INTELLIGENT
ENGINEERING
SERVICES

**September 20, 2021**

Mr. Tyler Boehme
Senior Project Manager
The University of Texas at San Antonio
One UTSA Circle
San Antonio, Texas 78249
210.458.8072 C
tyler.boehme@utsa.com

**Re: Assessment of existing floor structure live load capacity**
**Institute of Texan Cultures**
**The University of Texas at San Antonio**
**IES Project No: 1213012**

Dear Mr. Boehme:

This report provides a description of conditions observed in the existing building, and assessment of the existing floor structure live load capacity with recommendations based upon our observations and our analysis. We observed UTSA ITC on Monday, July 26, 2021, and again on Tuesday, July 27, 2021. Mr. Vinny Genco, Project Coordinator - Facilities provided access to the building, and accompanied me during the observation on the morning of July 26th.

UTSA ITC is a three-story concrete structure (see Photos 1-4), with an exterior corridor on the Second-Floor level wrapping around the entire perimeter of the building. The building has an overall building footprint area of approximately 87,840 square feet. Building areas indicated in this report were determined from construction plans dated October 28, 1966 for the main building, and September 8, 1967 for the dome structure within the main building. The accuracy of the building areas was not confirmed by field measurements, and should only be considered approximate. The front of the building faces West toward the Tower of the Americas.

The Institute of Texan Cultures opened as the Texas State Exhibits Pavilion, which was built for the 1968 HemisFair celebration. After the World's Fair, the museum was turned over to the University of Texas System and UTSA assumed administrative control of the museum in 1973, designating the Institute as a campus in 1986. The 1966 construction documents of the main building were produced by Caudill Rowlett Scott Architects and Callins and Wagner Associate Architects with Mullen and Powell as the structural Engineer of Record, all out of Texas. The 1967 dome structure within the main building construction documents were produced by Gordon Ashby / Usher-Follis Designers out of California.

Photographs taken during our observations (36 total) are included at the end of this report, and are referenced in the following narrative. Further, recommendations for the observed conditions are offered at the end of this report.

## GENERAL STRUCTURAL DESCRIPTION:

### Existing Main Building:

The 1966 main building is a concrete frame building that appears to conform to the Architectural and Structural plans we reviewed. The First-Floor level consists of larger open mechanical spaces and storage rooms with a few offices to support the maintenance staff and also holds the public lounge and restrooms. At the Second-Floor level, the main museum exhibits are showcased on this level for the general public without any restrooms on this level. The Third-Floor level is mainly used for offices and archival storage for UTSA personnel and the support staff of the museum itself. This level also contains access to the inside of the dome structure for general upkeep and maintenance of lighting and audio controls. Lastly there are two Roof levels consisting of a concrete Low Roof level where all the mechanical roof top units are located and a steel High Roof level which supports the dome structure below.

The structural drawings indicate that the building is founded on a deep foundation system consisting of drilled and under-reamed (belled) piers extended to a depth of 40 feet below the First-Floor level. The First-Floor level is more than halfway buried into the ground on the North and South ends of the building due to earth formed berms near the building (see S105 & S106). The First-Floor level also utilizes basement concrete retaining walls on the North and South ends of the building where the East and West sides of the building are at the ground level. The larger concrete columns that exceed a single pier capacity, and where the main building columns are located, bear directly on pier caps over belled piers and extend up to the concrete Low Roof level, and are typically spaced 42 feet on center. At the concrete Low Roof level along the perimeter of the dome opening within the building, concrete beams support steel columns extending up to the steel High Roof level.

The First-Floor slab is indicated to be an unstiffened, 5” thick concrete slab-on-grade with welded wire mesh reinforcing. For the mechanical areas on the First-Floor, the slab is indicated to be an unstiffened, 6” thick concrete slab-on-grade with 3/8” diameter reinforcing bars spaced at 12” on center each way (see S101). Concrete masonry walls were utilized on this floor since most of this floor was used for mechanical spaces and large storage rooms.

The type of fill (select or in-situ soil) to be used under the floor slab was not indicated on the structural drawings. Required compaction density for the fill materials used under the slab was also not indicated on the drawings. Concrete slabs for both typical conditions and the mechanical areas were poured on a 6” thick sand fill layer. This detailing with sand creates a permeable stratum under the slab that could potentially serve to provide access for potential perched water to drain under the building slab only to be trapped below the slab until the water is evaporated or otherwise naturally dissipated. All of the First-Floor level perimeter beams are in contact with the ground which typically provides a barrier from surface rainwater.

An elevated concrete floor system at the exhibit Second-Floor level consists primarily of 8” wide x 16” deep metal pan-formed concrete joists spaced at 38” on center with a 4½” thick slab creating a one-way pan joist system (see Photo 5). For the exterior corridor along the entire perimeter of the building, the elevated concrete floor system consists of 10” wide x 16” deep concrete pan joists spaced at 40” on center with a 4½” thick slab creating a one-way pan joist system. This area is depressed 7½” to allow a marble paving material to be installed as the exterior corridor flooring. The concrete columns from below, support reinforced concrete beams that carry the exterior granite facing panel walls at the exhibit Second-Floor level up to the Third-Floor level (see S102). The typical interior concrete girders are 96” wide x 24” deep that span from the main concrete building columns spaced at 42 feet on center (extending from the First-Floor level up to the concrete Low Roof level) to minor concrete columns spaced in between the main concrete building columns (only extending from the First-Floor level to the underside of the Second-Floor level).

The elevated floor system at the Third-Floor level consists primarily of deeper 6½” wide x 24” deep concrete pan joists spaced at 36½” on center in each direction with a 4½” thick slab creating a two-way waffle slab system (see Photo 6). The structure over the exterior corridor at the perimeter of the building consists of shallower 6½” wide x 20” deep concrete pan joists spaced at 36½” on center in each direction with a 5½” thick slab also creating a two-way waffle slab system that serves as the exposed ceiling of the exterior corridor below (see Photo 7). This area is depressed 3” to allow a 3” concrete topping to be at the same elevation of the finished floor at the perimeter of the building (see S103). There are two typical interior concrete girders 16” wide x 72” deep that span from the main concrete building columns. It appears that the main mechanical trunk lines were positioned between the two interior girders to minimize exposure within the exhibit area below on the Second-Floor level. These two interior girders are connected with three 10” x 10” concrete struts located at quarter points along the length of the girders. The struts are positioned toward the bottom of the two interior girders which have 18½” x 18½” openings through them between the struts for secondary air conditioning ducts that serve the exhibit area below. Exterior angled precast concrete panel walls from just below the Third -Floor level up to the concrete Low Roof level are supported by two cantilevered concrete beams extending off the two main building column line concrete girders (see S105 & S106).

At the concrete Low Roof level, the elevated system consists primarily of 8” wide x 20” deep concrete pan joists spaced at 38” on center in each direction with a 2½” thick slab creating a two-way waffle slab system (see Photo 8). The concrete joists were widened along the perimeter of the dome opening within the building, creating an approximate 8 ft wide band of wider joists consisting of 13” wide x 20” deep concrete pan joists to accommodate the supports for the steel High Roof level. The typical interior concrete girders are 24” wide x 60” deep and span between the main concrete building columns.

The steel High Roof level consists of steel, long span Type DLH joists 72” deep spaced at 6 feet on center and spanning 126 feet. These long span joists are supported by steel trusses fabricated from steel wide flange shapes with cantilevered steel columns from the concrete Low Roof level. A vertical triangular steel truss buttress is located approximately at the main building grid column lines at 42 feet on center to counteract side thrust (lateral force) coming from wind load over the surface of the exterior angled precast concrete panel walls up to the steel High Roof level. This buttress braces the side steel truss supporting the long span joists against buckling under these forces. The exterior angled precast concrete panel walls are supported from the concrete Low Roof level up to the steel High Roof level by the cantilevered concrete beams extending off the main building grid column line concrete girders (see S105 & S106).

**Existing Dome Structure:**

The irregular shaped dome structure within the main building is primarily suspended from the steel High Roof long span joists with steel rod hangers and miscellaneous steel shapes. Horizontal steel, wide flange beams are arranged in a radial pattern and located just below the Third-Floor level. These steel beams are supported by vertical steel tubes suspended from the long span joists on one end and supported by the concrete perimeter beams for the dome opening on the Third-Floor level on the opposite end, which also provides support for the bottom of the dome structure (see Photo 9). Altogether, these steel frames (consisting of the horizontal radial steel beams and vertical steel tubes) were positioned as tight as possible for the irregular shaped exposed coffered ceiling panels to be supported off these steel frames (see S105 & S106). There are a series of catwalks at three different elevations to access the top of the dome. These catwalks are accessed from the concrete Low Roof level to in-between the long span joists and hung at lower elevations above the top of the dome.

The typical irregular shaped ceiling coffer paneling was designated as Soft Goods material made with translucent plastic with poly-vinyl and co-polymer. These panels are supported by aluminum frames formed from extrusions of aluminum alloy with connecting fittings and hardware such as angle clips, straps, eyebolts and similar items which are supported by the steel frames and the long span joists as discussed above (see Photo 10). Plywood paneling and miscellaneous wood members were used to support items such as projectors, lights, speakers, and air conditioning grills.

**STRUCTURAL EXISTING CONDITIONS OBSERVATION:**

**Observed Collateral Loads for the Main Building:**

Our field observations primarily consisted of confirming collateral superimposed loads to the building structure from installed materials and finishes, and to confirm conformance of the structure with available existing plans (see S110-S112). These loads are typically added dead loads supported by the building system, or any interior weight of permanent materials such as the ceiling, mechanical duct systems, electrical systems, and plumbing systems. Typical ceiling collateral loads that were observed throughout the building on all levels are listed below in pounds per square foot. These loads come from the *American Society of Civil Engineers (ASCE) - Minimum Design Loads for Buildings and Other Structures.*

| | |
|---|---|
| Suspended Steel Channel System | 2 psf |
| Mechanical Duct Allowance | 4 psf |
| Plumbing | 2 psf |
| Lighting | 1 psf |
| Painted Gypsum Board | 3 psf |
| Acoustical Fiber Board (Lay-in) | 1 psf |
| Gypsum Plaster on Metal Lath | 10 psf |
| Five-ply felt and gravel | 6 psf |

Observed from the First-Floor level (see S107), and suspended from the exhibit Second-Floor level, lay-in ceiling was observed for typical office spaces, lounges, conference rooms and miscellaneous rooms for workspaces and light storage rooms (see Photo 11). Gypsum-board ceilings were observed for all restrooms, open areas near the entries into the First-Floor level, small storage rooms, and miscellaneous rooms that were too small to install a lay-in ceiling into (see Photo 12). Plaster was observed for the exterior soffits at the recessed entries on both the East and West side of the building (see Photo 13). A cloud ceiling was observed with a suspended channel system without the lay-in fiberboard at the main corridor (see Photo 14). For the First-Floor, there were exposed areas of the Second-Floor framing for large workspaces, mechanical rooms, corridors leading into the mechanical rooms, and large storage spaces (see Photo 15) and there were exposed areas of the Second-Floor framing with limited spray-on insulation in random mechanical rooms (see Photo 16).

Observed from the exhibit Second-Floor level, and supported by the Third-Floor framing (see S108), there was a cloud ceiling with a suspended channel system without the lay-in fiberboard throughout the entire floor within the interior of the building (see Photo 17). A gypsum-board ceiling was used with a suspended channel system below without the lay-in fiberboard within the opening for the dome creating a square band in the dome opening area (see Photo 18). A double gypsum-board ceiling was also present with a second layer of gyp-board supported by the upper layer of gyp-board within an irregular shaped transition area around the dome opening (see Photo 19). For the exterior corridor at the exhibit Second-Floor level, and supported by the Third-Floor framing, minimal lighting without any other collateral load was observed for the exposed exterior shallow waffle slab concrete framing (see Photo 20).

Observed from the Third-Floor level (see S109), and suspended from the concrete Low Roof framing, lay-in ceiling was observed for typical office spaces, lounges, conferences rooms and miscellaneous rooms such as workspaces and light storage rooms (see Photo 21). There were exposed areas of the concrete Low Roof framing observed for the large storage spaces and the mechanical rooms (see Photo 22).

In addition, and on the exhibit Second-Floor level, a 7½" depression (see S102) supported marble flooring that is sloped to drain the exterior perimeter corridor of the building. On the Third-Floor level, a 3" floor depression (see S103) supported a 3" concrete topping over the exterior corridor below (see Photo 23).

We did not go onto the top of the building to assess the roofing material. Based on an aerial photograph and with the typical grid column lines overlaid onto the aerial photograph, the mechanical units shown on the aerial photograph are visually close to the typical grid column lines as shown on the existing Structural drawings (see S104). The mechanical roof top units were shown with openings in the slab within the existing Structural drawings but the weights were not provided. The roofing material was assumed to be a built-up roof consisting of a five-ply felt with gravel.

**Observed Expansion Joints:**

The structural drawings indicate that the building contained expansion joints from the First-Floor level on up to the concrete Low Roof level. The expansion joint splits the building into two parts starting from the First-Floor level up to the exhibit Second-Floor level only creating one expansion joint location on these floors (see S101 & S102). From the exhibit Second-Floor level on up to the concrete Low Roof level (see S103), the expansion joint splits the building into three parts creating two locations of the expansion joint on each side of the dome opening within the building (see Photos 24-28).

## ASSESSMENT, STRUCTURAL CALCULATIONS AND RECOMMENDATIONS:

### General Building Assessment:

It appears that the foundation for the original part of this building is adequately performing as intended. Expansive clay soils at the site were evidently effectively addressed during the original construction of the building as we did not observe cracking or damage usually associated with soil movement issues.

The structural condition of the concrete superstructure and the concrete Low Roof framing appeared to be in good condition. The steel High Roof consisting of long span joists supported by steel trusses which supports the dome framing below appears to have performed well since the original construction.

## Miscellaneous Assessments:

At the request of the Facilities Project Coordinator, cracking was observed within the gyp-board wall on two sides within the large storage room on the Third-Floor level along the West side of the building (see Photos 29 & 30). One crack was located along the inside wall along the interior corridor and the second was located along the outside wall (see S103), both locations were centered over the expansion joint. This cracking was due to the gyp-board passing over the expansion joint without a metal expansion joint cover within the gyp-board wall.

A second request from the Facilities Project Coordinator was on the exhibit Second-Floor level in front of the Elevator 3 in the Southeast corner of the building where tapestry is currently being stored (see S102). The Facilities Project Coordinator was concerned with the weight of each tapestry roll bundled up in one area and the impact it has on the concrete structure below (see Photo 31). Cutsheets were not available for these rolls but I personally picked one up and I would guess each roll weighed approximately 100 pounds total. The structure below is currently designed for 100 pounds per square foot as discussed in the Structural Calculations section of this report for the exhibit Second-Floor level and is more than adequate to accept these bundled up rolls of tapestry.

Only a single existing structural repair was observed during the time of our visit located on the exhibit Second-Floor level exterior corridor at an exterior concrete column at an expansion joint located on the West side of the building at the expansion joint furthest South (see S103). Spalling was observed and repaired at the top of the exterior concrete column where a corbel type condition was utilized at this expansion joint to support a concrete beam (see Photo 32). An extension of the corbel was attached onto the existing concrete column to extend the corbel past the damaged area. It appears the same extension of the corbel was added to the opposite side of the concrete column to make the column look aesthetically symmetrical. This structural repair appears to be in good condition and no further action is needed at this location.

Three existing storage rooms, as described by their room numbers, were observed on the Third-Floor level (see S103) and the current storage conditions were noted as follows:

<u>3.07.02</u> (1555 SF), the First Storage Room located on the South side of the building contained cantilevered library metal storage racks in the middle of the room and metal catalogue rack stands with drawers along the walls (see Photo 33). All of the storage racks appeared they were not completely full and would have a weight limit of approximately 100 pounds per square foot if fully loaded. The metal catalogue rack stands with drawers were not opened to verify if they were full but would also have a weight limit of approximately 100 pounds per square foot if fully loaded.

<u>3.08.07</u> (3665 SF), the Second Storage Room located on the Southwest corner of the building has two rooms, a large open area and a small long narrow room over the cantilevered portion of the Third-Floor level (see Photos 34 & 35). Both rooms contain 4-post library metal storage racks and appeared to be completely full of books and would have a weight limit of approximately 100 pounds per square foot if fully loaded.

<u>3.04.19</u> (2014 SF), the Third Storage Room located on the Southeast corner of the building contained industrial compact mechanical assist mobile high density storage shelving (see Photo 36). The storage shelving appeared they were not completely full and would have a weight limit of approximately 185 pounds per square foot which will exceed the floor capacity if fully loaded as discussed in the Structural Calculations section of this report.

## Structural Calculations:

### Assumptions:

Specifications were not provided with the existing Structural drawings, therefore, certain assumptions had to be made for the concrete compressive strength and rebar yield strength. Based on our analysis of recreating the existing conditions as close as possible using observed collateral loads and room usage, we have calculated that a concrete compressive strength of 4000 psi for the structural concrete beams, slabs and columns was likely used to achieve the load capacities indicated in this report. Reinforcing steel yield strength was assumed to be 60,000 psi, which was commonly used at the time the building was constructed. For the piers, only the bearing capacity was used for our analysis and the concrete strength was not used, but more than likely, the concrete compressive strength is conservatively assumed as 3000 psi. None of these assumed material strengths was tested or verified in the preparation of this report.

### Current Live Loads:

Live loads are variable as they depend on usage which can change over time such as people or movable objects such as furniture. However, design codes can provide equivalent loads for various occupancies which are usually empirical and conservative based on experience and accepted practice. Typical live loads that were observed throughout the building on all levels are listed below in pounds per square foot. These loads come from the *American Society of Civil Engineers (ASCE) - Minimum Design Loads for Buildings and Other Structures.*

| | |
|---|---|
| Office | 50 psf |
| Restrooms | 50 psf |
| Corridor | 100 psf |
| Light Storage areas | 125 psf |
| Heavy Mechanical areas | 150 psf |
| Roof | 20 psf |

On the First-Floor level, the current use of the spaces has changed from what is indicated on the original existing Architectural drawings from open unassigned space, which more than likely used 100 psf live load throughout, to a defined usage for Offices, Corridors, and Light Storage areas (see S113). The current large Mechanical spaces were defined originally as 150 psf and the existing Restrooms as 50 psf.

The exhibit Second-Floor level was designed as an open exhibit area which utilizes 100 psf for the entire floor including the exterior corridor wrapping the entire perimeter of the building (see S114). No Restrooms were observed on the exhibit Second-Floor level.

Just like the First-Floor level, the Third-Floor level current use of spaces has changed from what is indicated on the original Architectural drawings from an open unassigned space, which more than likely used 100 psf live load throughout, to a defined usage for Offices, Corridors, and Light Storage areas (see S115). A few of the Offices, the Mechanical spaces and the Restrooms were defined originally within the existing Architectural drawings.

The concrete Low Roof level utilized 20 psf for the typical Roof live load with added loads for the mechanical units. Existing cutsheets for the units were not provided to us and the existing roof top unit loads were not listed in the existing Structural drawings. An assumed weight for each unit that was used for the analysis of the existing roof of 2000 pounds for the entire weight of the these units.

**Column Capacities:**
Column loads have been tracked down through beam design calculations, along with any moments applied to the concrete columns at each floor, from the concrete Low Roof level down to the piers just below the First-Floor level. Column steel percentages were also calculated which are a good indicator on how tight a column is designed with reinforcement or how much extra capacity is allowed. These designs were compared with the area of steel reinforcement and the gross area of each concrete column listed in the concrete column schedule within the original Structural drawings. Loads were also provided in the concrete column schedule at each level in the original Structural drawings and appear to be listed as unfactored actual total loads.

The minimum and maximum longitudinal reinforcement within a concrete column shall be at least 1% of the gross area and shall not exceed 8% of the gross area. A minimum of 1% is found throughout the entire structure for all levels and a maximum of 3% of the gross area. There are large (plus-shaped) concrete columns, that taper in the vertical plane creating a variable column size throughout, located at the main building column lines that extend from the pier caps just below the First-Floor level up to the under-side of the Third-Floor level and these column steel percentage are approximately 1% of the total gross area using the smallest profile on the tapered columns.

All other columns are either square shaped columns or rectangular shaped columns along the expansion joints. The typical columns sizes from the First-Floor level up to the underside of the exhibit Second-Floor level are 24” x 24” square columns and 18” x 24” rectangular columns along the single expansion joint both with a column steel percentage of approximately 1% of the total gross area. From the Third-Floor level up to the underside of the Low Roof level, the typical columns are 21” x 21” square columns and 21” x 42” rectangular columns along the two expansion joints which all range from 1% to 3% steel percentage.

Typical interior columns with the higher live loads were analyzed starting from the concrete Low Roof level with a square column size of a 21” x 21”. These smaller columns had a typical area of steel percentage in relationship to the gross area of approximately 3 percent. Through our analysis, the existing capacity used for this square column is around 63 percent consisting of the upper concrete column lift. The same typical interior columns which utilize a plus-shaped column starting from the Third-Floor level down to the First-Floor level had a typical area of steel percentage of 1% for the middle and lower concrete column lifts. The existing capacity used for the middle lift from the Third-Floor level down to the exhibit Second-Floor level only uses a capacity of about 32 percent, whereas, the existing capacity used for the lower lift from the exhibit Second-Floor level down to the First-Floor level only uses a capacity of about 38%. Our analysis shows that there is extra capacity for the concrete columns especially for the plus-shaped columns.

**Pier Capacities:**
Upon review of the existing Structural drawings, the bearing capacity was listed in a plan note on the concrete column schedule sheet as 23,000 psf allowable bearing pressure located in grey shale bearing stratum. The safety factor was not listed but was assumed as a factor of 2 which is common for pier bearing designs provided by Geotechnical engineers for the city of San Antonio. The building is founded on deep drilled reinforced concrete under-reamed (belled) piers drilled to a depth of 40 feet below the First-Floor level with either a 24” or a 30” diameter shaft and different bell diameters. Using the bearing pressure noted above, the calculated total allowable load, pier capacities are as follows:

| Bell Diameter | Capacity, k (1 kip = 1,000 lbs) |
|---|---|
| 24” | 72 k |
| 30” | 113 k |
| 36” | 162 k |
| 40” | 200 k |
| 42” | 221 k |
| 48” | 289 k |
| 54” | 365 k |
| 60” | 452 k |
| 64” | 514 k |

Pier caps are utilized under the larger column loads that exceed a single pier capacity and where the main building columns are located that extend up to the concrete Low Roof level and are typically spaced 42 feet on center. These pier caps have three different pier sizes utilizing four 48” diameter belled piers, four 54” diameter belled piers and four 60” diameter belled piers and each of these pier caps have a total capacity of 1156 k, 1460 k, and 1808 k respectively. The centers of the belled piers are positioned on a pier cap 6.5 feet apart which indicates a reduction in capacity would likely have been required. For example, the largest belled piers grouped together under an enlarged pier cap are 60” in diameter, which would typically require a 10 feet center to center spacing or 2D (2 x Bell Diameter) to utilize the full bearing value for each pier. The 6.5 feet spacing would most likely require a significant reduction in the capacity due to group effects in which we are assuming a .75 percent reduction in total capacity. With this reduction, the current loads on the larger piers within the pier caps are almost at full capacity, nonetheless, the loads from the columns are worst-case scenarios where each floor and roof are loaded at full live load capacity.

**Recommendations:**
As mentioned in the General Structural Description, the First-Floor unstiffened, 5” thick concrete slab-on-grade is typically designed for 100 psf but can be increased to 125 psf based on light storage rooms that were confirmed at our site visit. For the mechanical areas on the First-Floor level, the slab is indicated to be an unstiffened, 6” thick concrete slab-on-grade and is designed for a 150 psf but can be increased to 200 psf if required (see S116).

At the one-way pan joist framing supporting the exhibit Second-Floor level, the joists are the controlling structural element which are currently designed for 100 psf live load. From our analysis, the entire floor can be increased to a floor capacity of 150 psf live load if required for any future exhibit exceeding the current floor capacity (see S117). The girders are also currently designed for 100 psf live load and can be increased along with the floor slab and joists up to 150 psf live load.

The current Third-Floor level two-way waffle slab concrete joists are designed for 100 psf live load and no increase is available to this system. The two typical interior concrete girders as described in the General Structural Description are also designed for 100 psf live load. These two girders can be increased up to 150 psf live load but only close within the column grid lines which may not be useful to increase since this zone is very limited to the column grid lines directly over the concrete beams as shown in the appendix (see S118). The existing storage rooms on this level as described in the Miscellaneous Assessments section describes the First (3.07.02) and Second (3.08.07) storage rooms as being loaded with storage racks and shelving with a load of 100 psf. Since the slab and joists are the controlling members and are designed for 100 psf live load with a capacity used of 95%, the loading on this shelving within these two storage rooms can be taken to full capacity of the racks and shelving as long as they do not exceed 100 psf live load. For the Third (3.04.19) storage room, the industrial compact mechanical assist mobile high density storage shelving has a full design capacity of 185 psf live load that exceeds the 100 psf live load floor capacity that the structure is designed for with a capacity used of 95%. As noted in the Miscellaneous Assessments section for the Third (3.04.19) storage room, the mechanical assist mobile shelving is not fully loaded and is currently not overloading the existing concrete floor, but could potentially overload the floor if filled to capacity.

For the concrete Low Roof level two-way waffle slab concrete joists are designed for 20 psf roof live load with isolated locations for mechanical roof top units that was assumed to be 80 psf live load directly under the RTU footprints. Increasing the roof capacity was not analyzed and not investigated for this report.

**Summary:**

<u>First-Floor level</u>

- Existing 100 psf live load can be increased to 150 psf live load
- Existing 150 psf live load can be increased to 200 psf live load

<u>Exhibit Second-Floor level</u>

- The entire floor with an existing 100 psf live load can be increased to 150 psf live load.

<u>Third-Floor level</u>

- (3.07.02) First Storage Room is designed for 100 psf at 95% capacity and no increase is available over the concrete slab and joists.
- (3.08.07) Second Storage Room is designed for 100 psf at 95% capacity and no increase is available over the concrete slab and joists.
- (3.04.19) Third Storage Room is designed for 100 psf at 95% capacity and no increase is available over the concrete slab and joists.
- Existing 100 psf live load can be increased to 150 psf live load directly over all concrete beams only.

<u>Concrete Low-Roof level</u>

- Increasing the roof capacity was not analyzed and not investigated for this report.

**Limits:**

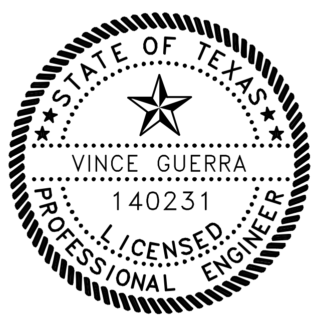
This assessment consisted of a visual observation only. Observations were limited to areas specifically addressed in the body of this report and should not be construed as involving an exhaustive review of all conditions present in the existing structure. Demolition or removal of materials was not conducted to gain access to hidden structural conditions, unless specifically noted otherwise in the report. No testing was performed to determine the strength and or quality of existing, in-place materials, and no floor elevation surveys were conducted as a part of this assessment.

We were not provided “as-built” record drawings, shop drawings, or related construction documentation reflecting actual in place construction, or engineering calculations to verify design assumptions and capacities for this structure. Therefore, we made the assumption that the facility was constructed using construction techniques typical as we understand them for the time period when the facility was constructed. Furthermore, our conclusions are based only upon our interpretations of our visual site observations made on the date(s) indicated.

Neither the observation, nor this report is intended to directly cover environmental, mechanical, electrical or architectural features, despite the fact that some of these conditions may be specifically noted because they potentially affect the structural performance of this building. Notify this office of any questions or comments regarding the information contained in this report. If none are received it is concluded than no exceptions are taken regarding the professional opinion(s) rendered.

Please feel free to contact our office, at your convenience, should you have any questions or comments regarding the matters addressed or if additional information is required. We appreciate the opportunity to be of service.

Sincerely,




Vince Guerra, PE
Assistant Project Manager
Intelligent Engineering Services, LLP
Texas Registered Engineering Firm F-432

Attachments:

**Attachment 1: Structural Photographs**
**Attachment 2: Structural Exhibits**