaphragm and collectors to increase the shear transfer strength (Fig. 2). The new collectors and FRP shear transfer strengthening were both designed as force-controlled actions. At limited areas, the compressive strength of the existing concrete beams was adequate for the collector compression demands but due to inadequate reinforcing quantity and development the collector tension demands required adding FRP tension collector strengthening on the existing beams. The FRP

Fig. 4. During shear wall reinforcing, the installation of rebar dowels required frequent coordination despite the design considerations because the existing reinforcing proved challenging to avoid. Filling abandoned drilled holes as the drilling proceeded was important to avoid losing excessive capacity of the existing concrete members.

tension collector and shear transfer demands were analyzed and specified on the structural drawings as force-controlled actions for deferred submittal design after coordination with various FRP suppliers to confirm feasibility of design requirements prior to bidding.

The retrofit design continued with the analysis of the new and previous retrofit concrete shear walls for in-plane shear and flexure as deformation-controlled elements per ASCE 41. Each individual wall pier (bounded by existing beams and columns) was analyzed and designed/strengthened with appropriate m-factors (~1.75 to 2.25 for BSE-1E and ~2.25 to 3.25 for BSE-2E). The existing concrete walls required shear strengthening, which was feasible using FRP. For walls requiring FRP strengthening, the required equivalent horizontal reinforcing with 60 ksi yield strength was specified in the structural drawings along with the existing material strengths and the classification of the member action for deferred submittal design.

The new and existing concrete shear walls were analyzed for out-of-plane shear and flexure as force-controlled elements per ASCE 41. Infills were determined to span either horizontally between columns or vertically between beams based on the available continuity of the reinforcing and the out-of-plane aspect ratios of the wall infills. The existing columns and beams were analyzed for the appropriate out-of-plane loads where required and confirmed to be adequate without any added strengthening.

Continuity of the shear walls for in-plane and out-of-plane demands through the existing concrete beams and columns was achieved by doweling rebar through the existing concrete members (Fig. 4). The strength and shear friction transfer of these bars was critical for the intended shear wall behavior to act as single large wall panels per wall elevation. The design utilized large diameter bars for these dowels to reduce the number of drilled holes and the likelihood of hitting/damaging existing reinforcing bars in the existing concrete members. At shear wall special boundary zones which continued through existing beams, it was selected to demolish the length of the beams where the boundary zone occurred while keeping the existing beam reinforcing intact. This enabled the boundary zone reinforcing to be placed continuously through the beam and the portion of beam demolished was then replaced by new concrete. Temporary shoring of the existing beam was required while the ends were demolished and was accomplished with small steel pipe members that were eventually embedded in the new concrete walls.

The preservation and protection of the historic plaster finishes on the interior faces of the new exterior shear walls required careful consideration during the construction of the shotcrete walls. Avoiding damage to the historic plaster was of paramount importance for the project. The contractor, in coordination with the design team, elected to use small light gauge steel shaft studs to hold the backing for the shotcrete wall along with the waterproofing membrane (Fig. 5). The shaft studs were buried inside the shotcrete wall without interrupting the continuity of the reinforcing.

All new and existing shear walls extended down to be supported on new concrete foundations. The new foundations were designed to carry the full seismic load, while the original building foundations under the existing concrete columns remained unchanged to support gravity loads. The new foundations were epoxy doweled into existing foundations for continuity and load transfers, utilizing the gravity loads from the existing columns to resist the overturning seismic loads on the new foundation.

The existing building foundations are at different elevations throughout the site, and at some locations, they are +/- 15 feet below grade. To avoid surcharging or undermining the existing foundations, the bottoms of the

City of LA Ordinance Requirements:

- *Buildings must meet 75% of current LA Building Code (LABC) seismic demands for strength with gravity members having deformation compatibility for 100% LABC drifts.*
- *OR Buildings must meet ASCE 41-17 Building Performance Objective for Existing Buildings (BPOE) using a tier 3 procedure. For a Risk Category III structure:*

Performance Level	Hazard Level
Damage Control	BSE-1E: Not less than 75% BSE-1N
Limited Safety	BSE-2E: Not less than 75% BSE-2N

- *Qualified Historic Buildings shall comply with the 2019 California Historical Building Code (CHBC), which permits maximum seismic forces to be limited to 40% of the building or component mass for Risk Category III buildings.*

Fig. 5. A shotcrete test panel with shear wall infill confirmed that proper protection of the plaster and concrete consolidation around the stud flanges was consistently achievable.

Fig. 6. Foundation construction required further coordination with the existing field conditions on a case-by-case basis; some steps that were previously assumed to be required were eliminated, and step heights were adjusted to reduce the number of steps as much as possible.

Fig. 7. After the portico columns were strengthened, they were analyzed to verify the plastic rotational capacity was adequate for the end rotations due to the inelastic building drift.

new and existing foundations were aligned in the structural drawings. This proved challenging during construction due to the required depth of excavations, adjacent property lines, adjacent building surcharges, and steps needed in the new foundations to accommodate elevation changes. Due to the project schedule, rebar shop drawings were produced based on the original drawings and fabrication began prior to excavation. Adjustments were made in the field as existing foundations were exposed and found to deviate from the original drawings, causing rebar congestion and step layout issues (Fig. 6).

The independent gravity columns of the Theatre that were not integrated into new concrete shear walls were analyzed as deformation controlled secondary members according to ASCE 41 for seismic drift compatibility. The analysis revealed that the independent columns had inadequate shear strength to develop the full expected plastic flexural capacity of the columns and were classified as shear-critical. To strengthen the columns to achieve a flexure-critical condition, the required FRP shear strengthening to develop the full expected plastic flexural capacity was specified on the structural drawings as equivalent grade 60 shear reinforcing along with the existing material strengths and the classification of the member action for deferred submittal design (Fig. 7).

The existing elaborate historic plaster ceiling system was another critical element to brace without compromising the historic fabric of the Theatre. The 1-inch-thick ceiling was supported only by vertical wires and created a potential falling hazard during a seismic event. Utilizing the option of meeting the California Historical Building Code (CHBC) as permitted by the ordinance, a new lateral bracing system was designed with light gauge steel framing.

Project Renovations Scope

Major renovations to the Theatre were included in the project with the goal of enhancing the audio and visual capabilities and user experience to achieve a state-of-the-art movie experience similar to other modern theaters. Additionally, in collaboration with the historical preservation architect Historic Resources Group, the design team was driven to further reveal and highlight the original architecture of this grand movie palace.

Although the original Theatre design did not include a lobby area, the new architectural layout required one that would also host a control booth. A new steel framed platform for these functions was designed to visually and acoustically separate the non-enclosed entry from the auditorium. Below the lobby platform a new concrete partial basement provides the necessary area for an emergency generator and mechanical equipment. A new light gauge steel framed projection booth on the low roof is accessed by elevator from the lobby and is capable of projecting modern movie formats and nitrate film formats (one of only five nitrate film projection theaters in the United States). Various stage improvements utilizing conventional steel and light gauge steel framing provide the infrastructure for presentations and for the large projection screen. Steel tension cable supported speakers in the auditorium provide high quality Dolby Atmos audio and a new auditorium seating platform constructed of tiered raised access floor systems leads from the lobby platform down to the stage. Finally, a new steel stair and catwalk provides access to the fly loft and original organ loft from inside the three-story structure at the southwest corner of the Theatre.

A defining feature of both the 1997/1998 renovations and the new renovations is the design for "reversibility." Reversibility is characterized as features designed for the possibility of future removal without negatively impacting the original features and structural stability of a building. Its applicability extends to architecture, mechanical, electrical, plumbing, structural, and all other renovation features of a building. This design feature is especially important for historic structures to avoid unnecessarily impacting historic fabric and to allow the structure to undergo future renovations without significantly altering the historic structure. The design team collaborated so the new renovations are designed to be either standalone without tying into the core and shell structure, or removable without negatively impacting the structural strength of the core and shell structure like the cantilever partition walls around the new lobby, or the tension cables supporting the speakers.

Fig. 8. Concrete frame repair at southwest portion consisted of shoring where required to remove and repair poorly placed concrete, mortar or epoxy injecting cracks, cleaning, and repairing corroded reinforcing, and placing repair mortar at spalls and to achieve proper clear cover on reinforcing bars.

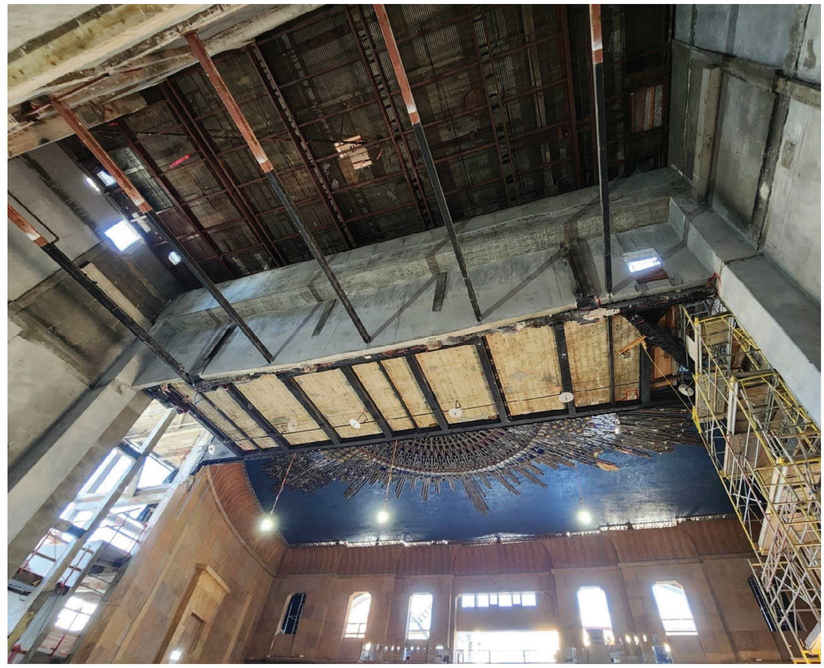

Fig. 9. To adequately transfer the new girder loads into the columns and to enlarge the column concrete area, an interior reinforced concrete column enlargement was designed to be epoxy doweled to the full height of the original column and to achieve direct bearing under the ends of the new proscenium girder.

Hidden Conditions and Damage

During construction, despite the initial findings of the visual condition assessment during design, after the removal of all the exterior plaster finishes widespread poor-quality concrete and damage was found. Poor quality concrete consisted of inadequate consolidation, rock pockets, honeycombing, and inadequate reinforcement clear cover. Damaged concrete consisted of cracking, reinforcement corrosion, spalling, and delamination. The cracked concrete is likely partially due to damage during the Northridge earthquake and other regional earthquakes. A repair program was enacted to rectify these structural issues encountered since the full design strength of many of the original elements was necessary for the retrofit (Fig. 8).

During the early phases of the design, it was noted that the original proscenium columns on each side of the stage did not exist, and no documentation of this modification could be found. The original proscenium was designed with a narrow view of the screen, and it is likely that as movie projecting technology progressed, the desire for a wider screen resulted in the widening of the proscenium. As construction commenced and finishes were removed, it was alarmingly discovered that the columns had been demolished without adequately modifying the structure of the roof to increase the proscenium girder span from the original approximately 42 feet to the modified approximately 70 feet. Considering the tributary roof areas loading the proscenium girder line consisted of the high roof over the stage, the organ loft roof, and the organ loft floor slab, large shoring towers to support the deficient proscenium girder were installed. A new large concrete girder was designed to support the tributary roof and organ loft loads and span approximately 70 feet between the perimeter columns. The original perimeter columns that now supported the new girder were analyzed for the increased load and found to be deficient for confinement and concrete area. Since the columns could only be enlarged towards the interior to avoid changing the exterior historic features of the Theatre, the requirements for added confinement were designed as additional FRP strengthening and the equivalent grade 60 confinement reinforcing was specified on the structural drawings for deferred submittal (Fig. 9).

During construction, it was also discovered that an undocumented sloping concrete topping slab had been previously added on the original low roof of the Theatre over the new lobby area. In some areas the topping slab was up to 12 inches thick and significantly increased the weight of the low roof compared to the original 3-inch slab. Since the weight of the topping slab was not considered in the retrofit design already under construction, as well as the requirements for installing FRP directly on the structural roof diaphragm slab, the topping slab was selected to be removed. Removal was time consuming and required care to be taken to avoid damaging the structural roof diaphragm slab because the topping slab was found to consist of hard rock concrete that was well bonded to the 3-inch slab.

Conclusion

The comprehensive retrofit and rehabilitation of the historic Egyptian Theatre in Hollywood stands as a remarkable example of how engineering innovation can merge with historical preservation and modern renovations to achieve seismic resilience and the successful re-birth of a landmark historic building. By adhering to the rigorous requirements of the City of Los Angeles' mandatory retrofit ordinance, the project successfully navigated the complexities of retrofitting a non-ductile concrete (NDC) structure without compromising its historical integrity. Through the strategic use of fiber-reinforced polymer (FRP) and traditional strengthening techniques, the Theatre was fortified to meet modern seismic demands while preserving its iconic architectural elements.

Moreover, the careful bracing of historic elements like the historical plaster ceiling and the reinforcement of the proscenium girder illustrated the commitment to maintaining the Theatre's historical character while upgrading its structural performance. ▪

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Melineh Zomorrodian, SE, joined Structural Focus in 2008 as an intern, and remained with the firm following her Masters graduation from UCLA. She specializes in structural analysis, design, and construction administration.