

NEW MEXICO STATE UNIVERSITY BRANSON LIBRARY FLOOR LOADING STUDY

FEBRUARY 2010

Prepared for:

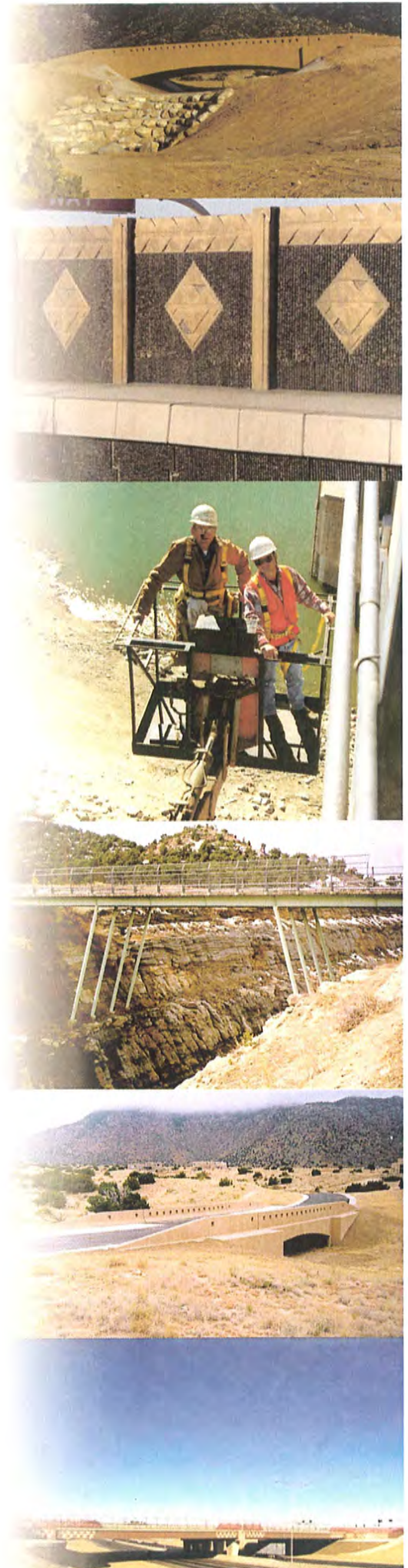
New Mexico State University
Las Cruces, New Mexico

Bohannon ▲ **Huston** INC.

ENGINEERING ▲

SPATIAL DATA ▲

ADVANCED TECHNOLOGIES ▲



NEW MEXICO STATE UNIVERSITY

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FINAL SUBMITTAL

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Prepared For:

New Mexico State University

Las Cruces, New Mexico

Prepared By:

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I. BACKGROUND AND PURPOSE

A. Background

Branson Library is located on the New Mexico State University campus in Las Cruces, New Mexico. The current building has been constructed over several years in four phases. The original phase was constructed in 1951. Additions were added in 1965, 1973, and 1980. The total building square footage is approximately 135,000 square feet. With the exception of the 1980 addition, each phase has four stories with roof and floor framing system of reinforced concrete columns, beams, joists and slabs. The original building is constructed of a one-way reinforced concrete slab system bearing on beams and girders which are supported by concrete columns. The 1965 addition's structural framing system is composed of a two-way conventional reinforced concrete waffle slab system. The 1973 addition is composed of a two-way conventional reinforced concrete flat slab system. The 1980 addition is a one-story building constructed with masonry walls and a steel joist roof framing system. Two utility tunnels running in the north-south direction are constructed below the first floor slab on grade. The utility tunnels are approximately six feet wide by eleven feet deep. The foundations for the original construction and all additions are drilled, reinforced concrete belled pier systems. Construction drawings for the first three phases were available for review during this analysis. However, the 1965 drawings reference specifications which were not available.

B. Purpose

New Mexico State University is interested in determining what the allowable live loads for this building are in order to determine whether the building is currently overloaded or if it can support additional stack loads. The building is currently heavily loaded in some areas and there has been some reported concerns regarding noticeable floor deflections observed throughout the building. The purpose of this study is to:

- Determine the service live load capacities of the existing framing systems for the original 1951, 1965 and 1973 phases.
- Determine whether the framing systems have the capacity to support additional stacks.
- Determine whether the observed floor deflections are greater than what would normally be expected for structures of similar age that utilize similar construction methods.

II. FIELD VERIFICATION

In order to verify the as-built construction, representative column, beam, joist and girder dimensions and spacing were measured in the field and compared to the construction drawings for all phases. Representative floor deflections were also measured throughout the building using a surveying level. Measured areas were chosen

where loading was high or where deflections were visible. These measured values were then compared to calculated deflection values with consideration of creep effects. See attached figures for the field measured deflections.

Locations for Windsor probe tests were chosen throughout the building to verify or determine concrete strengths for each phase. New Mexico State University contracted Southwestern Engineering, Inc. to perform the tests. The results are discussed in Section V.D.

III. BUILDING LOADS

A. Dead Loads (D)

Dead loads include the weight of all permanent materials and equipment supported by a structure, including the structure's self weight. The dead load of the structural members was calculated from the member sizes shown on the construction drawings for each phase. A collateral load of 10 pounds per square foot (psf) was assumed throughout the building to account for floor and ceiling treatments, light fixtures, electrical conduit and HVAC duct. Dead loads vary somewhat throughout the building due to change in member sizes and spacing and were accounted for in the analysis. The approximate, typical dead loads for an interior bay in each phase are shown below.

1. 1951 Phase Typical Floor

| | | |
|---------------------------------------|-----------|------------|
| 4" Thick Concrete Slab | 50 | psf |
| 12" x 14" Concrete Beams @ 7.5' o.c. | 17 | psf |
| 16" x 24" Concrete Girders @ 18' o.c. | 19 | psf |
| <u>Collateral Load</u> | <u>10</u> | <u>psf</u> |
| Total, approx | 96 | psf |

2. 1965 Phase Typical Floor

| | | |
|---|-----------|------------|
| 4" Thick Concrete Slab | 50 | psf |
| 5" x 11" Concrete Joists @ 2' o.c. each way | 36 | psf |
| 11" x 11" Concrete Joists @ 22.5' o.c. each way | 4 | psf |
| <u>Collateral Load</u> | <u>10</u> | <u>psf</u> |
| Total, approx | 100 | psf |

3. 1973 Phase Typical Floor

| | | |
|--|-----|-----|
| 8" Thick Concrete Slab | 100 | psf |
| Collateral Load | 10 | psf |
| <hr style="border: 0.5px solid black;"/> | | |
| Total, approx | 110 | psf |

B. Floor Live Loads (L)

The purpose of this study is to determine allowable live load capacities throughout the library based on its existing structural conditions. Since the existing library was constructed in phases over a period of approximately 30 years, the code requirements for floor live loads may have varied over that time. However, the current code required minimum floor live loads for Libraries from ASCE 7-05 are:

1. Reading Rooms 60 psf
2. Corridors above first floor 80 psf
3. Stack Rooms 150 psf*

* The Stack Room live load is based on non-mobile, double faced library book stacks where the stacks do not exceed 90 inches in height, the shelf depth does not exceed 12 inches per face, and parallel rows of double-faced stacks are separated by clear-space aisles no less than 36 inches wide.

C. Wind Loads (W)

Wind loads were evaluated for the purpose of determining story drift for calculating the column capacities. The site specific wind load per ASCE 7-05 is a horizontal load of 12.67 psf.

D. Earthquake Loads (E) and Snow Loads (S)

Earthquake loads are outside the scope of this report. Since this report is for interior floor loads, roof snow loads are not applicable to this analysis.

E. Load Combinations (LC)

The applicable load combinations from the 2005 American Society of Civil Engineers' *Minimum Design Loads for Buildings and Other Structures* (ASCE 7-05), Section 2.3. – "Combining Factored Loads Using Strength Design" are:

$$1.2D + 1.6L \quad (\text{Equation 2.3.2.2})$$

$$1.2D + 1.6W + L \text{ (Equation 2.3.2.4)}$$

IV. SYSTEM CRITERIA

A. Gravity and Lateral Force Resisting Systems

1. Upper Stories

The existing structural concrete framing components in the first three phases of the building were analyzed to determine the maximum service live loads (L) that could be supported on the second, third and fourth floors.

- a) 1951 Phase: The original building is constructed of a one-way reinforced concrete slab system bearing on beams and girders which are supported by concrete columns.
- b) 1965 Phase: The 1965 addition is a two-way reinforced concrete waffle slab system with drop panels at the concrete column locations.
- c) 1973 Phase: The 1973 addition is a two-way reinforced concrete flat slab system with drop panels and capitals at the concrete column locations.

2. Ground Floor

The ground floor for all phases is designed as a slab-on-grade, except in the locations of the utility tunnels. The capacity of a slab-on-grade is a function of the concrete and reinforcement strength and the bearing capacity and modulus of subgrade reaction of the underlying soil. Since no soil parameters were specified in the construction drawings and a geotechnical evaluation is outside the scope of this study, the exact service live load capacity of the ground floor was not determined. However, it is conservative to assume that the live load capacity is at least 150 psf.

The floor over the utility tunnels is designed as a one-way slab in the 1965 phase. The construction drawings for this phase show one utility tunnel running under a vertical utility chase with an underground basement extending to the west within column gridlines C-D/5-6 bay. A communications room is located on top of this basement on the ground floor. Since this area cannot be used for stacks, the floor slab capacity was not calculated in this analysis.

The 1973 phase construction drawings show two utility tunnels running north-south under the ground floor extending past the phase on the south into the 1965 phase. It is presumed that the westernmost of these tunnels was built after the 1965 phases, since it is missing from the construction drawings of that phase. The floor slab is detailed as a slab-on-grade and implies that the floor above the tunnel was intended to support the

dead and live loads of the ground floor over the approximate 6 foot width of the two utility tunnels. Since the construction drawings for the tunnel roof were not available for analysis, it is recommended that the floor live loads over the tunnel locations in the 1965 and 1973 phases not be loaded in excess of the 60 psf reading room load. See Figures 2.2 and 3.2 for the locations of the utility tunnels under the ground floor.

V. Material Strength

Strengths noted on the construction drawings are listed in Subsections A – C.

A. 1951 Phase

1. Superstructure Concrete: $f_c = 3,750$ psi
2. Slabs on grade: $f_c = 2,000$ psi
3. All Reinforcing: ASTM A15, $f_y = 40,000$ psi

B. 1965 Phase

1. Slabs and beams: f_c and f_y are not noted on the construction drawings
2. Columns: $f_c = 5,000$ psi for first floor columns, $f_c = 3,000$ psi for all other columns and drilled piers, f_y is not noted on the construction drawings.

C. 1973 Phase

1. Slabs and drop panels: $f_c = 3,000$ psi, f_y is not noted on the construction drawings
2. Columns: $f_c = 5,000$ psi for first and second floor columns, $f_c = 4,000$ psi for third floor columns, $f_c = 3,000$ psi for fourth floor columns all other columns and drilled piers, $f_y = 60,000$ psi or 75,000 psi.

D. Concrete Testing

The slab concrete strength was estimated using non-destructive Windsor probe tests according to ASTM C803, performed by Southwest Engineers Inc. Refer to Appendix C for test results. Windsor probe tests determine the compressive strength of a structure by driving three small probes into the concrete with a known amount of force. The average penetration depth of these probes is empirically related to the concrete strength. The Windsor probe equipment was calibrated using concrete of known strength from a local supplier. The results are shown in Table 1.

Table 1. Windsor Probe Test Results for Concrete Slabs

| Phase | Floor | Column Gridlines | Average Penetration Depth | Compressive Strength, f'_c (psi) | |
|-------|-----------------|------------------|--------------------------------------|--------------------------------------|--------------|
| 1951 | 2 nd | E-F/1-2 | 1.975 | 4,525 | |
| | | E-F/5-6 | 2.125 | 5,620 | |
| | | C-D/4-5 | 2.00 | 4,750 | |
| | | A-B/5-6 | 2.15 | 5,850 | |
| | 3 rd | E-F/1-2 | 1.95 | 4,750 | |
| | | E-F/5-6 | 1.95 | 4,300 | |
| | | C-D/4-5 | 2.725 | 5,850 | |
| | | A-B/5-6 | 2.00 | 4,750 | |
| | 4 th | E-F/5-6 | 2.125 | 6,050 | |
| | | C-D/4-5 | 2.05 | 5,175 | |
| | | B-C/5-6 | 2.12 | 5,720 | |
| | | A-B/1-2 | 2.125 | 5,850 | |
| | | | 1951 Phase Maximum Strength = | 6,050 | |
| | | | 1951 Phase Minimum Strength = | 4,300 | |
| | | | 1951 Phase Average Strength = | 5,300 | |
| | | | 1951 Phase Design Strength = | 3,750 | |
| 1965 | 2 nd | A-B/2-3 | 2.12 | 4,975 | |
| | | A-B/9-11 | 2.15 | 5,850 | |
| | | B-C/4-5 | 2.05 | 5,175 | |
| | | C-D/3-4 | 2.15 | 6,050 | |
| | 3 rd | A-B/2-3 | 2.15 | 6,050 | |
| | | A-B/9-11 | 2.185 | 6,050 | |
| | | B-C/4-5 | 2.125 | 5,850 | |
| | | C-D/3-4 | 2.2 | 5,625 | |
| | 4 th | A-B/2-3 | 2.15 | 6,140 | |
| | | A-B/9-11 | 2.055 | 5,260 | |
| | | B-C/4-5 | 2.05 | 5,175 | |
| | | C-D/3-4 | 2.151 | 5,950 | |
| | | | | 1965 Phase Maximum Strength = | 6,140 |
| | | | | 1965 Phase Minimum Strength = | 4,975 |
| | | | | 1965 Phase Average Strength = | 5,700 |
| | | | | 1965 Phase Design Strength = | 3,000 |

(Table 1 Continued)

| Phase | Floor | Column Gridlines | Average Penetration Depth (mm) | Compressive Strength, f'_c (psi) |
|--------------------------------------|-----------------|------------------|--------------------------------|------------------------------------|
| 1973 | 2 nd | A-B/3-4 | 2.00 | 4,750 |
| | | A-B/5-6 | 2.125 | 5,850 |
| | | B-C/2-3 | 2.025 | 4,975 |
| | | B-C/6-7 | 2.05 | 5,175 |
| 3 rd | | A-B/3-4 | 2.075 | 5,400 |
| | | A-B/5-6 | 2.10 | 5,625 |
| | | B-C/2-3 | 2.00 | 4,850 |
| | | B-C/8-9 | 1.90 | 4,100 |
| 4 th | | A-B/3-4 | 2.1 | 5,625 |
| | | A-B/5-6 | 2.1 | 5,725 |
| | | B-C/2-3 | 1.9 | 4,100 |
| | | B-C/6-7 | 2.15 | 6,050 |
| 1973 Phase Maximum Strength = | | | 6,050 | |
| 1973 Phase Minimum Strength = | | | 4,100 | |
| 1973 Phase Average Strength = | | | 5,200 | |
| 1973 Phase Design Strength = | | | 3,000 | |

E. Summary

From Table 1, the measured slab concrete strengths are all higher than the design slab concrete strengths. Therefore, the average measured concrete strength for the slabs and beams in each phase were used to find the live load capacity of those members in each bay in the building. The design concrete strengths were used to find the live load capacity of the columns, since the actual column strengths were not measured. The column strengths did not control for any bay of the building.

Since no steel strength was specified for the 1965 phase, f_y of 60,000 psi was assumed since it was consistent with the calculated capacities and deflections and with the observed deflections. The steel strength used in the slab and drop panels on the 1973 phase was not specifically called out on the construction drawings, but was assumed to be 60,000 psi to match the minimum steel strength called out for the columns.

VI. ANALYSIS SUMMARY

Since each phase used a different structural system, the most appropriate method of load path analysis was used to distribute loads to the structural members of each phase. In all phases, the possibility of uneven distribution of live load was taken into account to determine the worst case load distribution under live load. Then the capacity of each structural element (slab, beam, girder or column) was also calculated. The factored dead load was then subtracted from each member's capacity and the allowable service live load was then determined from

the remaining member capacity. The member with the lowest allowable live load capacity framing into each bay was used to determine the load capacity for that bay.

There are three types of deflection that occur in structural systems, short-term deflection, long-term deflection or creep, and settlement. There is also some margin of error in levelness of beams and slabs inherent in construction. It may not be possible to tell the difference from deflections due to loads, creep and settlement from deflections due to errors in construction.

The *2006 International Building Code* (IBC) limits allowable short-term deflections for each structural member to a percentage of the member length. Under dead plus live loads, the maximum allowable deflection is calculated using the ratio $l/240$, where l is the member's span, in inches. For the slabs, beams and girders at Branson Library, this is a relative deflection between the members' ends and their centers. This deflection can be estimated using the members' restraining conditions, loading pattern and section properties. Since concrete cracks under loads, only a percentage of the member's moment of inertia is useful in resisting deflection. The American Concrete Institute *Building Code Requirements for Structural Concrete* (ACI 318) conservatively suggests using a moment of inertia equal to 35 percent of the total uncracked section. This conservative assumption was used in the analysis. Short-term deflection can be larger than expected if supporting formwork is removed too early after a structural concrete member is cast.

Concrete members can also be subjected to long-term deflection or creep. Creep is when the deformation of a member continues to increase under a constant load over a long period of time. The severity of creep is dependent on the concrete strength, average relative humidity over the structure's life and the magnitude of load placed on the member. The additional deflection from creep can be anywhere from negligible to two or three times the short-term deflection.

Settlement of the structure's foundation can also affect the deflections observed in a building. Settlement can occur uniformly over the entire foundation of a building, which would not be observable within the structure itself, or it can occur differentially over the foundation. Differential settlement is observable within a building and is measured as the relative floor deflection between column locations. Total and differential settlements are usually estimated by a geotechnical engineer at the outset of the design of a building. While no geotechnical report for any of the phases of Branson Library were available for review, total settlements up to two inches and differential settlements of one inch are common estimates and will be used as benchmarks for this analysis.

A brief explanation of the load capacity and deflection analyses specific to each phase is described below.

A. Phase I – 1951

1. Load Capacities

The slabs in this phase were detailed and analyzed as one-way slabs bearing on the beams and/or girders. The beams and girders are detailed as continuous spans in the construction drawings. However, they were analyzed both as continuous and simple spans and the higher load capacity for each was used as the resulting load capacity. The beams had a higher load capacity when they were analyzed as continuous beams over their girder supports. The girders had a higher load capacity when they were analyzed as simple spans over their column supports. The bays between column gridlines C and D in all floors were constructed and analyzed as a one-way joist system.

The resulting controlling service live loads for the 1951 building phase vary from 63 psf to 161 psf and are shown for each bay in Figure 1.1. The bays between column gridlines C and D have a load capacity of 100 psf, indicating that these bays were intended as reading rooms or corridors. This is consistent with the architectural floor plans in the construction drawings which show these bays as offices or seminar rooms. From the site visit, some of these bays on various floors are currently loaded with stacks. It is recommended that these stacks be moved out of these areas and the space be used as corridors, offices or reading rooms.

With the exception of the landing at the top of what was originally a stairwell, currently an elevator, the remainder of Phase I appears to have been intended to be designed for stack rooms. The actual service live load capacities vary from 80 to 105 percent of the current code live load requirement of 150 psf for stack rooms. Therefore, the stack room definition given in Section III.B should be evaluated on a bay-by-bay basis to determine how the stacks shall be arranged to meet individual bay load capacities. Guidance on this is discussed in Section VII.

2. Deflections

Since the beams are supported by the girders in the 1951 phase, the total allowable deflection at the center of each bay is equal to the allowable deflections in the beams plus the allowable deflection in the girders. The length of a typical beam and girder are 16.7 feet and 21 feet, respectively. Therefore the typical total allowable short-term, relative deflection is 1.9 inches at the center of the bay. Under the calculated service live load capacities, the expected relative deflection at the center of the bay varies from 0.5 inches for the corridor/reading room areas, to 1.4 inches for the stack room areas.

Figures 1.2, 1.3 and 1.4 show the measured deflections throughout the 1951 phase. All measured deflections are normalized with respect to the highest measured point on each floor. The high point is defined as 0 deflection. From the measured deflections, the 1951 phase appears to have settled towards the north

and west. The largest differential settlement from the southeast corner of the original phase to the center of the north edge is approximately 2.5 inches and was measured on the second floor. This general trend is consistent for all floors, though it is less pronounced on higher floors since column loads decrease at each progressive floor towards the top.

Differential settlements between columns and centers of their respective bays are well within the 1.9 inches allowable even without consideration for creep effects and are generally less than the estimated predicted deflections.

B. Phase II – 1965

1. Load Capacities

The majority of this phase was designed as a two-way waffle slab with drop panels at the column location. The geometry, stiffness and loading of the phase allow the application of the direct design method outlined in ACI 318, Chapter 13. This method breaks the floor slab into strips in both directions and distributes loads by set percentage amount to various locations in each strip. The load capacity of each location in the strip is then compared to its applied load, from which the service live load can be determined. The area between column grid lines B-E and 6-7, excluding the vertical utility chase, was constructed as one-way slabs supported on beams. This section of the addition consists of the stairwell and elevator shaft, restrooms, hallway and reading/study balcony and was not intended for heavy loads.

The resulting controlling service live loads for the 1965 addition is approximately 140 psf throughout the areas of the building supported on waffle slabs, see Figure 2.1. This corresponds to a live load capacity equal to 93 percent of the current code live load requirement of 150 psf for stack rooms. Therefore, the stack room definition given in Section III.B should be evaluated to determine how the stacks shall be arranged to meet a bay load capacity of 140 psf. Guidance on this is discussed in Section VII.

2. Deflections

Since the 1965 addition is a two way slab system, the allowable short-term, relative deflection is 1.1 inches. Figures 2.2, 2.3 and 2.4 show the measured deflections throughout the 1965 addition. All measured deflections are normalized with respect to the highest measured point on each floor. The high point is defined as 0 deflection. Under the calculated service live load capacities, the expected relative deflection at the center of the bay varies from 0.9 inches to 1.4 inches for the stack room areas.

From the measured deflections, some differential column settlement is observed with the maximum differential column settlement of 1.44 inches measured on the northwest corner of the second floor. Differential settlements between columns and centers of their respective bays are within the 1.1 inches

allowable even without consideration for creep effects and are generally less than the estimated predicted deflections.

C. Phase III – 1973

1. Load Capacities

This addition was designed as a two-way flat slab with drop panels at the column locations. The geometry, stiffness and loading of the phase allow the application of the equivalent frame design method outlined in ACI 318, Chapter 13. This method breaks the floor slab into strips in both directions. Loads are distributed throughout the strips based on structural analysis of a continuous beam. The load capacity of each location in the strip is then compared to its applied load, from which the service live load can be determined.

The resulting controlling service live loads for the 1973 addition varies from 120 to 124 psf, see Figure 3.1. This corresponds to a live load capacity equal to approximately 80 percent of the current code live load requirement of 150 psf for stack rooms. Therefore, the stack room definition given in Section III.B should be evaluated to determine how the stacks shall be arranged to meet the bay load capacities of 120 to 124 psf. Guidance on this is discussed in Section VII.

2. Deflections

Since the 1963 addition is a two way slab system, the allowable short-term, relative deflection is 1.1 inches. Figures 3.2, 3.3 and 3.4 show the measured deflections throughout the 1951 phase. All measured deflections are normalized with respect to the highest measured point on each floor. The high point is defined as 0 deflection. Under the calculated service live load capacities, the expected relative short-term deflection at the center of the bay varies from 0.3 inches to 0.5 inches under the calculated service live load capacity. Using the creep coefficient calculation from *Design of Concrete Structures 13th Edition*, by Arthur Nilson, et al., the creep coefficient for a 36 year old building is 3.0. Meaning, deflections up to 3.0 times the short-term deflections can be expected. This translates to expected long-term deflections up to 0.9 to 1.5 inches under the live load capacity.

From the measured deflections, some differential column settlement is observed with the maximum differential column settlement of 2.8 measured in the middle of the south edge of the third floor. Differential settlements between columns and centers of their respective bays often exceed the allowable deflection limit of 1.1 inches. The relative deflection between the columns and bay centers vary from 0.4 to 2.6 inches on the second floor, from 0.12 to 0.96 inches on the third floor and 0.84 and 3.24 inches on the fourth floor. Based on observations made during the field visits, the third addition may currently be loaded in excess of the allowable live load capacity of 118 psf. Therefore, some of these large deflections may be due to overloading

of the slab. Another contributing factor could be creep and construction errors, since the floors are loaded approximately equally, but show large variations in deflections from floor to floor. It is possible, for example, that the forms were removed prematurely after casting the fourth and second floors.

VII. CONCLUSIONS AND RECCOMENDATIONS

It is recommended that NMSU evaluate each bay of each floor and confirm that each bay is loaded according to its intended use as corridors, reading rooms or stack rooms based on the load capacities shown in Figures 1.1, 2.1 and 3.1. The areas of most concern are bays which are loaded with stacks. Each bay loaded with stacks, that has a capacity of at least 150 psf, shall be evaluated to ensure that the stack dimensions do not exceed 90 inches in height, 24 in total width, and have a clear space aisle of at least 3 feet wide as described in Section III.B.

Stack-loaded bays that have a service live load capacity less than 150 psf shall be evaluated to determine how to arrange the stacks so that their load capacities are not exceeded. This may be achieved by increasing the minimum aisle width to a dimension greater than 3 feet, decreasing the maximum stack height to a dimension less than 90 inches, decreasing the shelf depth per face to a dimension less than 12 inches, using only one face per shelf or a combination of these options. This exercise is particularly important for bays located between column grid lines C-D bays in Phase I, and all the bays in Phase III.

The measured deflections throughout Phases I and II are within what is allowed by the current IBC code and what would be expected for a building of similar age and use. The measured deflections in Phase III exceed the allowable deflections from the current IBC code. The reason for this is likely due to a combination of overloading, creep and construction errors.

In summary, the majority of Branson Library is likely currently loaded at or near its capacity. It is not recommended that additional stacks be added, except on the ground floor in locations that are not directly above the underground utility tunnels.

VIII. REFERENCES

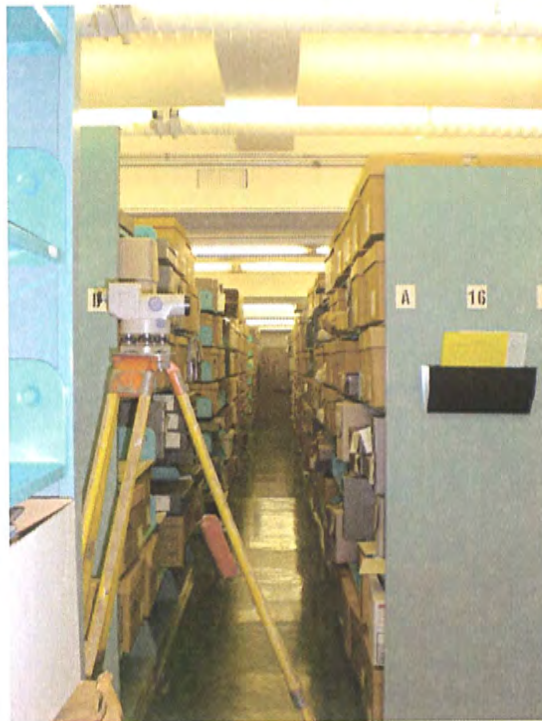
International Code Council, *International Building Code*, IBC2005

American Society of Civil Engineers, *Minimum Design Loads for Buildings and Other Structures*, ASCE Standard 7-05

Building Code Requirements for Structural Concrete, American Concrete Institute (ACI 318-05).

APPENDIX A – Representative Photos

Phase I – 1951







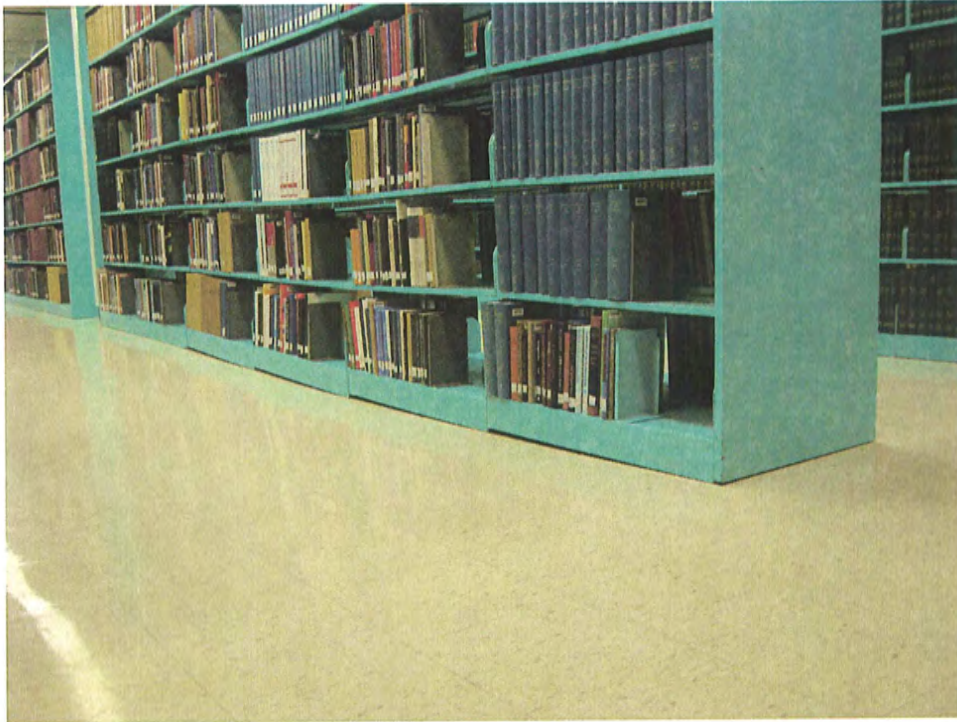
Phase II – 1965





Phase III – 1973







APPENDIX B – Figures

APPENDIX A – Representative Photos

Phase I – 1951

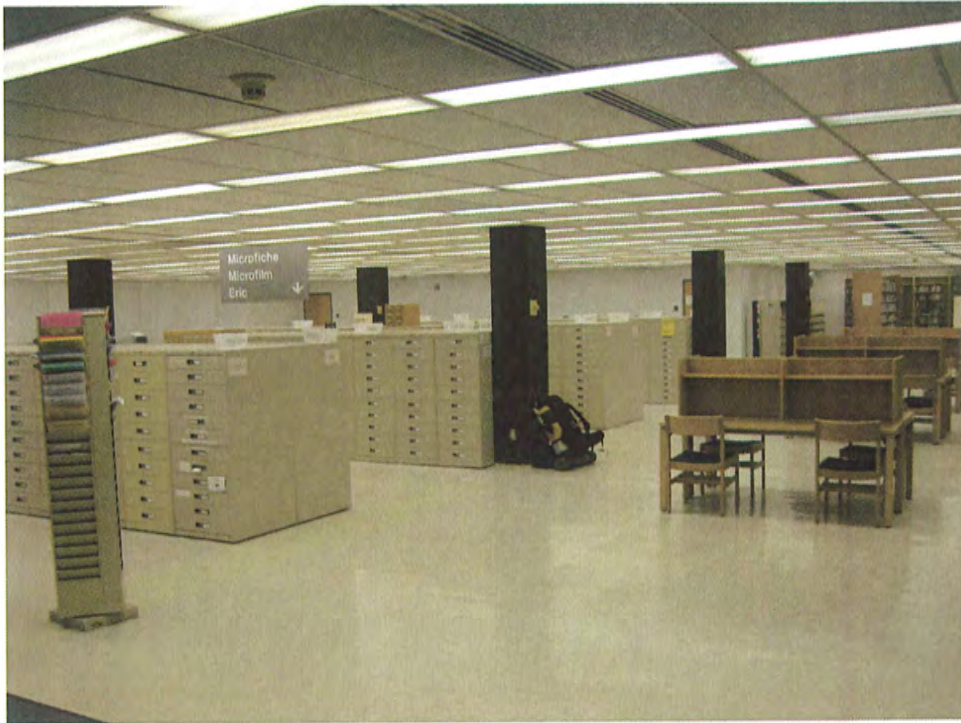




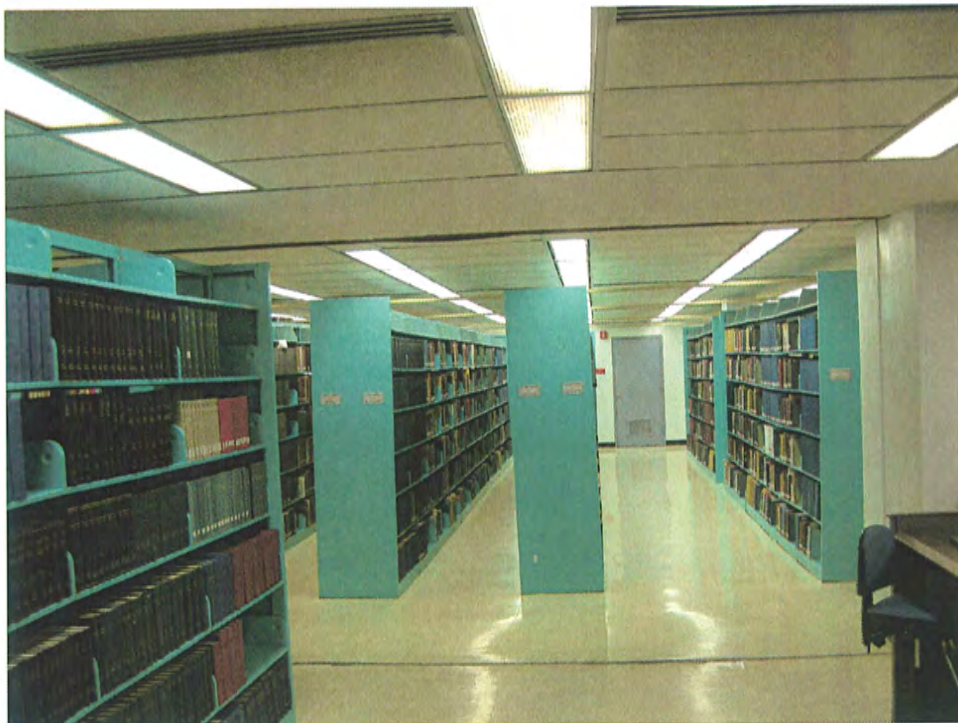


Phase II – 1965





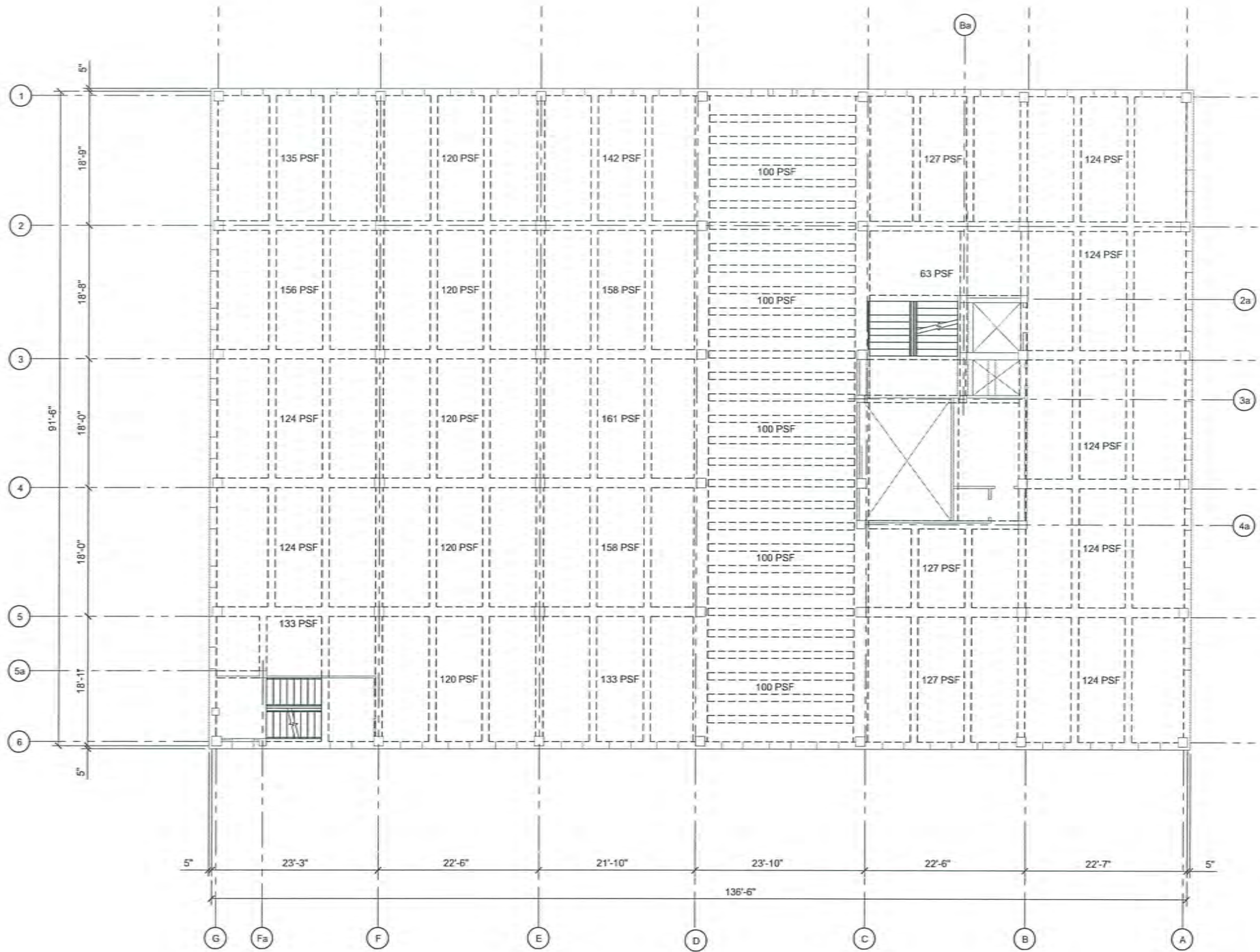
Phase III – 1973





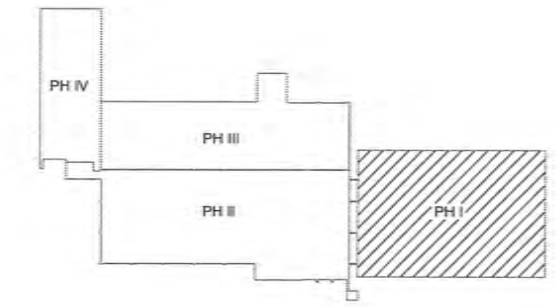


APPENDIX B – Figures



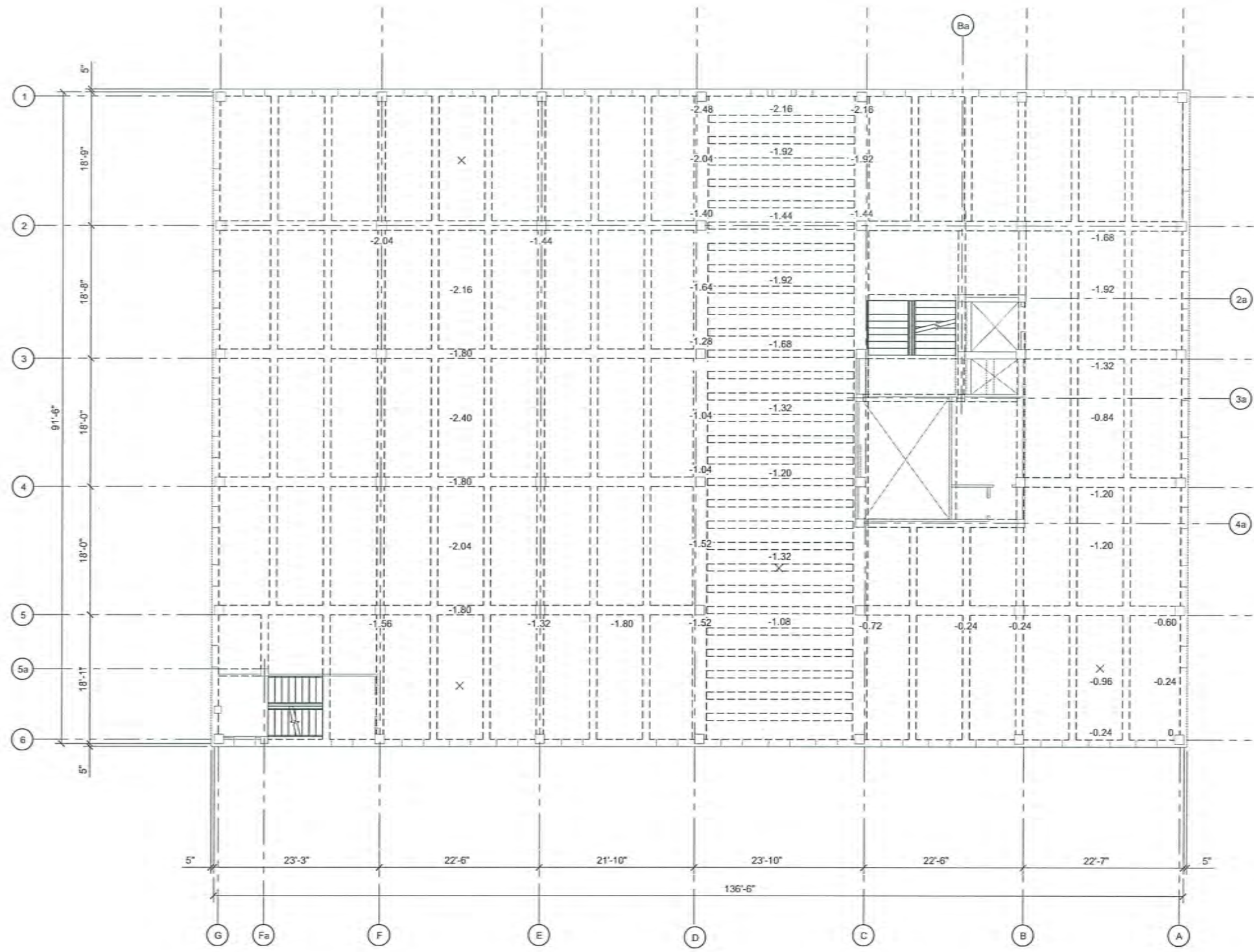
LEGEND:

xx PSF SERVICE LIVE LOAD CAPACITY



A BAY LOAD CAPACITIES - ALL FLOORS

P:\2010097\STRUCT\Drawings\ACAD\Drawings\PH I\PH I_BLDG.dwg Feb 10, 2010 - 10:44am

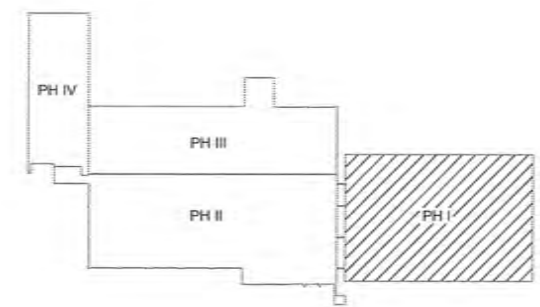


LEGEND:

- X.XX FIELD MEASURED DEFLECTION (INCHES) FROM HIGH POINT
- X HAMMER TEST LOCATION

NOTES:

1. IBC ALLOWABLE RELATIVE DEFLECTION (NOT INCLUDING SETTLEMENT) = 1.9 INCHES AT CENTER OF BAY
2. EXPECTED RELATIVE LONG-TERM DEFLECTION UNDER SERVICE LIVE LOAD CAPACITIES (NOT INCLUDING SETTLEMENT) = 0.5 TO 1.3 INCHES AT CENTER OF BAY



KEY PLAN



A SECOND FLOOR - PLAN - MEASURED DEFLECTIONS

P:\20100001\STRUCT\Drawings\ACAD\Phase I\Branson PH I_2nd Fl_M0.dwg Feb 05, 2010 - 5:22pm

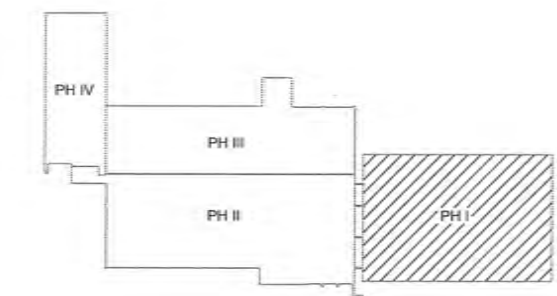


LEGEND:

- X.XX FIELD MEASURED DEFLECTION (INCHES) FROM HIGH POINT
- X HAMMER TEST LOCATION

NOTES:

1. IBC ALLOWABLE RELATIVE DEFLECTION (NOT INCLUDING SETTLEMENT) = 1.9 INCHES AT CENTER OF BAY
2. EXPECTED RELATIVE LONG-TERM DEFLECTION UNDER SERVICE LIVE LOAD CAPACITIES (NOT INCLUDING SETTLEMENT) = 0.5 TO 1.3 INCHES AT CENTER OF BAY

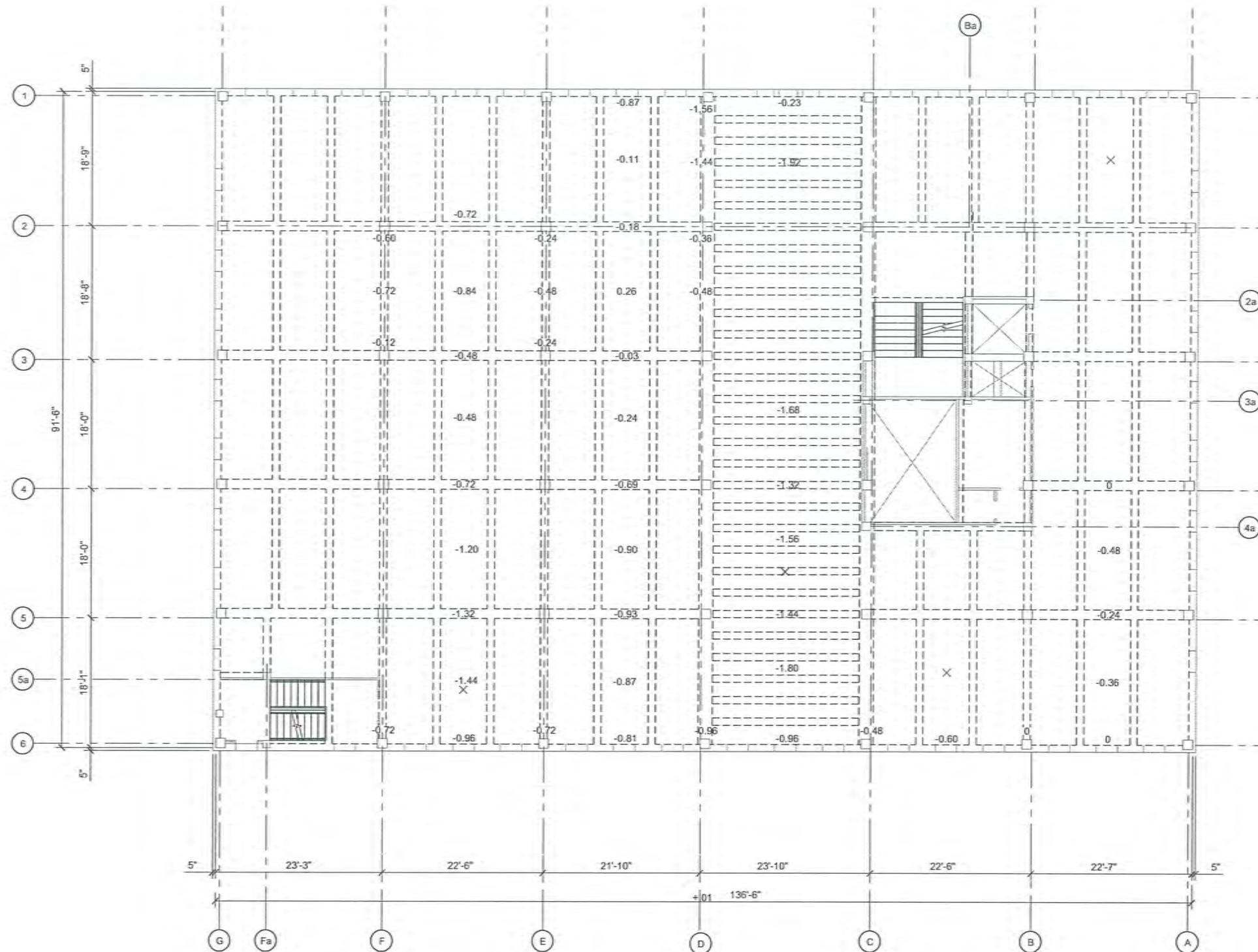


KEY PLAN

A THIRD FLOOR - PLAN - MEASURED DEFLECTIONS



P:\20100007\STRUCT\Drawings\ACAD\Phase I\Drawings\PH I - 3rd Flr_M0.dwg Feb 08, 2010 - 6:25pm

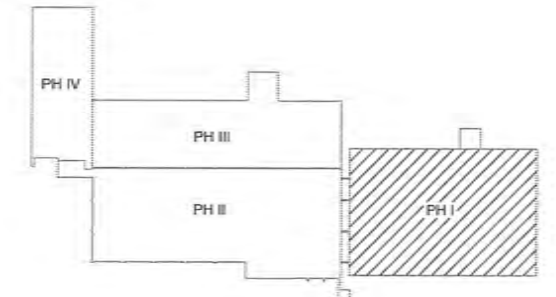


LEGEND:

- X.XX FIELD MEASURED DEFLECTION (INCHES) FROM HIGH POINT
- X HAMMER TEST LOCATION

NOTES:

1. IBC ALLOWABLE RELATIVE DEFLECTION (NOT INCLUDING SETTLEMENT) = 1.9 INCHES AT CENTER OF BAY
2. EXPECTED RELATIVE LONG-TERM DEFLECTION UNDER SERVICE LIVE LOAD CAPACITIES (NOT INCLUDING SETTLEMENT) = 0.5 TO 1.3 INCHES AT CENTER OF BAY

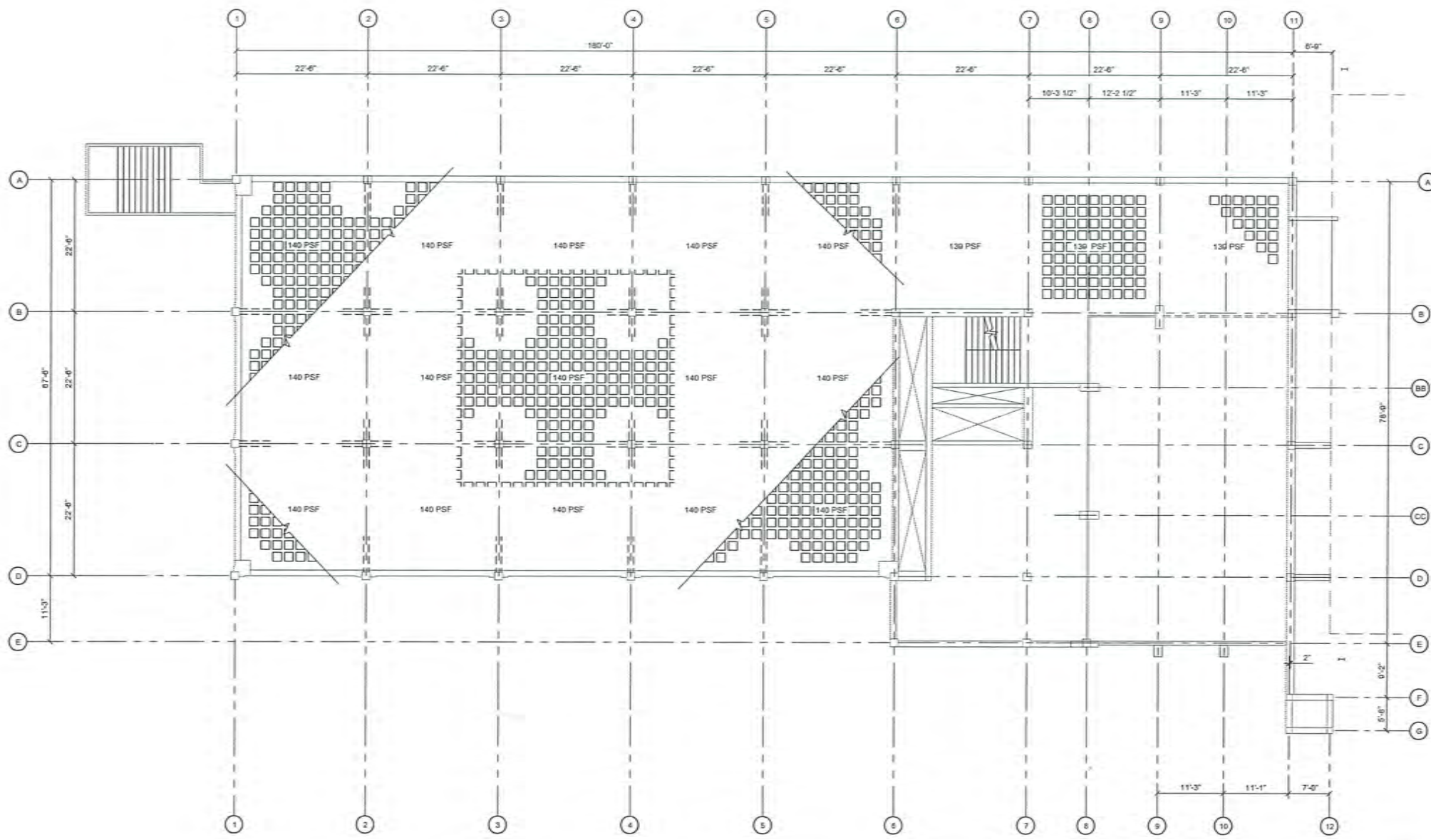


KEY PLAN



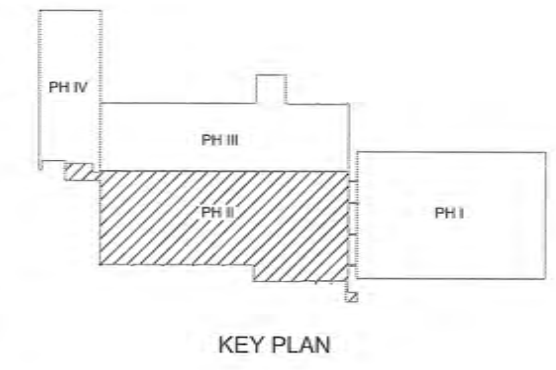
A FOURTH FLOOR - PLAN - MEASURED DEFLECTIONS

P:\20100001\STRUCT\Drawings\ACAD\Phase I\Branson PH I_4th FL_00.dwg Feb 05, 2010 - 2:29pm



LEGEND:

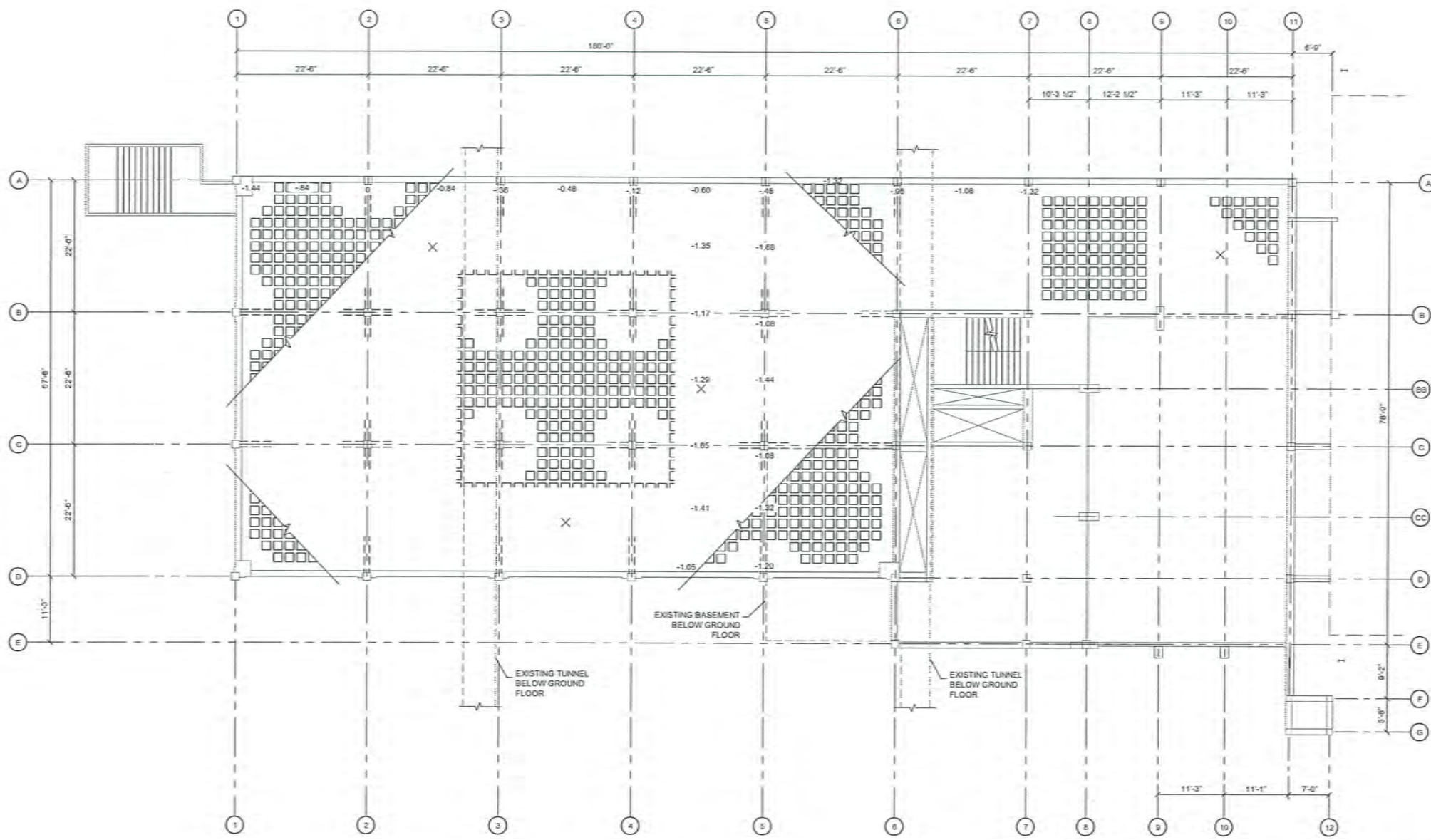
xx PSF SERVICE LIVE LOAD CAPACITY



A PLAN - BAY LOAD CAPACITIES - ALL FLOORS



P:\20100097\STRUCT\Drawings\A000\Phase II\Branson PH II_JP_RLC.dwg Feb 10, 2010 - 10:26am



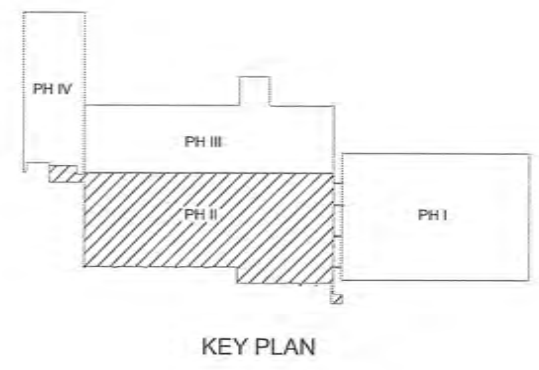
LEGEND:

-X.XX FIELD MEASURED DEFLECTION (INCHES) FROM HIGH POINT

X HAMMER TEST LOCATION

NOTES:

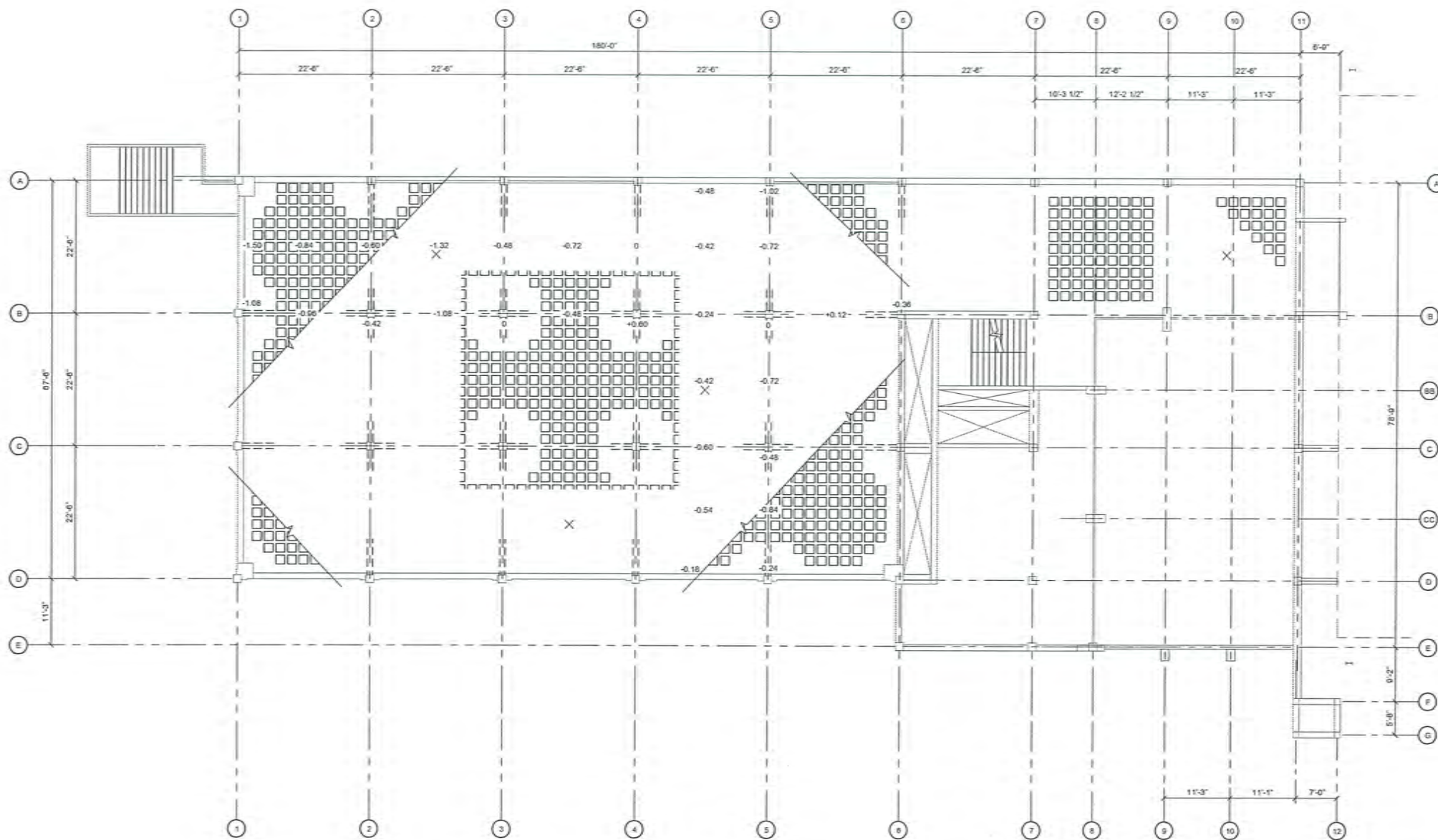
- IBC ALLOWABLE RELATIVE DEFLECTION (NOT INCLUDING SETTLEMENT) = 1.1 INCHES AT CENTER OF BAY
- EXPECTED RELATIVE DEFLECTION UNDER SERVICE LIVE LOAD CAPACITIES (NOT INCLUDING SETTLEMENT) = 0.9 TO 1.4 INCHES AT CENTER OF BAY



A SECOND FLOOR - PLAN - MEASURED DEFLECTIONS



P:\20100097\STRUCT\Drawings\KCO\Phase II\2nd FP_JD.dwg Feb. 05, 2010 - 5:37pm

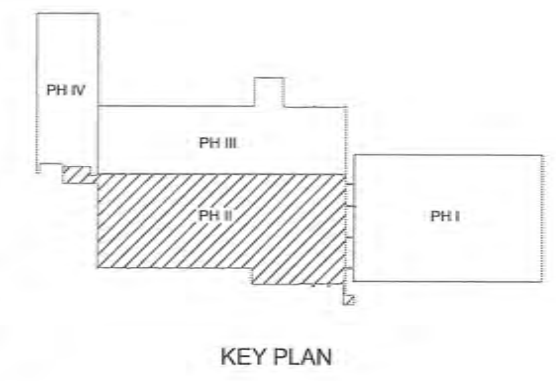


LEGEND:

- X.XX FIELD MEASURED DEFLECTION (INCHES) FROM HIGH POINT
- X HAMMER TEST LOCATION

NOTES:

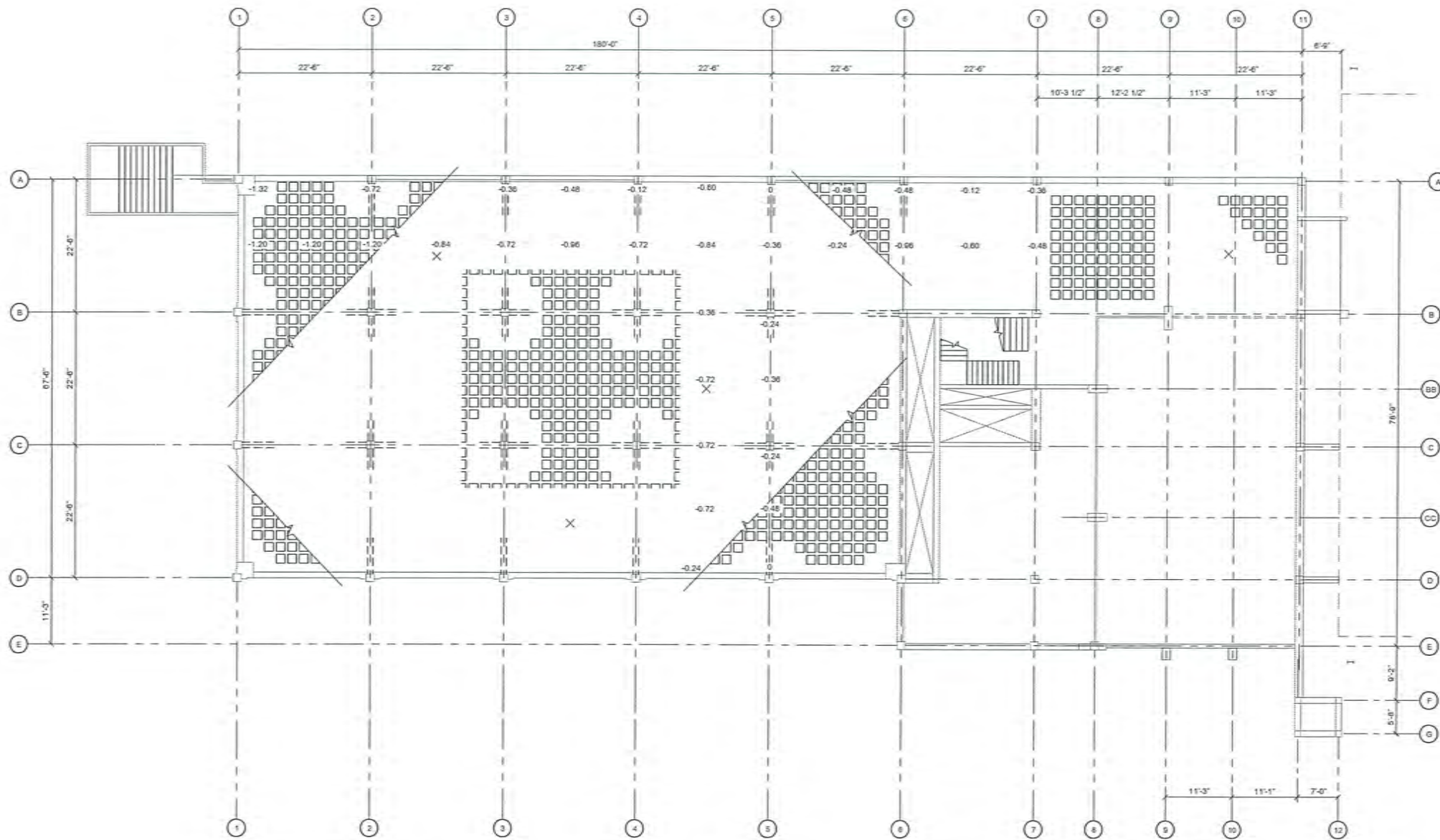
1. IBC ALLOWABLE RELATIVE DEFLECTION (NOT INCLUDING SETTLEMENT) = 1.1 INCHES AT CENTER OF BAY
2. EXPECTED RELATIVE DEFLECTION UNDER SERVICE LIVE LOAD CAPACITIES (NOT INCLUDING SETTLEMENT) = 0.9 TO 1.4 INCHES AT CENTER OF BAY



A THIRD FLOOR - PLAN - MEASURED DEFLECTIONS



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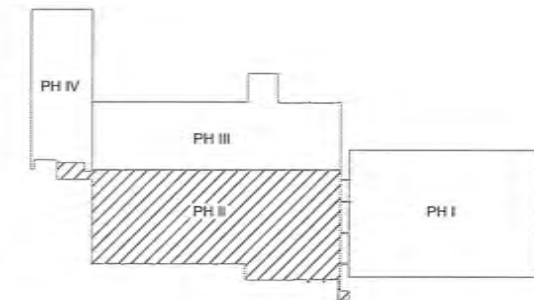


LEGEND:

- X.XX FIELD MEASURED DEFLECTION (INCHES) FROM HIGH POINT
- X HAMMER TEST LOCATION

NOTES:

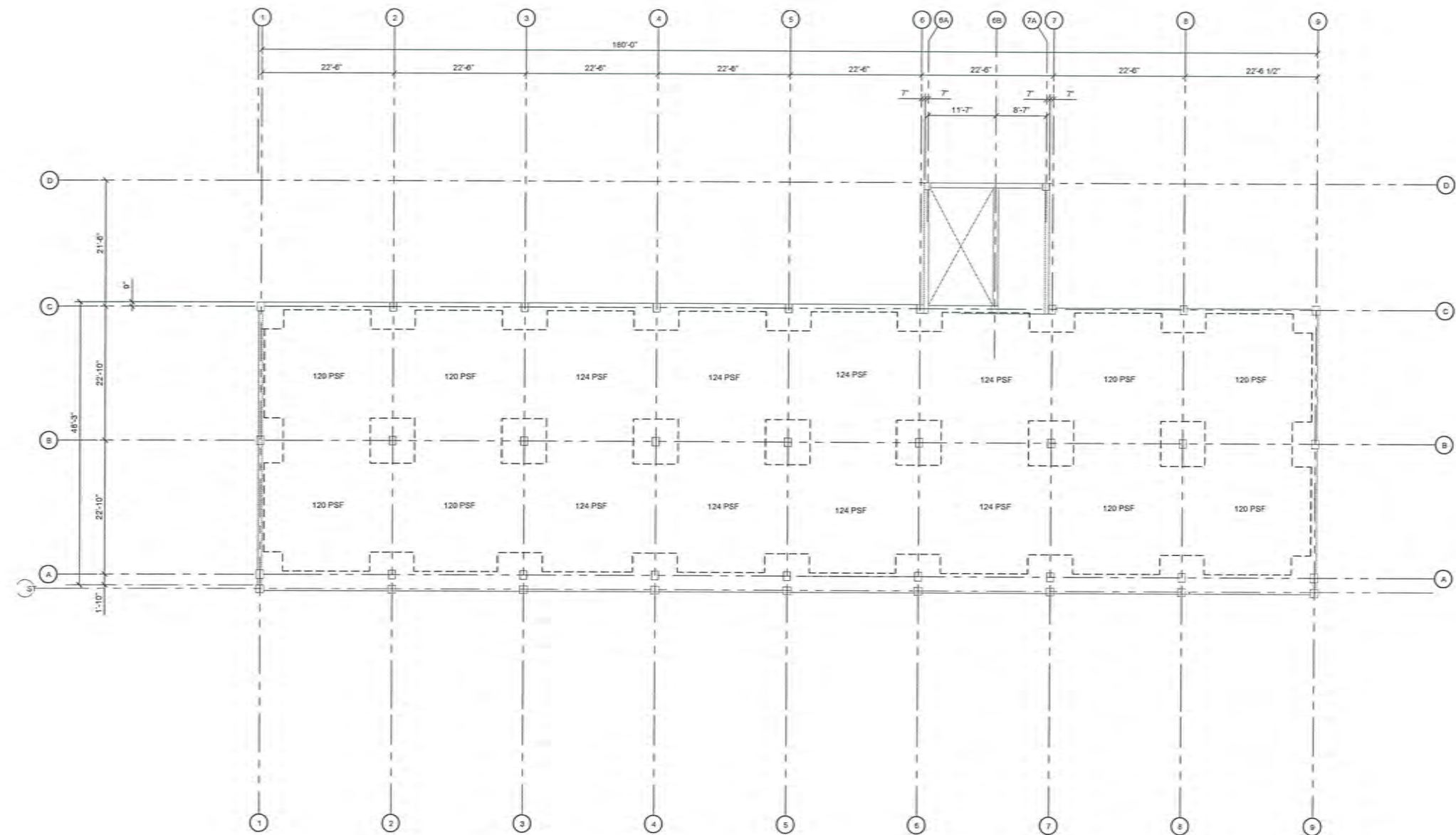
1. IBC ALLOWABLE RELATIVE DEFLECTION (NOT INCLUDING SETTLEMENT) = 1.1 INCHES AT CENTER OF BAY
2. EXPECTED RELATIVE DEFLECTION UNDER SERVICE LIVE LOAD CAPACITIES (NOT INCLUDING SETTLEMENT) = 0.9 TO 1.4 INCHES AT CENTER OF BAY



KEY PLAN

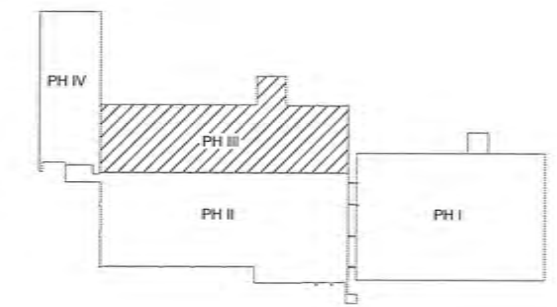
FOURTH FLOOR - PLAN - MEASURED DEFLECTIONS





LEGEND:

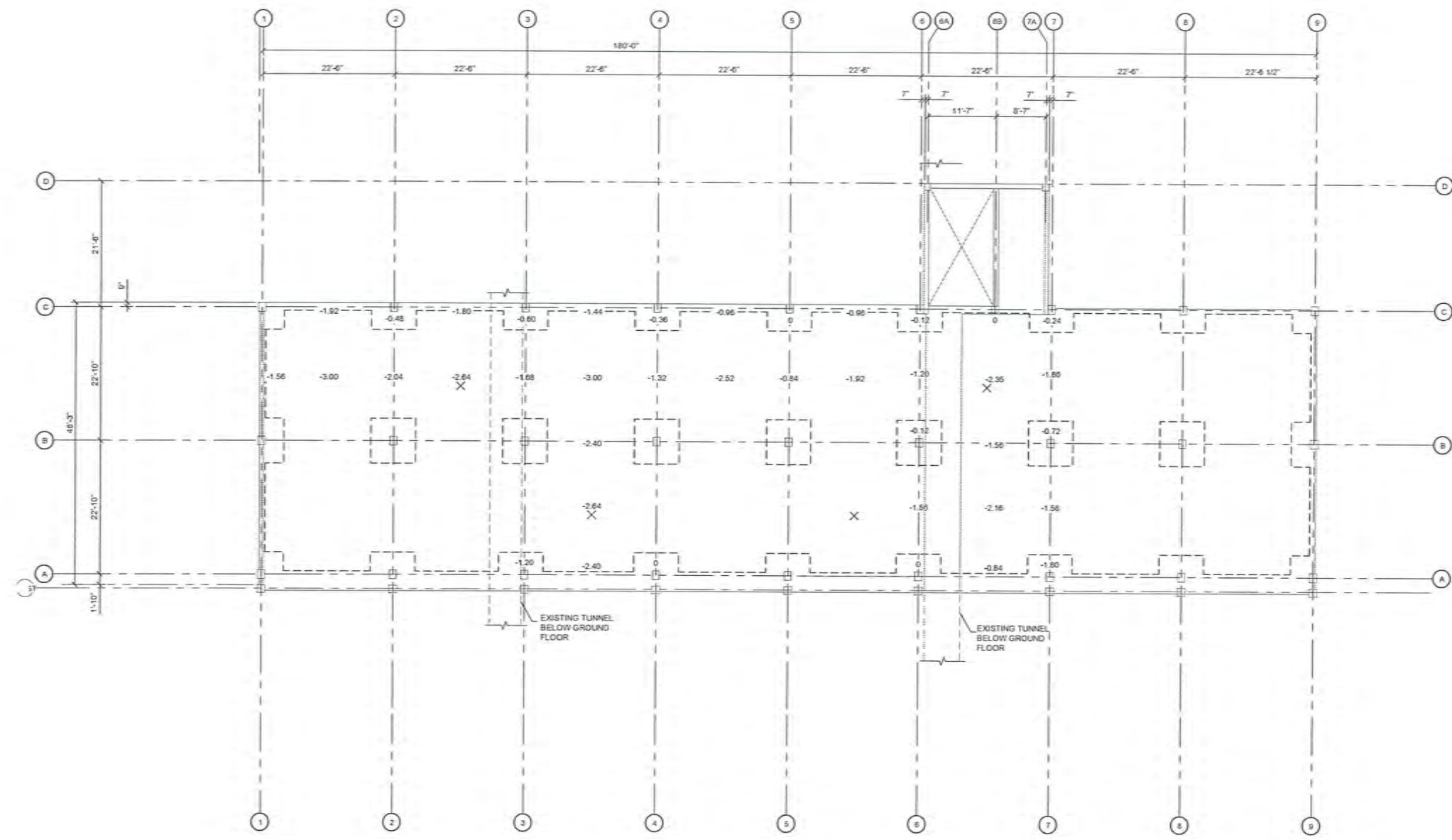
xx PSF SERVICE LIVE LOAD CAPACITY



KEY PLAN

A BAY LOAD CAPACITIES - ALL FLOORS



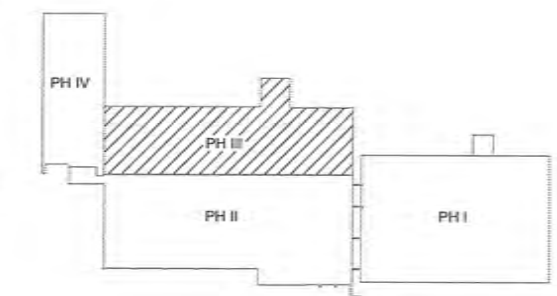


LEGEND:

- X.XX FIELD MEASURED DEFLECTION (INCHES) FROM HIGH POINT
- X HAMMER TEST LOCATION

NOTES:

1. IBC ALLOWABLE DEFLECTION (NOT INCLUDING SETTLEMENT) = 1.1 INCHES AT CENTER OF BAY
2. EXPECTED SHORT-TERM DEFLECTION UNDER SERVICE LIVE LOAD CAPACITIES (NOT INCLUDING SETTLEMENT) = 0.3 TO 0.5 INCHES AT CENTER OF BAY
3. EXPECTED LONG-TERM DEFLECTION UNDER SERVICE LIVE LOAD CAPACITIES (NOT INCLUDING SETTLEMENT) = 0.9 TO 1.5 INCHES AT CENTER OF BAY

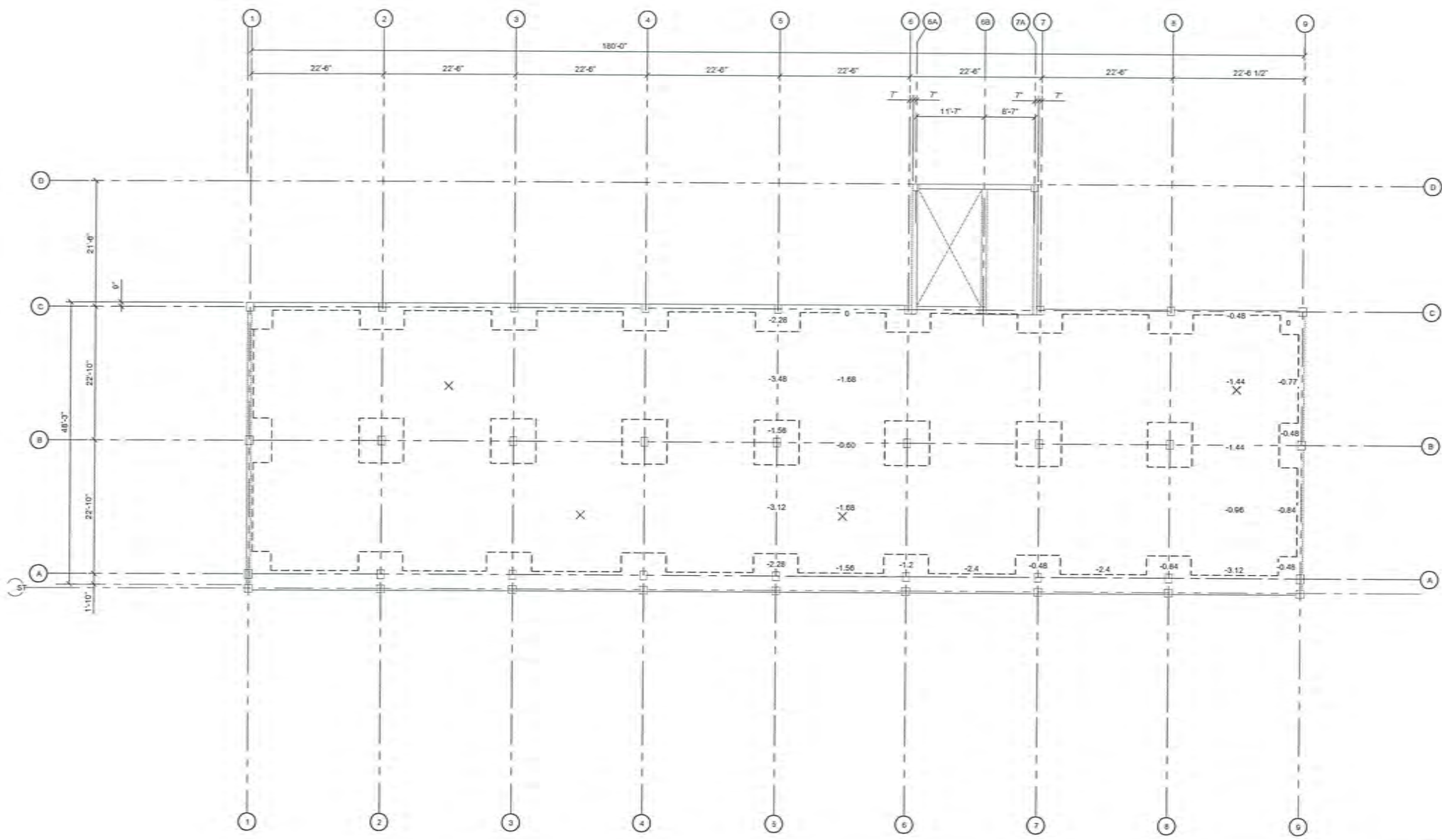


KEY PLAN

A SECOND FLOOR - PLAN - MEASURED DEFLECTIONS



P:\2010097\51007\Drawings\CAD\Phase III\Bohannon PH III_2nd FL_Meas.dwg Feb 08, 2010 - 5:30pm

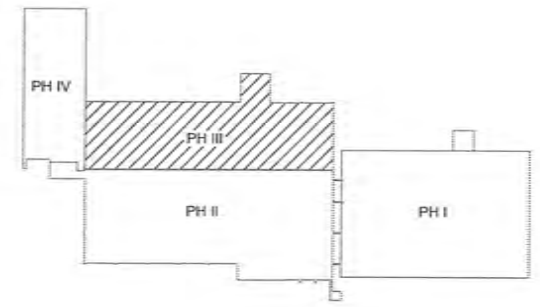


LEGEND:

- X.XX FIELD MEASURED DEFLECTION (INCHES) FROM HIGH POINT
- X HAMMER TEST LOCATION

NOTES:

1. IBC ALLOWABLE DEFLECTION (NOT INCLUDING SETTLEMENT) = 1.1 INCHES AT CENTER OF BAY
2. EXPECTED SHORT-TERM DEFLECTION UNDER SERVICE LIVE LOAD CAPACITIES (NOT INCLUDING SETTLEMENT) = 0.3 TO 0.5 INCHES AT CENTER OF BAY
3. EXPECTED LONG-TERM DEFLECTION UNDER SERVICE LIVE LOAD CAPACITIES (NOT INCLUDING SETTLEMENT) = 0.9 TO 1.5 INCHES AT CENTER OF BAY

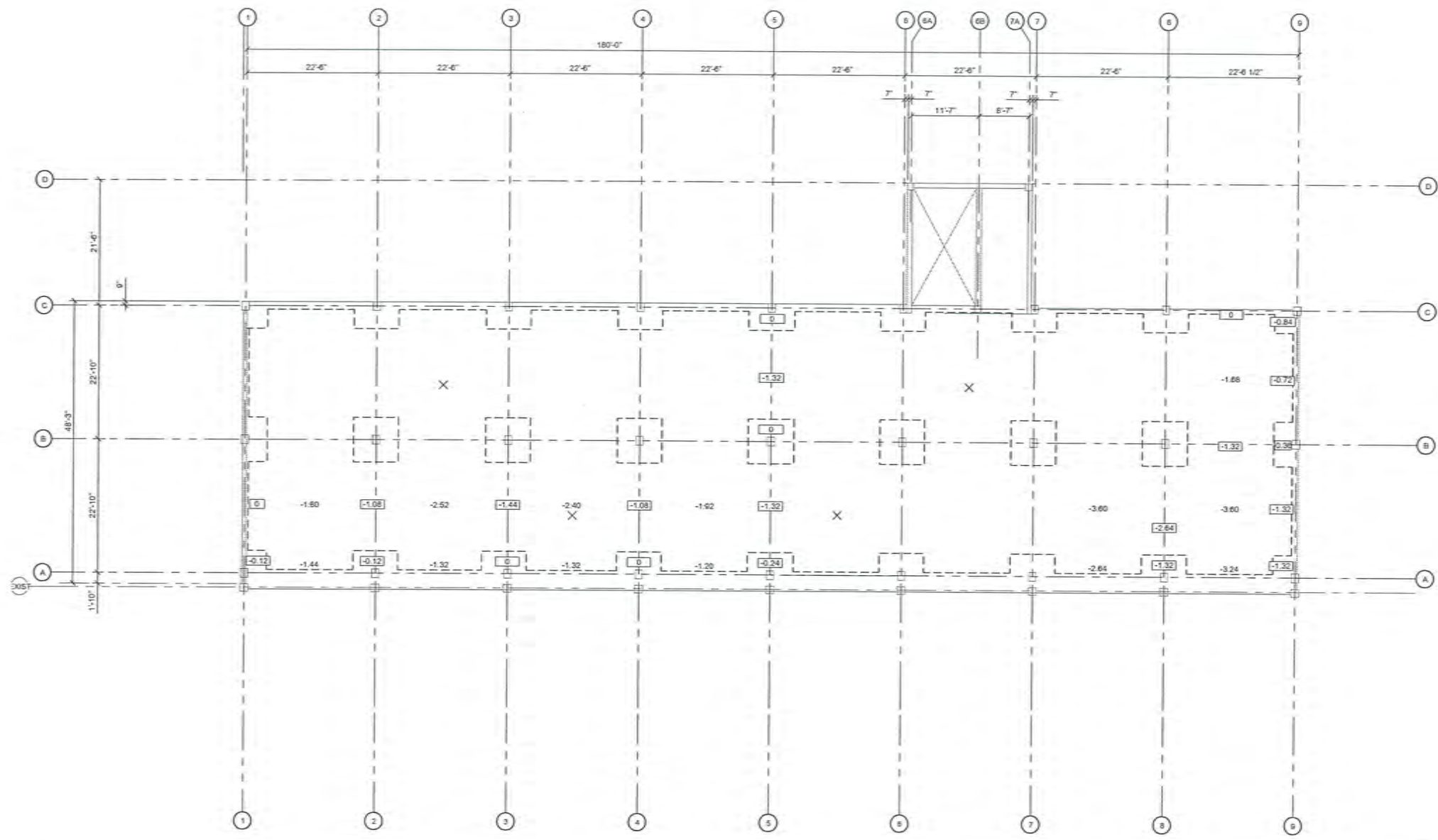


KEY PLAN

A THIRD FLOOR - PLAN - MEASURED DEFLECTIONS



P:\200807\Struct\Drawings\MOU\Phase III_SLF_Mid.dwg Tab 08, 2010 - 5:49pm

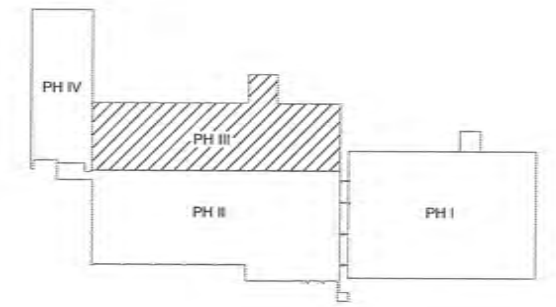


LEGEND:

- X.XX FIELD MEASURED DEFLECTION (INCHES) FROM HIGH POINT
- X HAMMER TEST LOCATION

NOTES:

1. IBC ALLOWABLE DEFLECTION (NOT INCLUDING SETTLEMENT) = 1.1 INCHES AT CENTER OF BAY
2. EXPECTED SHORT-TERM DEFLECTION UNDER SERVICE LIVE LOAD CAPACITIES (NOT INCLUDING SETTLEMENT) = 0.3 TO 0.5 INCHES AT CENTER OF BAY
3. EXPECTED LONG-TERM DEFLECTION UNDER SERVICE LIVE LOAD CAPACITIES (NOT INCLUDING SETTLEMENT) = 0.9 TO 1.5 INCHES AT CENTER OF BAY

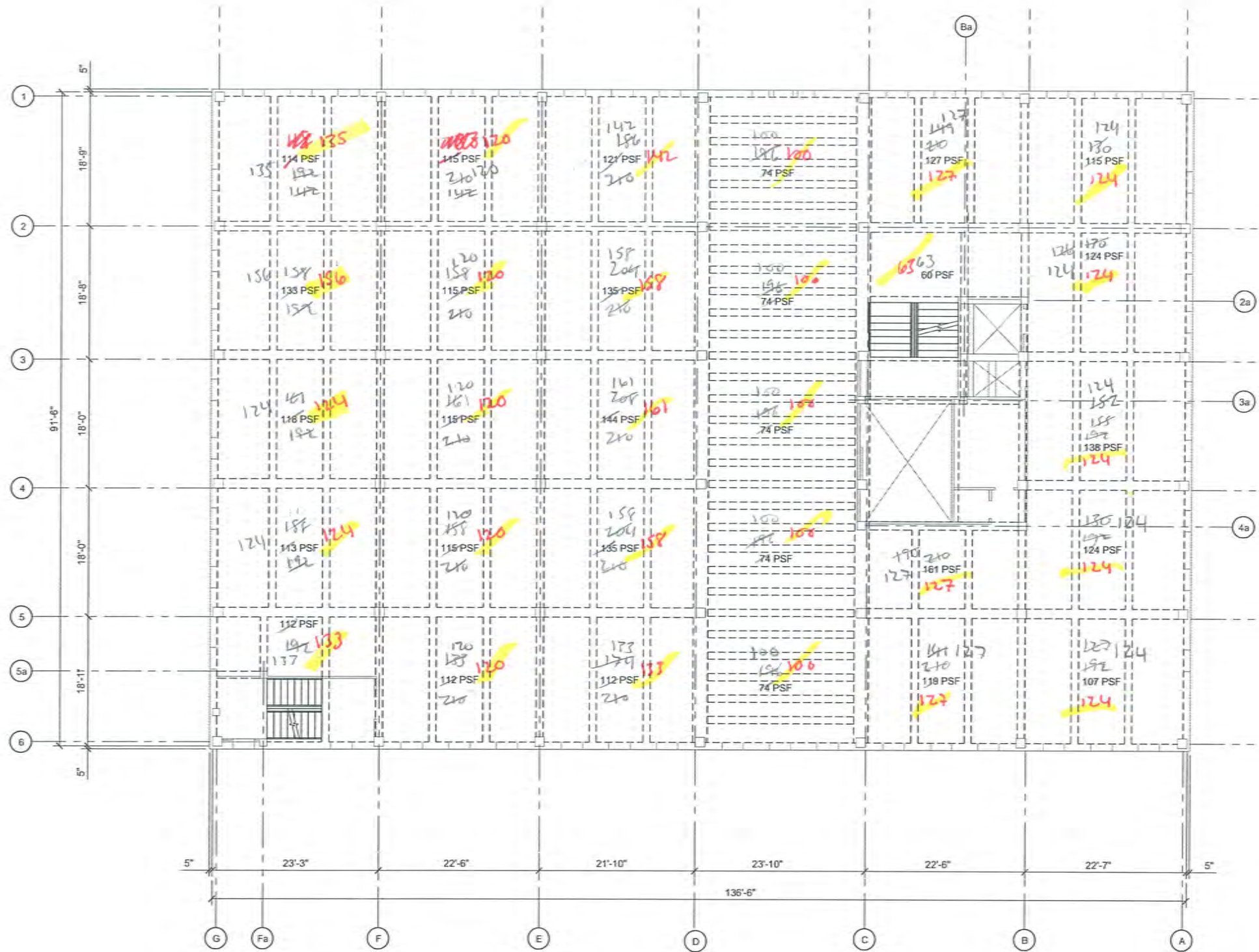


KEY PLAN

A FOURTH FLOOR - PLAN - MEASURED DEFLECTIONS



P:\2010097\STRUCT\Drawings\CAD\Draw PH II_4th_FF_00.dwg Feb 08, 2010 - 2:44pm

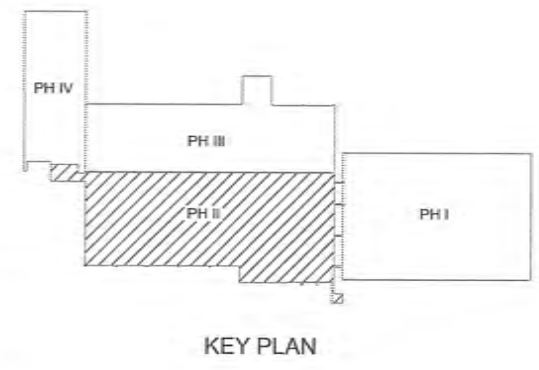


A BAY LOAD CAPACITIES - ALL FLOORS



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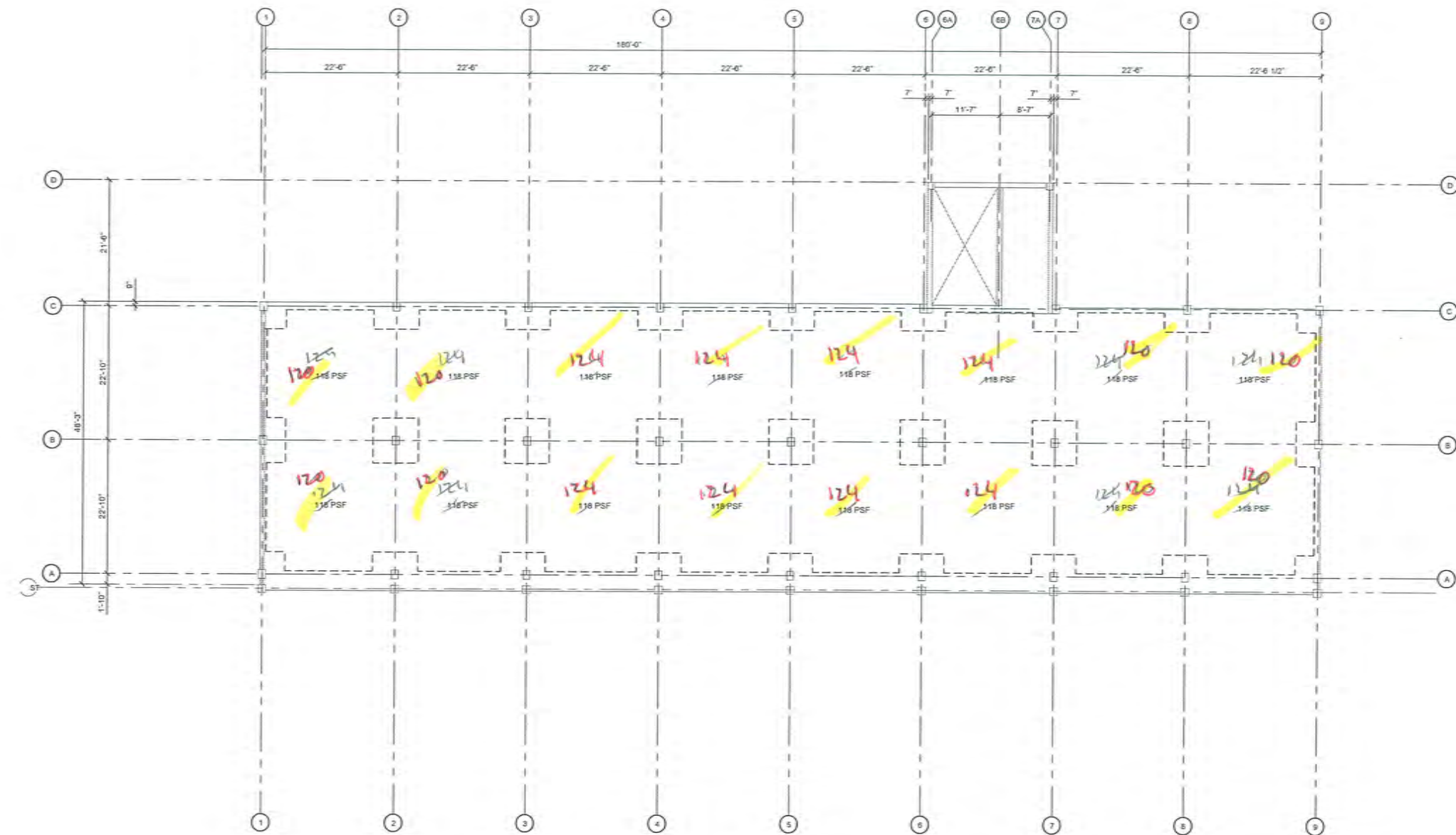
xx PSF SERVICE LIVE LOAD CAPACITY



A PLAN - BAY LOAD CAPACITIES - ALL FLOORS

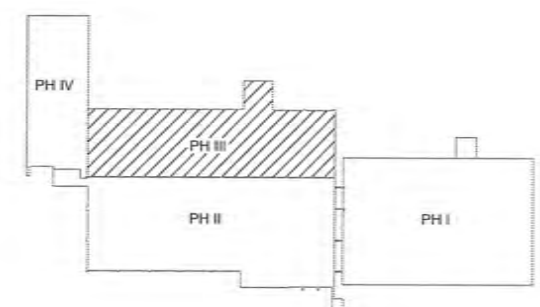


P:\21010097\STRUCT\Drawings\ACAD\Phase II\JP_BC.dwg Feb 05, 2010 - 5:20pm



LEGEND:

xx PSF SERVICE LIVE LOAD CAPACITY



KEY PLAN

A BAY LOAD CAPACITIES - ALL FLOORS



F:\2010097\STRUCT\Drawings\CAD\Phase III\JP_B.Cad.dwg Feb 05, 2010 - 5:35pm



SOUTHWEST ENGINEERING, INC.

475 Archuleta Road
Las Cruces, New Mexico
88005

Phone 575-526-3381
Fax 575-526-1762

January 15, 2010

OFS Engineering
Attn: David Bollschweiler
MSC 3545
PO Box 30001
Las Cruces, NM 88003-8001

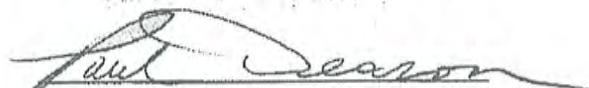
Re: Windsor Probe Results
NMSU Branson Library
SEI30021

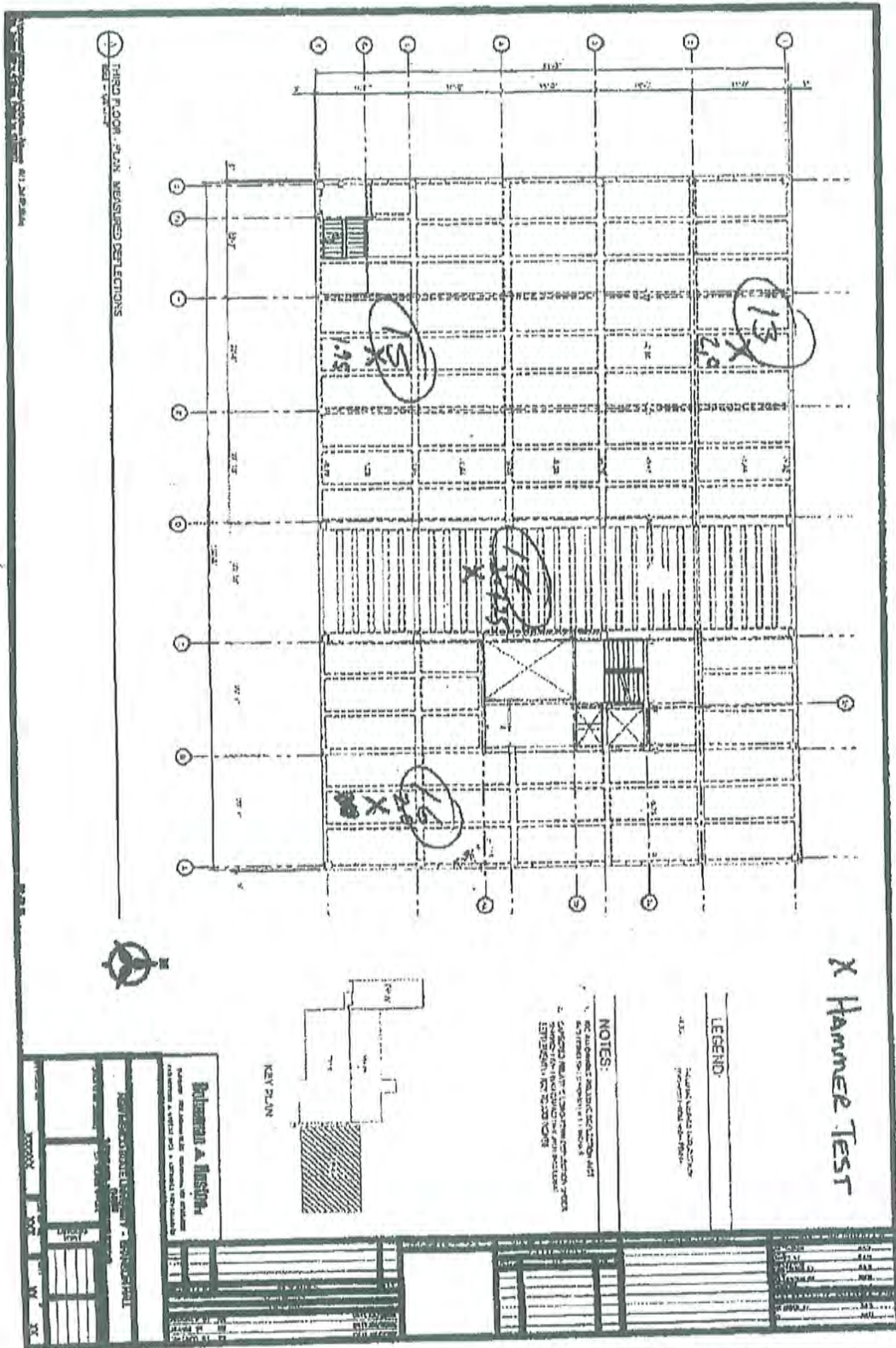
Southwest Engineering, Inc. tested the concrete floors for compressive strength at the above referenced project. Testing was done in compliance with ASTM C803 using Driver Unit No. 8728. The locations are numbered and marked on the enclosed plats. The results are as follows:

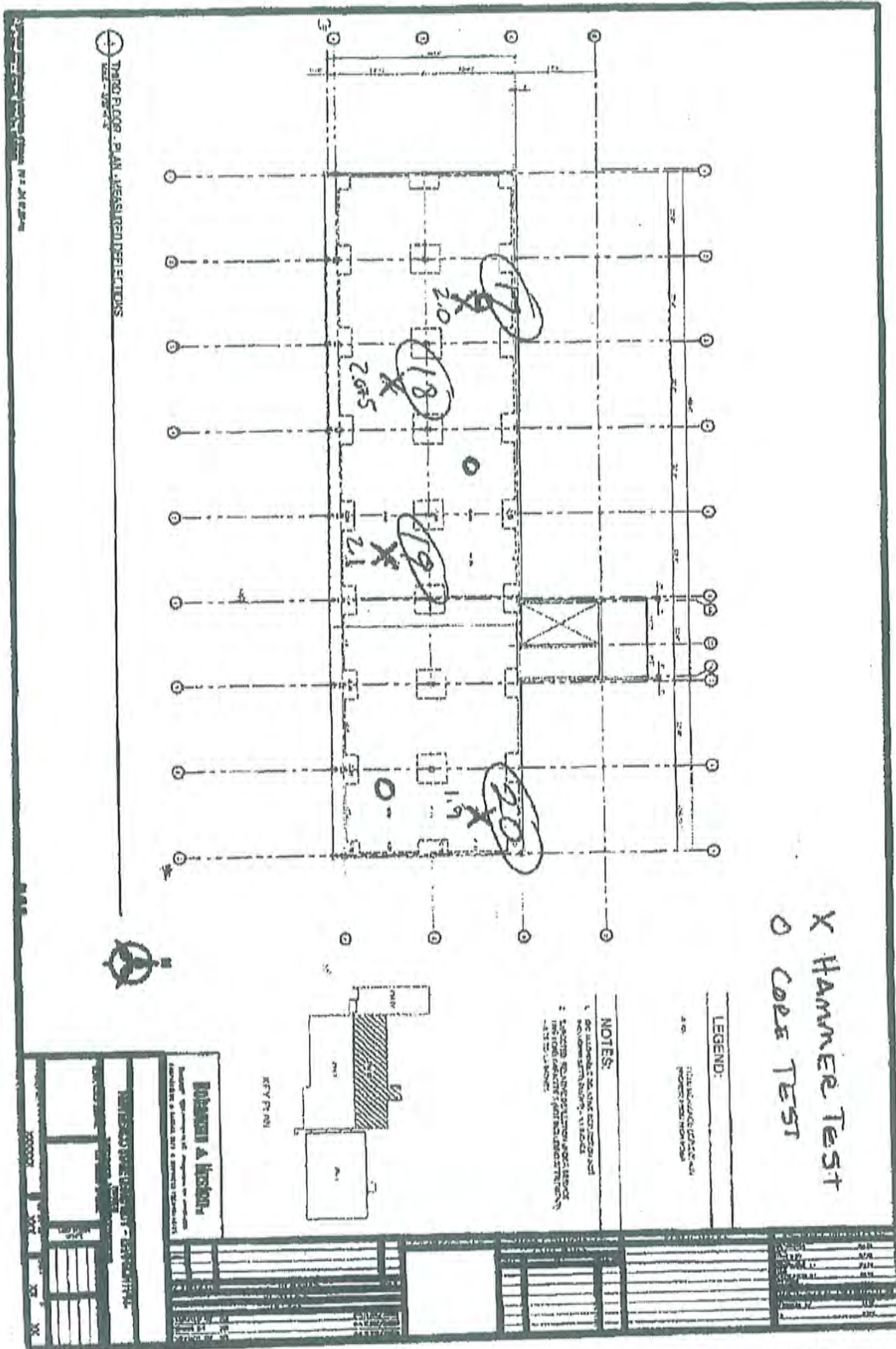
| Second Floor | | Third Floor | | Fourth Floor | |
|--------------|------|-------------|------|--------------|------|
| No. | PSI | No. | PSI | No. | PSI |
| 1 | 4525 | 13 | 4750 | 25 | 5850 |
| 2 | 5620 | 14 | 5850 | 26 | 5175 |
| 3 | 4750 | 15 | 4300 | 27 | 6050 |
| 4 | 5850 | 16 | 4750 | 28 | 5720 |
| 5 | 4975 | 17 | 4850 | 29 | 4100 |
| 6 | 5850 | 18 | 5400 | 30 | 5625 |
| 7 | 5175 | 19 | 5625 | 31 | 5725 |
| 8 | 6050 | 20 | 4100 | 32 | 6050 |
| 9 | 4975 | 21 | 6050 | 33 | 6140 |
| 10 | 4750 | 22 | 5850 | 34 | 5175 |
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| 12 | 5175 | 24 | 6050 | 36 | 5260 |

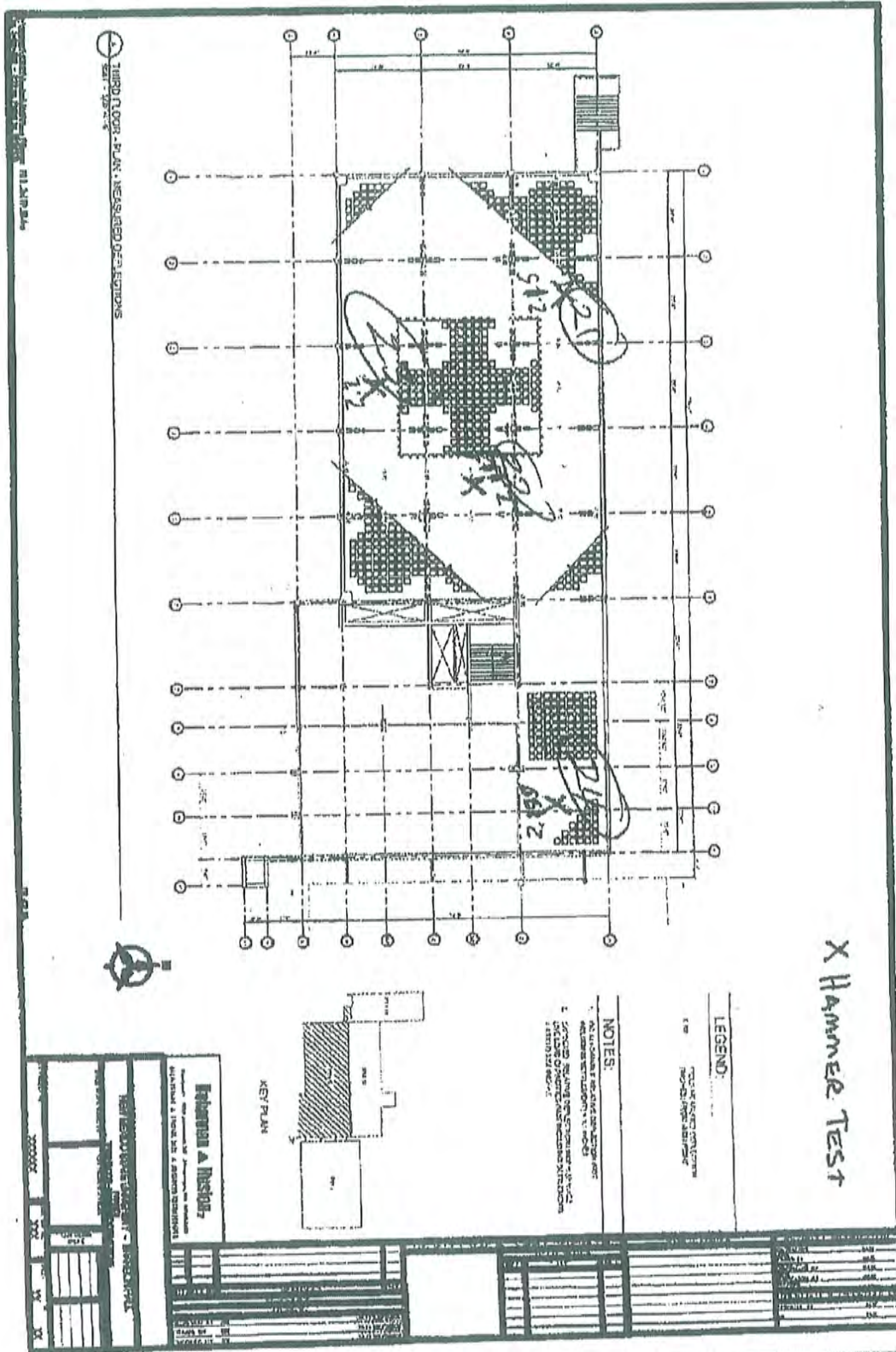
If you have any questions or require further information, please do not hesitate to contact this office.

SOUTHWEST ENGINEERING, INC.


Paul Deason







526-925

