SUBMITTAL TO THE BOARD OF SUPERVISORS
COUNTY OF RIVERSIDE, STATE OF CALIFORNIA 335

FROM: Economic Development Agency

SUBMITTAL DATE:
April 14, 2011

SUBJECT: 11th Street Jail Demolition – In-Principle

RECOMMENDED MOTION: That the Board of Supervisors approve in-principle the demolition of the 11th Street Jail.

BACKGROUND: A seismic evaluation report (Exhibit A) of the 11th Street Jail found the facility grossly overstressed and in need of significant seismic strengthening. It also described water intrusion problems, structural cracking, and deteriorating electrical, plumbing, and mechanical systems which are all in need of replacement. The report concluded that the facility had reached the end of its 50 year life cycle, and the cost to strengthen, abate, and repair the facility would likely exceed the cost to replace the facility. The Sheriff's Department has been advised of this recommendation and expressed no objection. The anticipated cost for demolition of the structure and rehabilitation of the south wall of the Historic Courthouse is $500,000.

Robert Field
Assistant County Executive Officer/EDA

FINANCIAL DATA	**Current F.Y. Total Cost:**	$ 0	**In Current Year Budget:**	Yes
	Current F.Y. Net County Cost:	$ 0	**Budget Adjustment:**	No
	Annual Net County Cost:	$ 0	**For Fiscal Year:**	2010/11

COMPANION ITEM ON BOARD OF DIRECTORS AGENDA: No

SOURCE OF FUNDS: To Be Determined

Positions To Be Deleted Per A-30 ☐

Requires 4/5 Vote ☐

C.E.O. RECOMMENDATION: APPROVE

BY:
County Executive Office Signature Jennifer L. Sargent

Departmental Concurrence

Reviewed by CIRTEAM
Christopher Hans

Dep't Recomm.: ☐ Consent ☒ Policy

Per Exec. Ofc.: ☐ Consent ☒ Policy

Prev. Agn. Ref.: 3.27 of 4/20/10 | **District:** 2 | **Agenda Number:** 3.50

RF:LB:TM:CW:FG:trv FM08110004032 10792 S:\Project Management Office\FORM 11'S\Form 11's in Process\10792 11th St Jail Demo In-Principle 042011

(Rev 08/2010) EDA-001a-F11 Form 11 (Rev 06/2003)

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Exhibit A

January 18, 2007

Mr. Steven B. Jewett, MBA
Deputy Director, Maintenance
Facilities Management
3133 Mission Inn
Riverside, California 92501

Reference: Seismic Evaluation Report
County of Riverside Probation and Jail Facilities
Riverside, California

Dear Mr. Jewett:

This letter will summarize our Seismic Evaluation Report dated January 2007. Johnson & Nielsen Associates has completed the seismic evaluation of the County Probation and Jail Facilities based on the requirements of the current code. Our consensus regarding the current state of the old jail portion of the structure is as follows:

1. The County Probation and Jail Facilities building is approaching its 50-year life cycle.

2. The building has not experienced any severe earthquakes in its lifetime; therefore, the integrity of the building was not validated with the designed earthquake. The closest earthquakes (Landers and Big Bear Quakes, 1992) occurred more than 50 kilometers away from this building. The structure is located about 10 kilometers south of the San Jacinto Fault, which has potential to generate earthquake with 7.2 magnitudes.

3. The Building is considered non-ductile, meaning the lateral load resisting system could fail without warning in the event of severe earthquake.

4. Under the current design seismic load, the building is grossly overstressed and needed strengthening with a minimum of 10-inch thick of reinforced concrete.

5. The foundation is not sufficient to support the needed rehabilitation and would require strengthening with wider concrete.

6. Falling hazards on the cantilever slab around the building may exists due to spalling in cracking and deteriorating slab.

7. There is no sufficient building separation to dissuade pounding between buildings. This may result in damages on connecting bridges and adjacent buildings due to severe ground shaking.

8. Numerous water intrusion problems, which were mostly the result of structural cracking and leaking of old plumbing system, have put the building under constant repair.

9. We estimate that the minimum cost to improve the structural integrity of the building and foundation is $2.7 million. This amount does not include the following:

   - Possible Asbestos Abatement
   - Demolition and disposition of environmental waste
   - Removal and replacement of architectural elements in work area
   - Removal and replacement of electrical elements in work area
   - Removal and replacement of plumbing elements in work area
   - Removal and replacement of Mechanical system in work area
   - ADA compliance
   - Relocation of occupants during repair/rehab work
   - Repair of water damages
   - Other non-structural aspects

Based on the premises presented above, Johnson & Nielsen Associates concluded that the cost to repair and perform seismic rehabilitation of the jail facility would likely exceed the cost of rebuilding.

Thank you for this opportunity of assisting you in this endeavor. We hope that this letter will be helpful in determining what necessary action to take. If you have any questions, please do not hesitate to let us know.

Very truly yours,

Johnson & Nielsen Associates

WEN Y. LIN

WYL/hec

SEISMIC EVALUATION

COUNTY OF RIVERSIDE PROBATION AND JAIL FACILITIES

Riverside, California

L06-48

for

County of Riverside
Facilities Management Department
3133 Mission Inn Avenue
Riverside, California 92507

JOHNSON & NIELSEN ASSOCIATES
Consulting Structural Engineers
911 South Primrose Avenue, Suite D
Monrovia, California 91016

January 2007

TABLE OF CONTENTS

Executive Summary

This report evaluates the seismic performance of the building's lateral load resisting system. Seismic evaluation of the Probation and Jail Facilities of the County of Riverside, California is based on the California Building Code (CBC) Chapter 16A, Division VI-R, "Earthquake Evaluation and Design for Retrofit of Existing State-Owned Buildings."

The structural system of the Probation and Jail Facilities building was designed in 1960 by Brandow & Johnston Structural Engineers of Los Angeles. The four-story building is essentially rectangular in plan measuring approximately 109 feet in east-west direction and 73 feet in north-south direction. The building has an 8-foot high, 8-inch thick concrete parapet wall around the perimeter. On top of the parapet, it has a 6-foot wide, 5-inch thick cantilever collar slab and decorative beams at 21'-6" on center (o.c.) at the west and south side of the building. All exterior walls above the second floor, except the north wall, were covered with 4-inch thick brick veneer. At the northwest corner of the building, the exterior concrete stair is used as emergency access from the fourth and third floors down to the first floor. On the second, third, and fourth floors, there is a bridge tunnel connecting the adjacent court building. The bridge tunnels are constructed of 6-inch concrete side walls and 4½-inch top and bottom slabs. The building is supported on spread footing at interior columns and continuous footing at perimeter concrete shear walls.

Roof framing consists of 5-inch thick cast-in-place one-way concrete slab over 24-inch deep beams and 26-inch deep girders. The concrete beams and girders are supported by interior columns and pilasters at the exterior concrete bearing shear walls.

All floor framings consist of 5 inches cast-in-place one-way concrete slab except at the second floor, which is *4½ inches thick*. Concrete slabs are supported by 16 to 24-inch deep cast-in-place concrete beams and 26-inch deep girders. Beams and girders are supported by interior concrete columns and pilasters in exterior concrete bearing shear walls. The size of columns varies, ranging from 16 square inches at upper level to 20 square inches at the first floor. Please refer to Pictures 1 through 5 and Figures 1 through 5 for elevation and plans of the building. The concrete used for construction of slabs, beams, columns and concrete walls has a compressive strength of 3750 pounds per square inch (psi).

The lateral load resistances of the building are provided by 10-inch thick concrete shear walls between the second floor and the roof and 13-inch concrete shear walls on the first floor.

The existing building is located approximately 10 kilometers south of the San Jacinto Fault (San Jacinto Valley) per Page O-33 of "Maps of Known Active Fault" by ICBO, 1997 Edition.

Stress and shrinkage cracks on concrete slab and shear walls along with some interior water damages were observed during our site walk. Epoxy injection to repair cracks wider than 0.01 inch is recommended.

Three-dimensional finite element computer model of the building was developed using the program ETABS. Rigid diaphragms were modeled for both the roof and all floors. Besides performing evaluation of the strength of members of the building, the drift of the building under the design load was also determined. The evaluation was compared to the drift limit stated in CBC, Section1630.9.

In addition to static force analysis, a linear response spectrum analysis was performed using the standard CBC spectrum curve with $C_a = 0.44$ and $C_v = 0.64$. The first twelve modes of vibration were included resulting in a participation of 100% of the building mass. The modes were combined using the complete quadratic combination (CQC) method.

Total base shear of the response spectrum analysis was scaled down to about the same magnitude for static force procedure per CBC 1631.5.4. P-Δ effects, which include dead and live loads, are included in the analysis. Story shear and member forces for each modes considered were combined using CQC combination. Directional effects of the seismic load were considered using the square root of the sum of the square (SRSS) method of seismic loads in two orthogonal directions.

Shear and flexure demands were checked for all concrete shear walls and spandrel beams. It is determined that all shear walls are grossly overstressed both in shear and flexural strength. All spandrel beams are also inadequate in both shear and flexural strength. It appears that the entire building needs an overhaul of the lateral load resisting system.

The existence of possible falling hazards was studied. Items that were specifically investigated include the following:

A. The connection of exterior anchored brick veneer appears to have the capacity to support the design seismic force. No hazard was identified here.

B. It appears that the cantilever perimeter slab is reinforced adequately to resist net upward seismic force due to vertical acceleration.

The most recent significant earthquakes that hit Riverside area are Landers and Big Bear earthquakes which occurred in 1992. The larger Landers earthquake, with a magnitude of 7.4, is about 90 kilometers away from the building while the Big Bear earthquake, with a magnitude of 6.5, is about 50 kilometers from the site. Although the building appears to have survived both earthquakes, it does not meet the life-safety standard outlined in CBC Chapter 16, Division VI-R. Therefore, strengthening of the existing building is strongly recommended. One possible technique that may be used to repair shear walls is the application of shotcrete to enhance flexural and shear capacity. A minimum of ten inches shotcrete will be needed for strengthening the shear walls. The building's seismic demand will also be increased due to additional building weight from application of shotcrete. In order to support the increased wall weight, the existing continuous footing at the perimeter of shear walls will also need to be widened. The estimated cost to repair and strengthen the building is $2.7 million.

1.0 Introduction

This report was prepared at the request of the County of Riverside of California. It evaluates the seismic performance of the building's lateral load resisting system and studies potential falling hazards. Seismic evaluation of the Probation and Jail Facilities of the County of Riverside is based on the California Building Code Chapter 16A, Division VI-R, "Earthquake Evaluation and Design for Retrofit of Existing State-Owned Buildings." Procedures to evaluate the performance of the building include visual observation of the building, review of all available plans, development of three-dimensional computer model of building's structural system and production of engineering calculations to determine the structural demand and capacity of the lateral load resisting system.

The available plans of the building for review include the following:

- Architectural plans – Sheets A1 through A40 and Sheet 1 prepared by Herman O. Ruhnau, Architect of Mission Inn Rotunda, Riverside, California, dated April 11, 1960.
- Structural plans – Sheets S-1 through S-18 prepared by Brandow & Johnston, Structural Engineers of Los Angeles, dated April 11, 1960.

This study represents our opinion of the structural conditions of the facility based on our general review of the plans, site observation and calculations. This review is not intended to preempt the responsibilities of the original design consultants in any way.

2.0 Structural System of Existing Building

The structural system of the Probation and Jail Facilities building was designed in 1960 by Brandow & Johnston, Structural Engineers of Los Angeles. The four-story building is essentially rectangular in plan measuring approximately 109 feet in east-west direction and 73 feet in north-south direction. Please refer to Figures 1 through 5 for structural framing plans. Typical story height for the building is 12 feet except for the second floor which is 15 feet. The building has an 8-foot high, 8-inch thick concrete parapet wall around the perimeter. On top of the parapet, it has a 6-foot wide, 5-inch thick cantilever collar slab and decorative beams at 21'-6" on center (o.c.) at the west and south side of the building. All exterior walls above the second floor, except the north wall, were covered with 4-inch thick brick veneer. The exterior wall of the first floor is partially retaining, height ranging from one foot at the northwest corner to six feet at the northeast corner. At the northwest corner of the building, the exterior concrete stair is used as emergency access from the fourth and third floors down to the first floor. On the second, third, and fourth floors, there is a bridge tunnel connecting the adjacent court building. The bridge tunnels are constructed of 6-inch concrete side walls and 4½-inch top and bottom slabs. At the separation joint, no physical separation is allowed between adjacent buildings at the bridge tunnels. Please see Pictures 14 and 15. The building is supported on spread footing at interior columns and continuous footing at perimeter concrete shear walls. The allowable soil bearing pressure, stated on drawings, is 4000 pounds per square feet (psf). Slab-on-grade concrete consists of a 4-inch thick concrete with 6x6 – 10x10 W.W.F. No soils report is available for review. The original soils report was prepared by LeRoy Crandall & Associates of Los Angeles.

Roof framing consists of 5-inch thick cast-in-place one-way concrete slab over 24-inch deep beams and 26-inch deep girders. The concrete beams and girders are supported by interior columns and pilasters at the exterior concrete bearing shear walls.

All floor framings consist of 5 inches cast-in-place one-way concrete slab except at the second floor, which is 4½ inches thick. Concrete slabs are supported by 16 to 24-inch deep cast-in-place concrete beams and 26-inch deep girders. Beams and girders are supported by interior concrete columns and pilasters in exterior concrete bearing shear walls. The size of columns varies, ranging from 16 square inches at upper level to 20 square inches at the first floor. Please refer to Pictures 1 through 5 and Figures 1 through 5 for elevation and plans of the building. The concrete used for construction of slabs, beams, columns and concrete walls has a compressive strength of 3750 pounds per square inch (psi). All reinforcing steel used are ASTM-A15, grade 40 steel.

The lateral load resistances of the building are provided by 10-inch thick concrete shear walls between the second floor and the roof and 13-inch concrete shear walls on the first floor. Walls around the elevators are full height, 8-inch concrete walls, extending from the foundation up to 8 feet above the roof of the building. All shear walls were found to be reinforced meeting the minimum 0.25% code requirements. Minimum jamb reinforcing is provided for all shear walls at the ends and openings with two #5. Vertical reinforcing in all concrete columns, except at the fourth floor, are tied with #5 spirals at 2.5 inches o.c. and lap spliced with 28-bar diameter above floor. Column ties above the fourth floor are #3 bars at 14 inches o.c. Reinforced concrete pilasters are provided at each grid lines intersecting with exterior bearing walls. Pilasters are reinforced with additional vertical steel ranging from six #8 at upper level to twelve #11 at first floor. Reinforcing ties are provided at each pilaster with #3 bars at 10 inches o.c. for 10 inches thick wall and #3 bars at 13 inches o.c. for 13 inches thick wall.

The existing building is located approximately 10 kilometers south of the San Jacinto Fault (San Jacinto Valley) per Page O-33 of "Maps of Known Active Fault" by ICBO, 1997 Edition.

3.0 Observed Damages

During our casual field walk on December 13, 2006, numerous damages were observed as indicated in the list below. Pictures shown in this report only represent the condition for the type of damages described. The extent of damages shall be assessed by a testing laboratory employed by the County.

1. Cracking in perimeter cantilever concrete slab at top of parapet walls are prominent from view. Please refer to Pictures 2 through 7.

2. Stress and shrinkage cracking at exterior concrete shear wall of first floor. Please refer to Pictures 10 through 13. Similar cracking might have occurred on other floors which are not visible due to presence of veneer.

3. Damages on wall finishes due to water leakage in plumbing pipes were found on many interior partitions and ceilings. Please refer to Pictures 16 through 21. In one location, evidence of damage to existing concrete slab was observed. Please

refer to Picture 18, where rust stain of reinforcing steel on concrete slab is visible.

4. Insufficient building separation to adjacent building. Please refer to Pictures 14 and 15. No real separation was found.

5. The roof is ponded with rain water a week after the first rainfall of the season. It appeared that the roof is improperly sloped and maybe insufficient roof drains. Please refer to Picture 9.

4.0 Seismic Evaluation Criteria

The evaluation of this building is based on the criteria contained in the 2001 CBC Chapter 16A, Division VI-R. Both static force procedure and CBC response spectrum analysis with 5% damping were used. See Figure 6 for the plot of the response spectrum curve used. Demands placed on the structure by earthquake ground motion and the ultimate capacity of the structure system were compared using load combination equations 44A-5 and 44A-6 of 2001 CBC.

$$\Phi\, C_n = 1.05\, D + 0.25\, L + \beta E \qquad (44A\text{-}5)$$

$$\Phi\, C_n = \beta E - 0.9D \qquad (44A\text{-}6)$$

Generally, most structural elements constructed according to the current building code will have sufficient ductility to allow demands greater than their calculated capacity with β=1.0. Please refer to Figure 7 for comparison of material behavior with or without ductility. Concrete buildings built before 1976 are considered limited or non-ductile. Clear indication of lack of ductility of this building can be found from typical reinforcing details of construction documents which only provides 28-diameter splice of reinforcing steel. Under current code requirements, the same rebar splice will require 38 to 64-diameter lap splice. Due to lack of ductility of this building, the β factor ranging from 2.5 to 3.5 were used in the above equations for shear wall stress check. For the evaluation of this existing structure, the following β value was used:

Element	β factor
Shear walls governed by shear	
Shear	3.5
Flexure	2.5
Boundary	3.0

Three-dimensional finite element computer model of the building was developed using the program ETABS. Rigid diaphragms were modeled for both the roof and all floors. This assumption significantly reduces the computational complexity of analyzing the building's lateral load resisting system and also improves computational accuracy for shear wall analysis. Wall, column, and beam elements were used to model the building. An isometric representation of the computer model is shown on Figure 8. Besides performing evaluation of the strength of members of the building, the drift of the building under the design load was also determined. The evaluation was compared to the drift limit stated in CBC, Section1630.9.

In addition to static force analysis, a linear response spectrum analysis was performed using the standard CBC spectrum curve with $C_a = 0.44$ and $C_v = 0.64$. The first twelve modes of vibration were included resulting in a participation of 100% of the building mass. The modes were combined using the complete quadratic combination (CQC) method. The three fundamental modes (two translational modes and one rotational mode) of buildings with uncracked stiffness are shown on Figure 9. As expected, it was observed that the north-south direction translation had the longest period of 0.138 second, given the narrow width of the shear wall in that direction. The second translation mode of 0.114 second was primarily on the east-west direction. Lastly, the third mode of 0.079-second vibration is rotational translation. Numerous iterations were required before a model can realistically represent the existing structure. Much of the modeling complexity is the result of the presence of high parapet, perimeter cantilever slab and interior gunite partition that occurred at the third and fourth floors. Mass of these elements were assumed lumped at the respective floor and roof diaphragms.

Vertical distribution of base shear is very similar between the static force procedure and response spectrum method. In static procedure, 43% of base shear is distributed at the roof level, whereas with the response spectrum analogue, 44 % of base shear is distributed to the roof level. Total base shear of the response spectrum analysis was scaled down to about the same magnitude for static force procedure per CBC 1631.5.4. P-Δ effects, which include dead and live loads, are included in the analysis. Story shear and member forces for each modes considered were combined using CQC combination. Directional effects of the seismic load were considered using the square root of the sum of the square (SRSS) method of seismic loads in two orthogonal directions.

The model was further refined by considering changes in the building stiffness due to concrete cracking. The building's global stiffness was modeled as cracked with 50% of the uncracked stiffness as recommended by FEMA 310. The first period for the fundamental modes of the 50% cracked model were increased from 0.138 to 0.17 second. This shift in period is well within the maximum range of design response spectrum curve. The Figure shows the shape of response spectrum curve changes in dynamic behavior with decreasing stiffness (increase in period).

5.0 Seismic Performance of Existing Building

The calculated maximum drift of 0.00035h with a 50% cracked section was found well within the allowable limits of 0.025h. The existing building is found to be very stiff but is insufficiently reinforced. Existing reinforcing steels in all shear walls and pilasters were input into the computer model for stress check. One of the major problems with the existing building is that it does not have enough splice length on reinforcing steels. All existing bars are spliced with a 28-bar diameter lap, which is far shorter than the 54-bar diameter for shear walls required by today's code. Shear and flexure demands were checked for all concrete shear walls and spandrel beams. It is determined that all shear walls are grossly overstressed both in shear and flexural strength. All spandrel beams are also inadequate in both shear and flexural strength. It appears that the entire building needs an overhaul of the lateral load resisting system. Additional computer models were created with added 4 and 6 inches of shotcrete to existing building resulting in 13% and 18% increase in base shear. The weight of the building is also increased by 21% and 27%, respectively.

For 6-inch shotcrete strengthening scheme, all shear walls were reinforced with maximum allowed reinforcing steels. More than 50% of the shear walls are still found overstressed due to shear forces exceeding the maximum allowed by the member. The amount of reinforcing steel required for spandrel beams are found impossible to install within limited 6 inches shotcrete space. It is estimated that at least a 10-inch thick of shotcrete is required to have a workable space for installation of both shear and flexural reinforcing steels.

The perimeter cantilever slab over parapet was checked per CBC 1630.11 for potential falling hazard due to vertical seismic acceleration and found to be adequate.

The existence of possible falling hazards was studied. Items that were specifically investigated include the following:

A. The connection of exterior anchored brick veneer appears to have the capacity to support the design seismic force. No hazards were identified here.

B. It appears that the cantilever perimeter slab is reinforced adequately to resist net upward seismic force due to vertical acceleration.

6.0 Repair Recommendations

Repair of observed damages listed in Section 3.0 above shall be considered as follows:

1. Cracking in existing collar slab over perimeter parapet may not present immediate structural problem. However, the cracking should be sealed to prevent further damages to the reinforcing steel which may cause spalling to covering concrete.

2. Cracking on existing concrete shear walls wider than 0.01 inch shall be repaired with epoxy injection to restore the strength of the existing shear walls.

3. It appeared that the old existing plumbing system needs an overhaul. Water damages from leaks of existing piping are visible at various walls on the first and second floors. Complete replacement of the existing system maybe in order. A plumbing engineer shall be consulted with this problem.

4. To minimize effect of building ponding due to seismic deflection, the building shall be physically separated with a minimum of 2-inch gap between adjacent buildings at each connecting tunnels. Seismic joint shall also be installed around all sides of each tunnel at the separation joints.

5. Insufficient roof slope and drainage system shall be addressed by the Architect. Ponding of water is part of the water leakage problem of this building.

7.0 Seismic Retrofit Recommendations

The most recent significant earthquakes that hit Riverside area are Landers and Big Bear earthquakes which occurred in 1992. The larger Landers earthquake, with a magnitude of 7.4, is about 90 kilometers away from the building while the Big Bear earthquake, with a magnitude of 6.5, is about 50 kilometers from the site. Although the building appears to have survived both earthquakes, it does not meet the life-safety standard outlined in CBC Chapter 16, Division VI-R. Therefore, strengthening of the existing building is strongly recommended. One possible technique that may be used to repair shear walls is the application of shotcrete to enhance flexural and shear capacity. A minimum of ten inches shotcrete will be needed for strengthening the shear walls. The building's seismic demand will also be increased due to additional building weight from application of shotcrete. In order to support the increased wall weight, the existing continuous footing at the perimeter of shear walls will also need to be widened.

In addition to shear wall strengthening, removing the perimeter collar slab and existing exterior veneer shall also be considered to reduce the mass of the building, which will in turn reduce seismic loading of the building.

8.0 Cost Estimate

With the presence of brick veneers on most parts of the exterior walls and narrow space between adjacent buildings, the strengthening work must be performed from the inside of an occupied building. The general condition of the building will significantly affect the cost of repair. Some of the anticipated difficulties are as follows:

A. Removal and protection of building contents

B. Removal and replacement of ceiling and floor partitions and wall finishes in work area

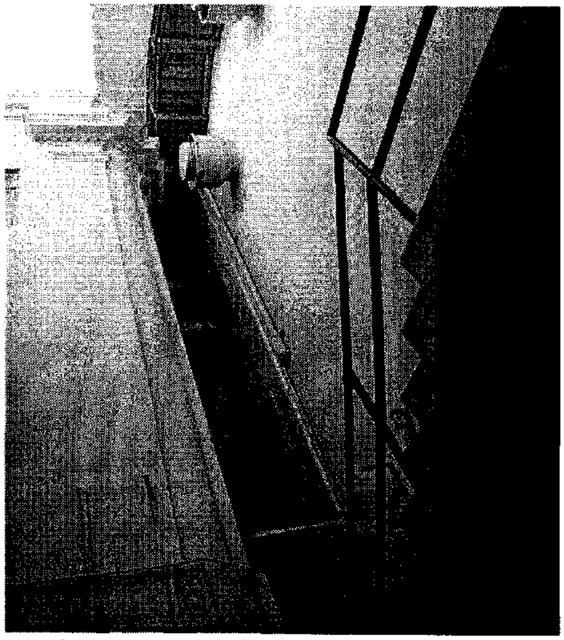
C. Limited work hours

D. Control between work and occupied areas of the building

A conceptual estimate of cost to strengthen the building and repair cracks are as follows:

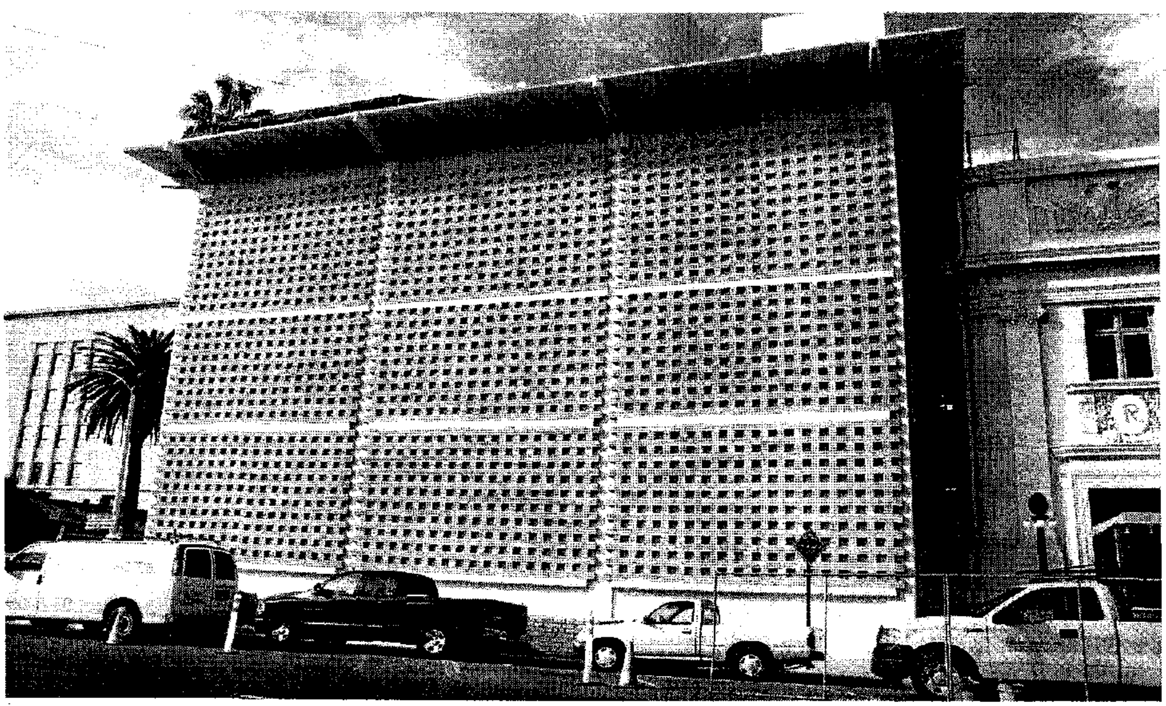
1.	Repair cracks wider than 0.01 inches on perimeter cantilevered slabs	$ 150,000.00
2.	Repair cracks wider than 0.01 inches on existing concrete walls	$ 970,000.00
3.	Strengthen existing concrete shear walls with 10-inch shotcrete	$ 1,520,000.00
4.	Widen perimeter shear wall footings	$ 86,000.00
	Total	$ 2,726,000.00

The above cost estimate presented does not include costs associated with extensive removal and replacement of Architectural, Electrical and Mechanical finishes or other non-structural aspects that must always be considered during seismic rehabilitation.

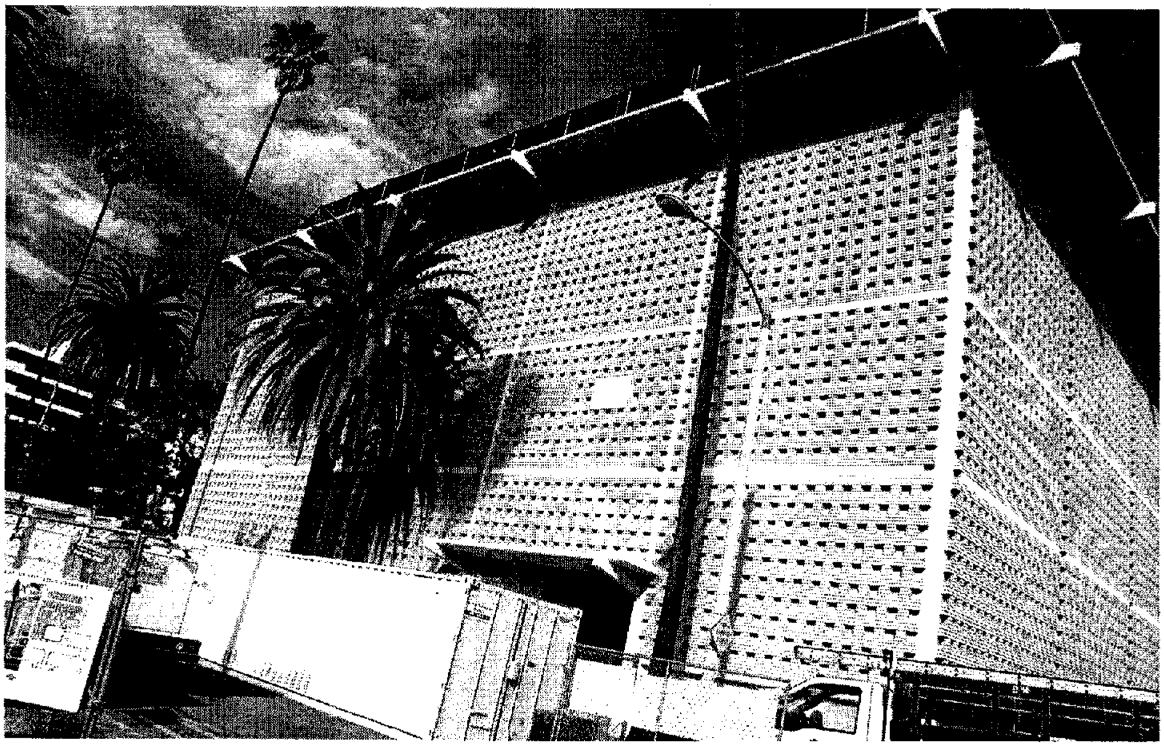


Picture 1 – North-West Elevation at Adjacent Building

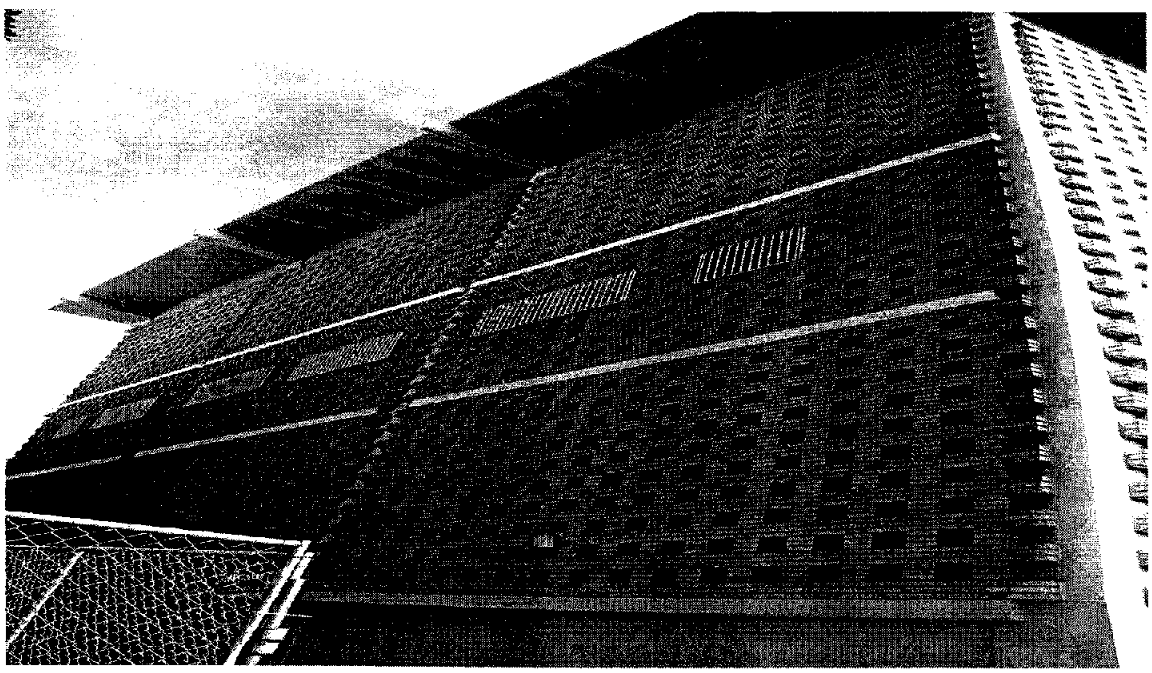
Picture 2 – North-East Elevation at Adjacent Building



Picture 3 – East Elevation

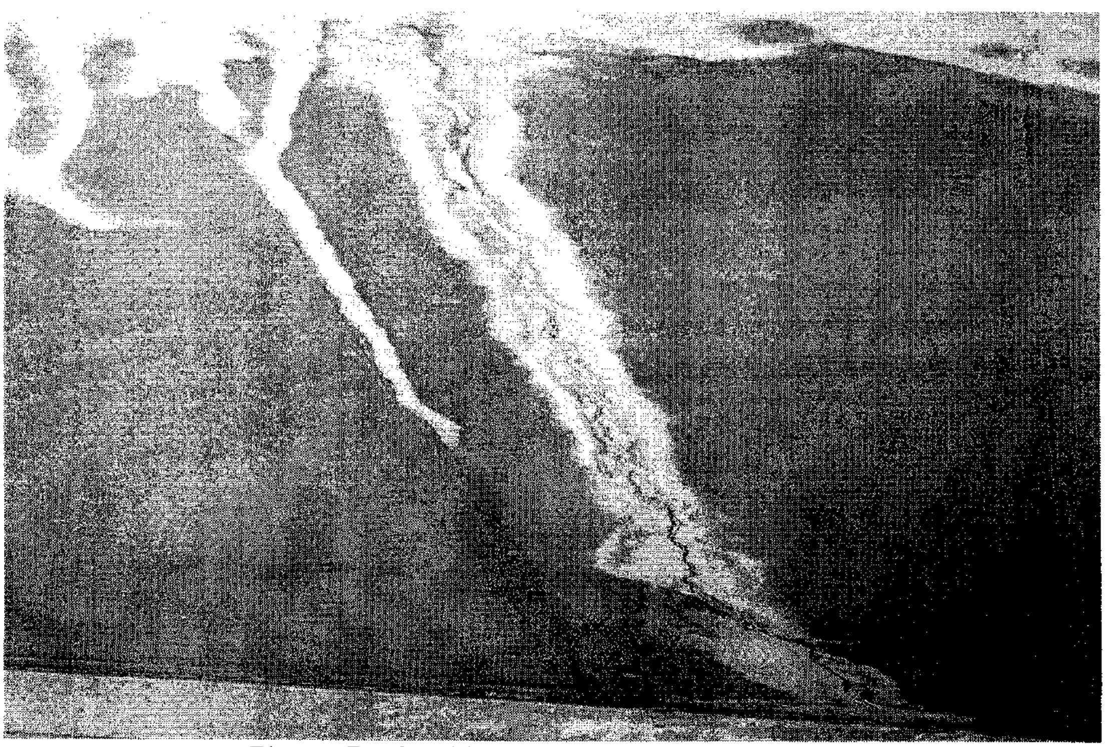


Picture 4 – South Elevation

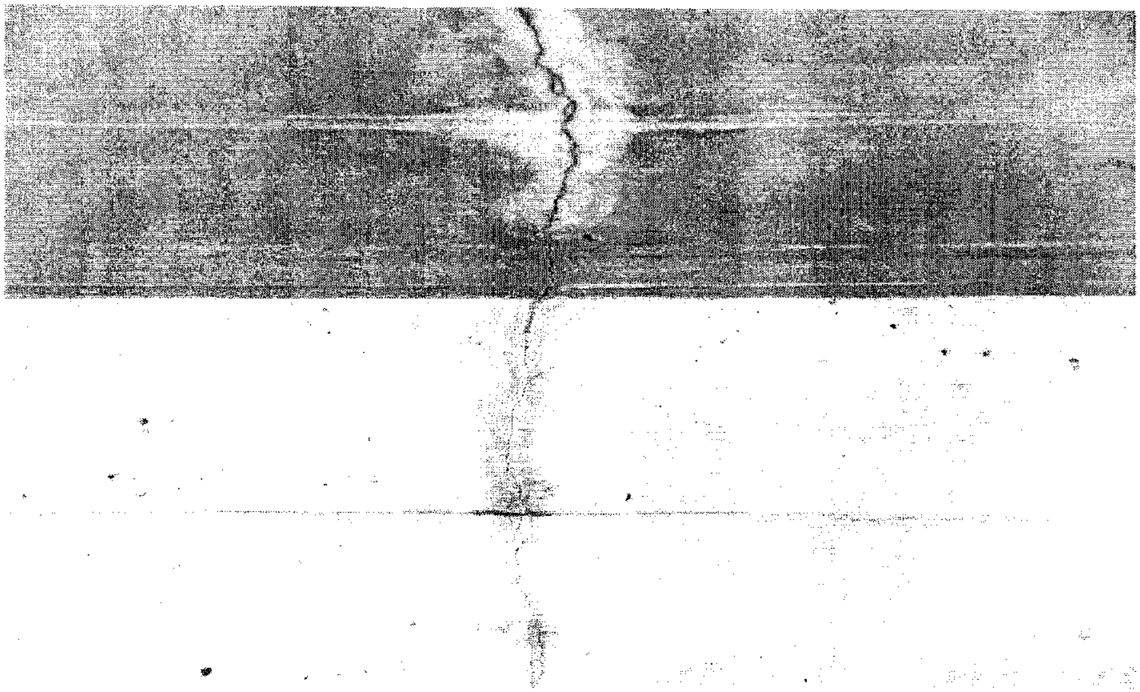


Picture 5 – West Elevation

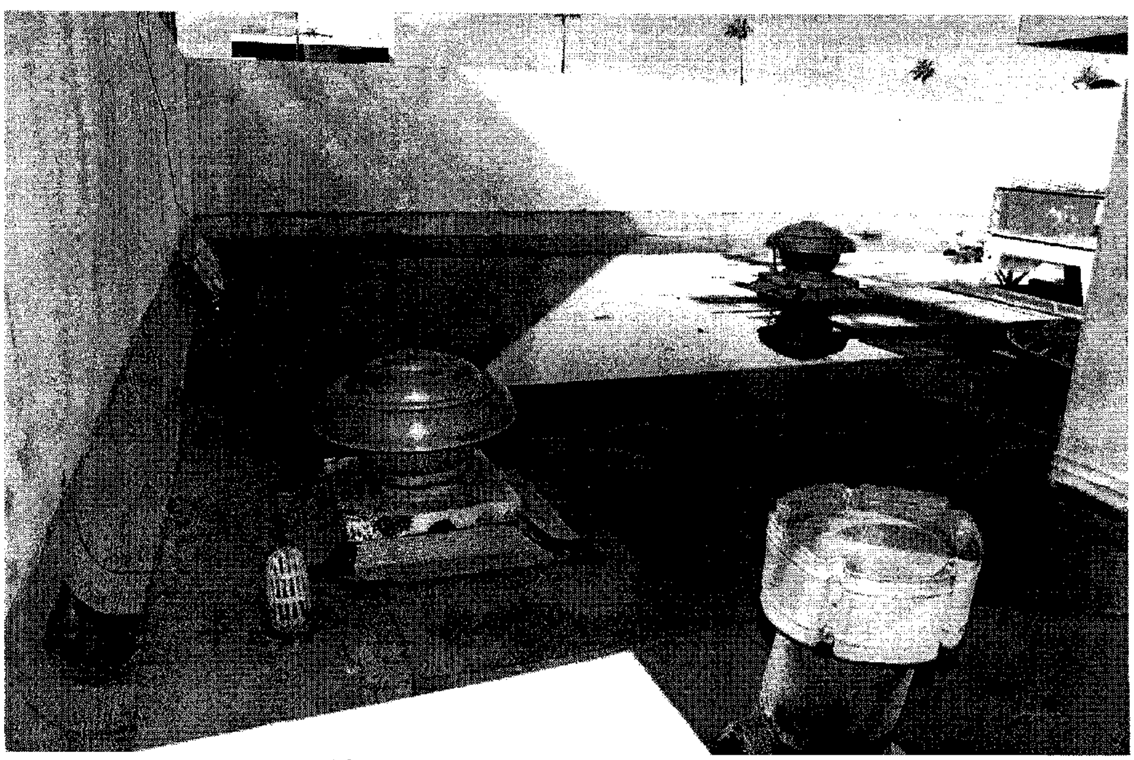
Picture 6- Cracking on perimeter cantilever slab



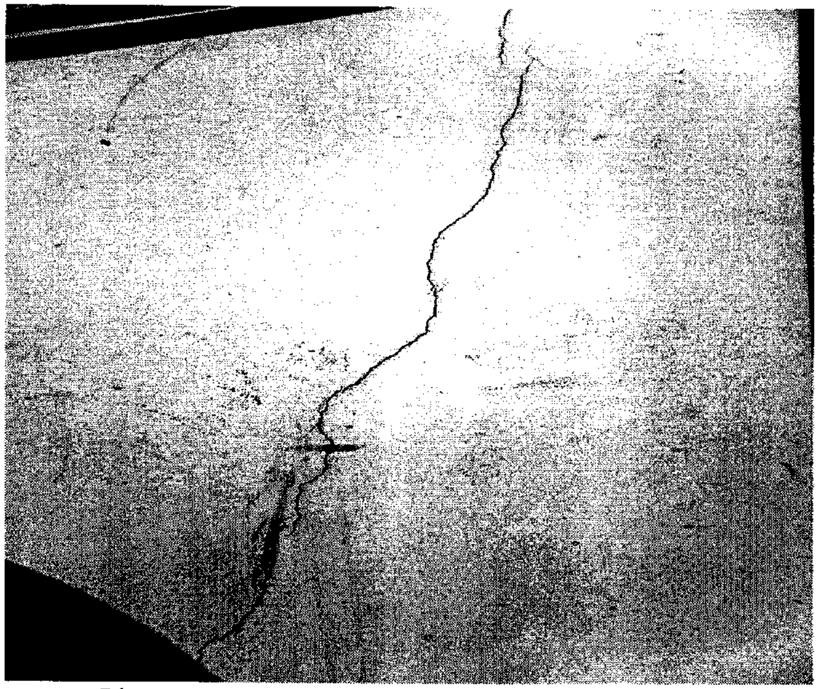
Picture 7 – Cracking on perimeter cantilever slab



Picture 8 – Cracking on perimeter cantilever slab and wall

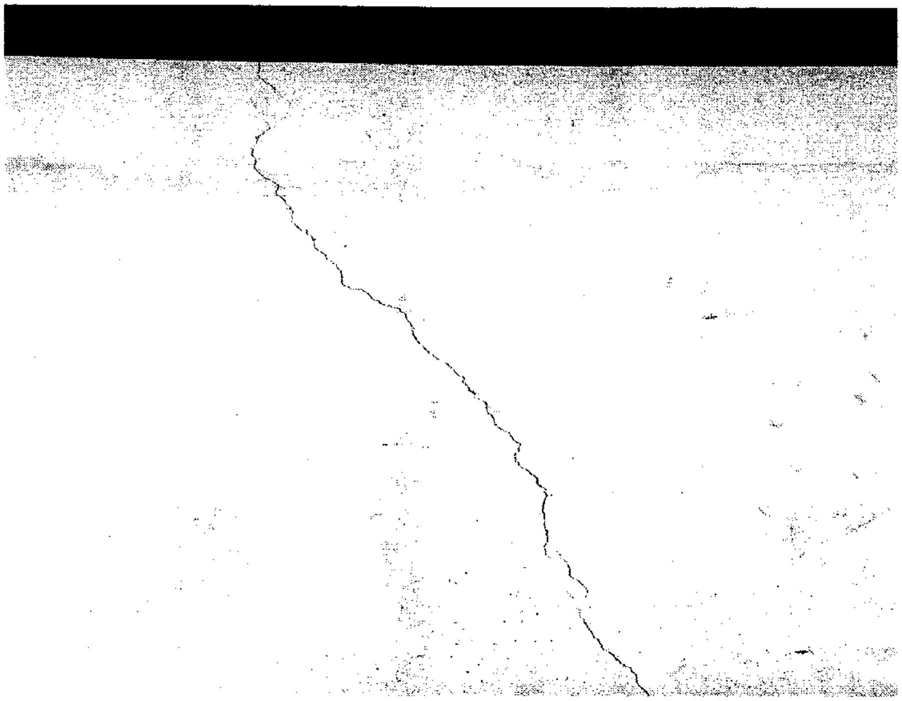


Picture 9 – Water ponding on roof slab

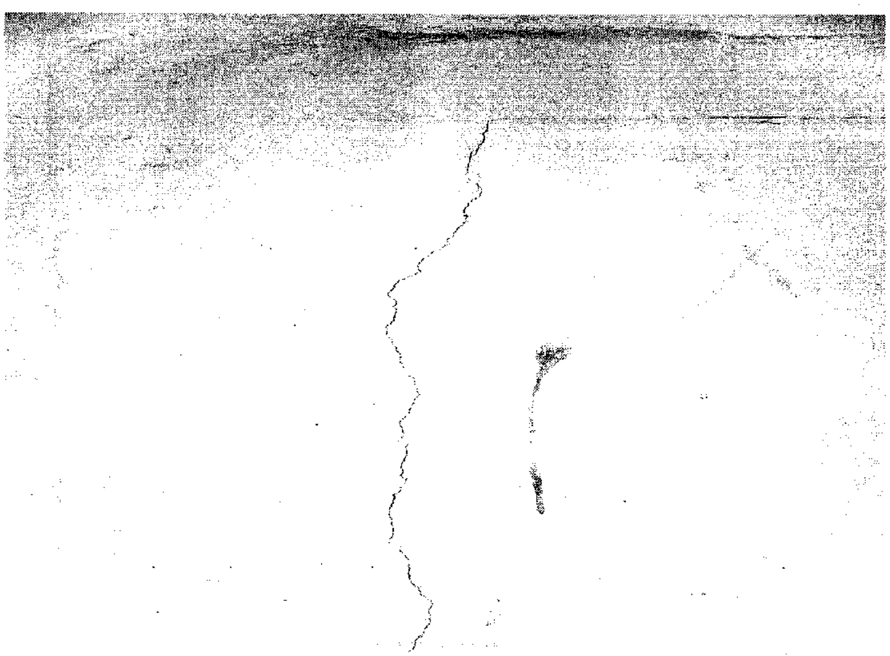


Picture 10 – Cracking on first floor perimeter wall

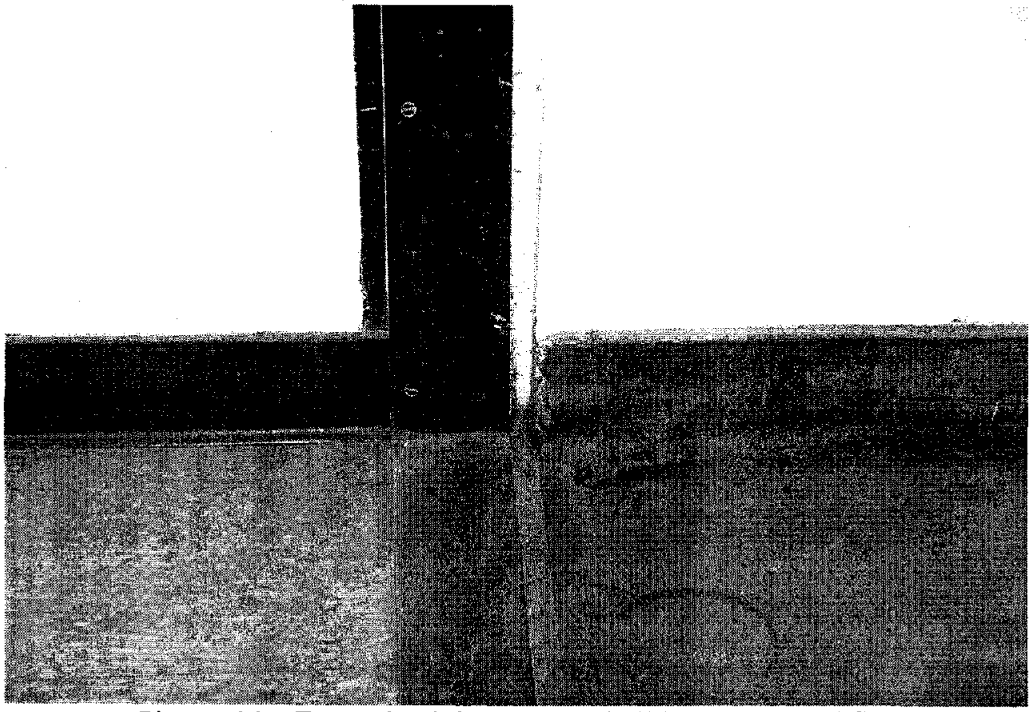
Picture 11 – Cracking on first floor perimeter wall



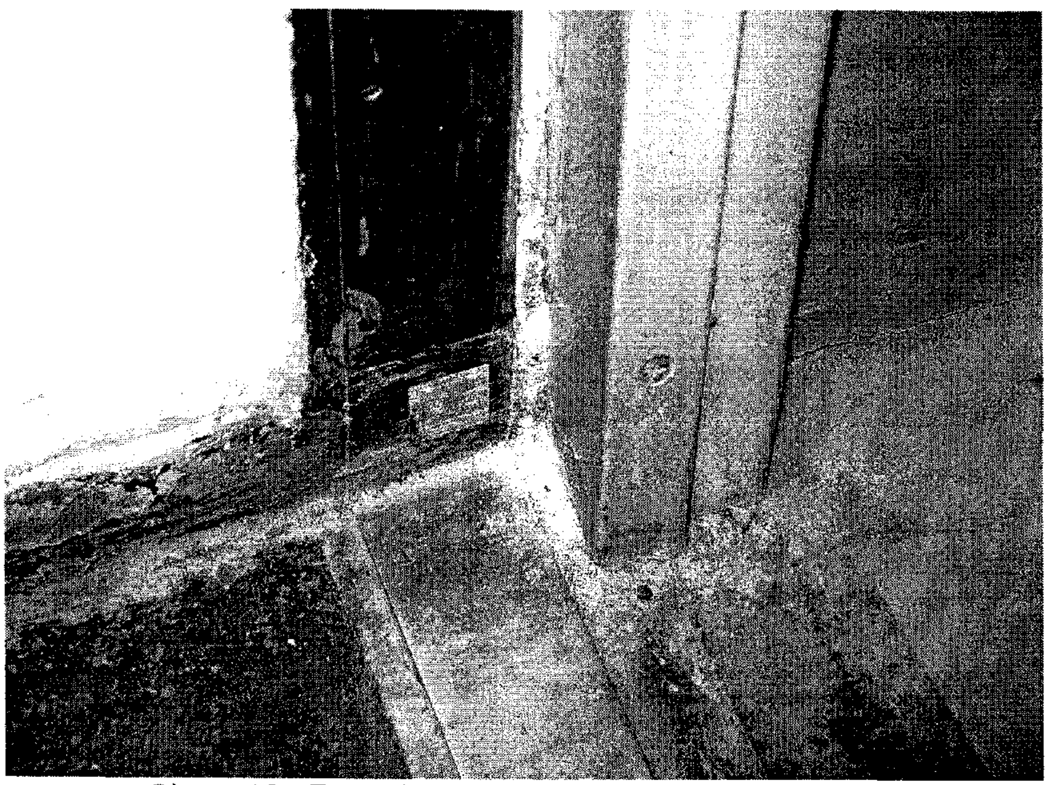
Picture 12 – Cracking on first floor perimeter wall



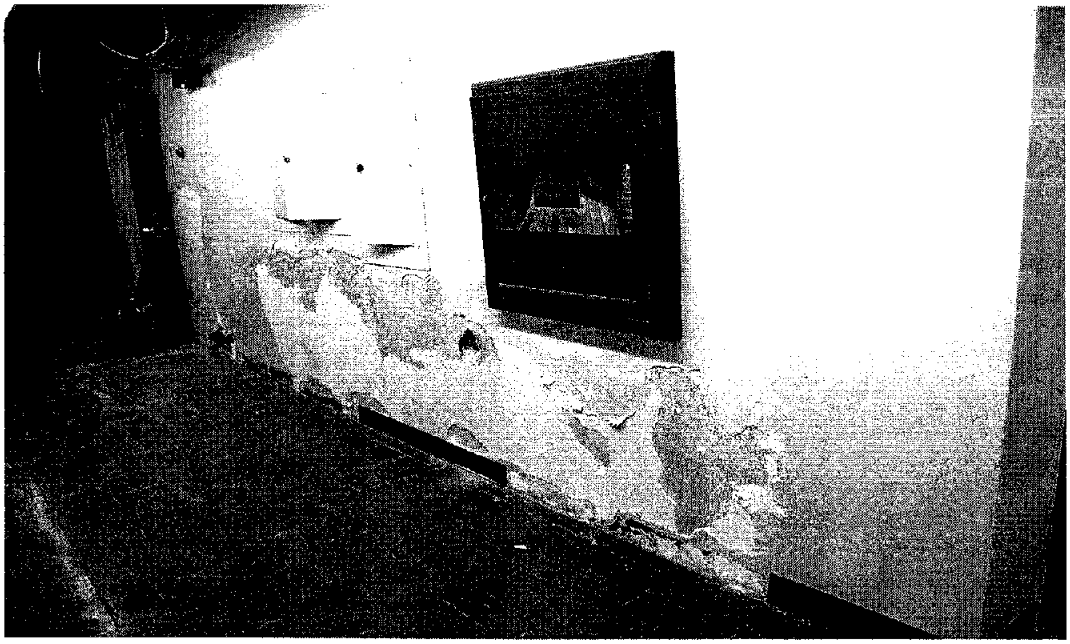
Picture 13 – cracking on first floor perimeter wall



Picture 14 – Expansion joint between buildings at second floor

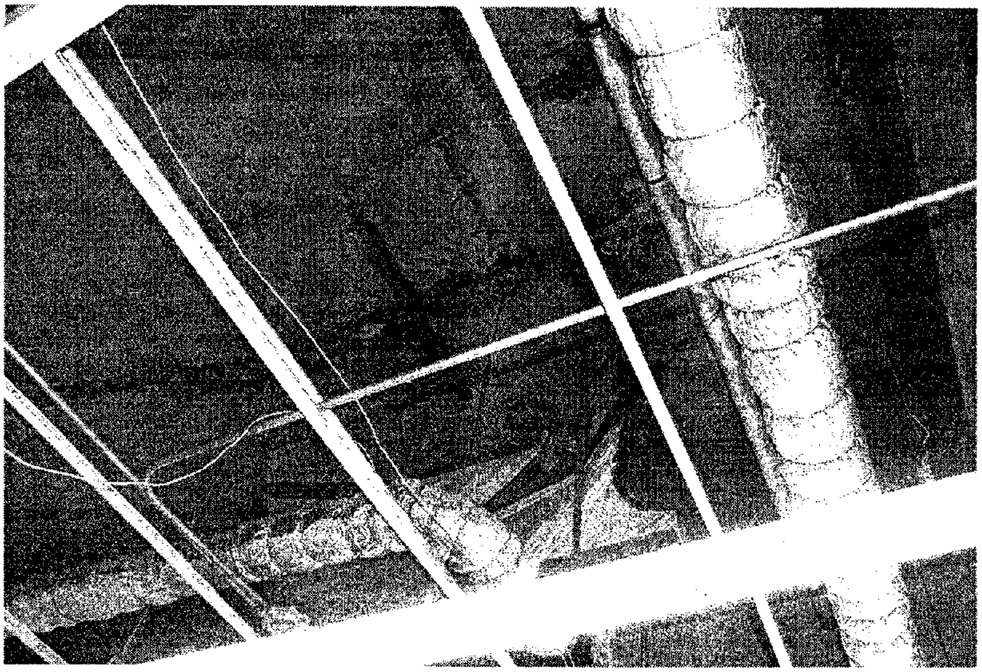


Picture 15 – Expansion joint between buildings at third floor

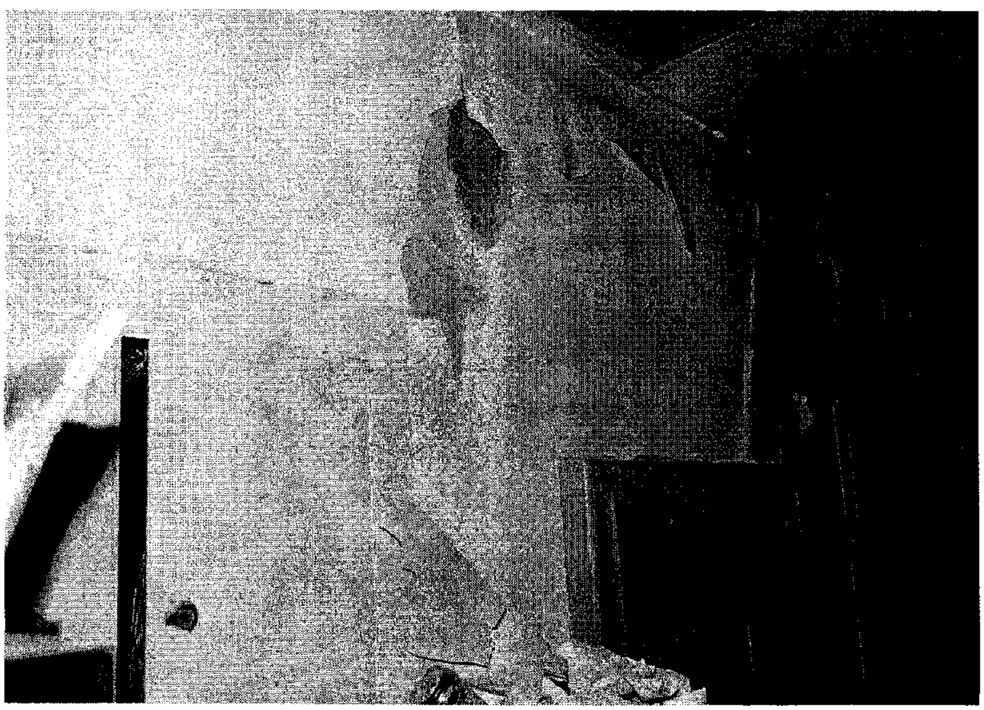


Picture 16 – Water-damaged interior partition

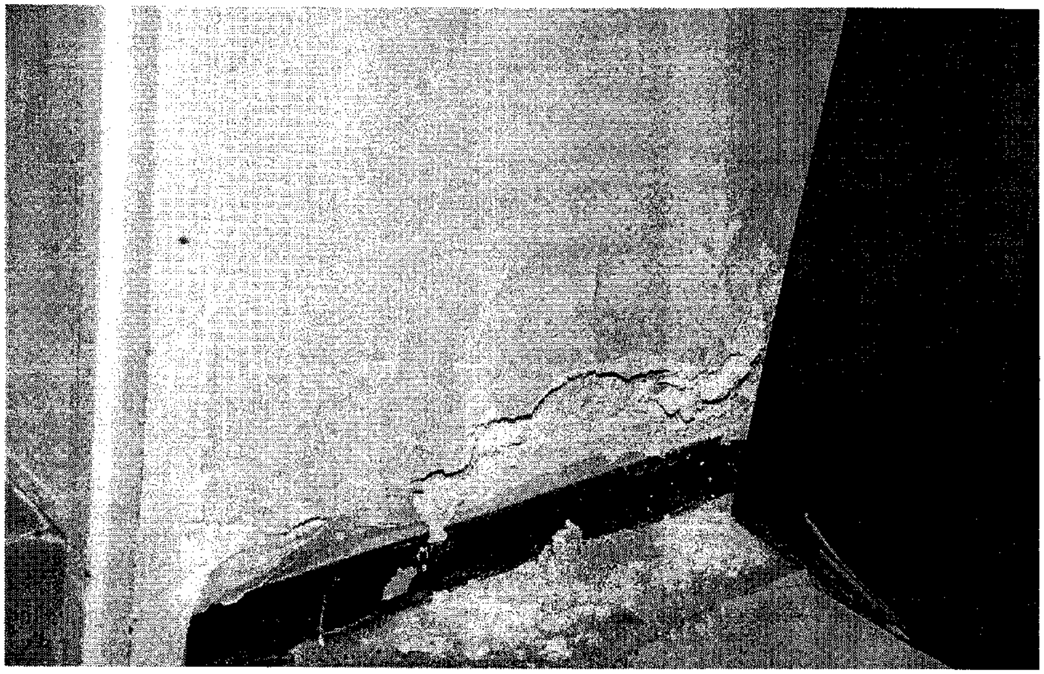
Picture 17 – Water-damaged interior partition



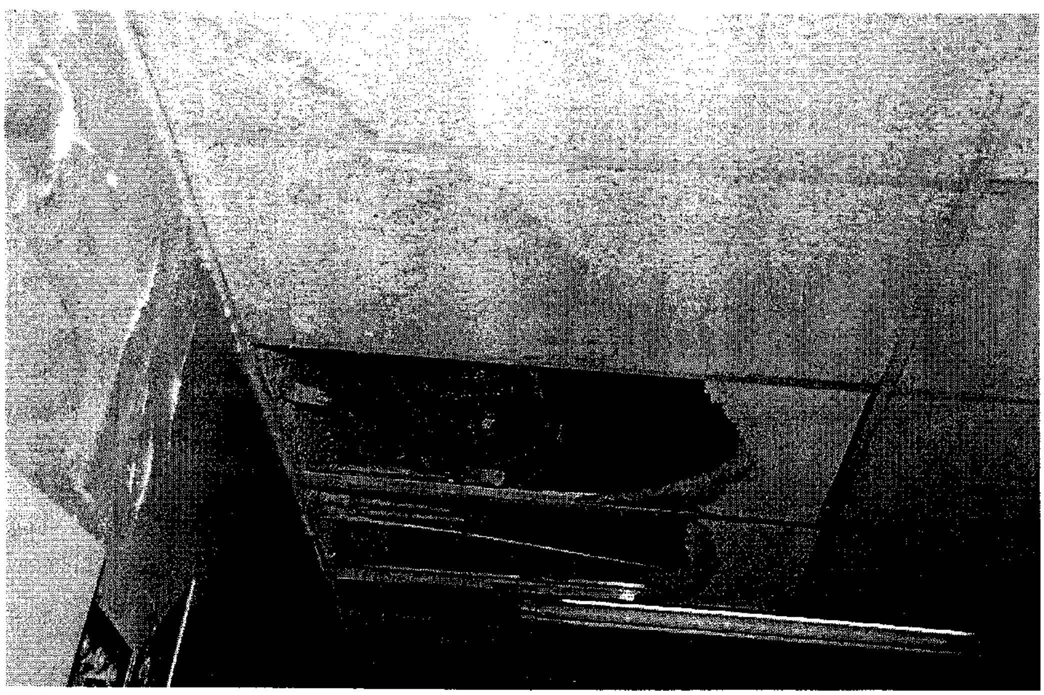
Picture 18 – Water-damaged second floor slab



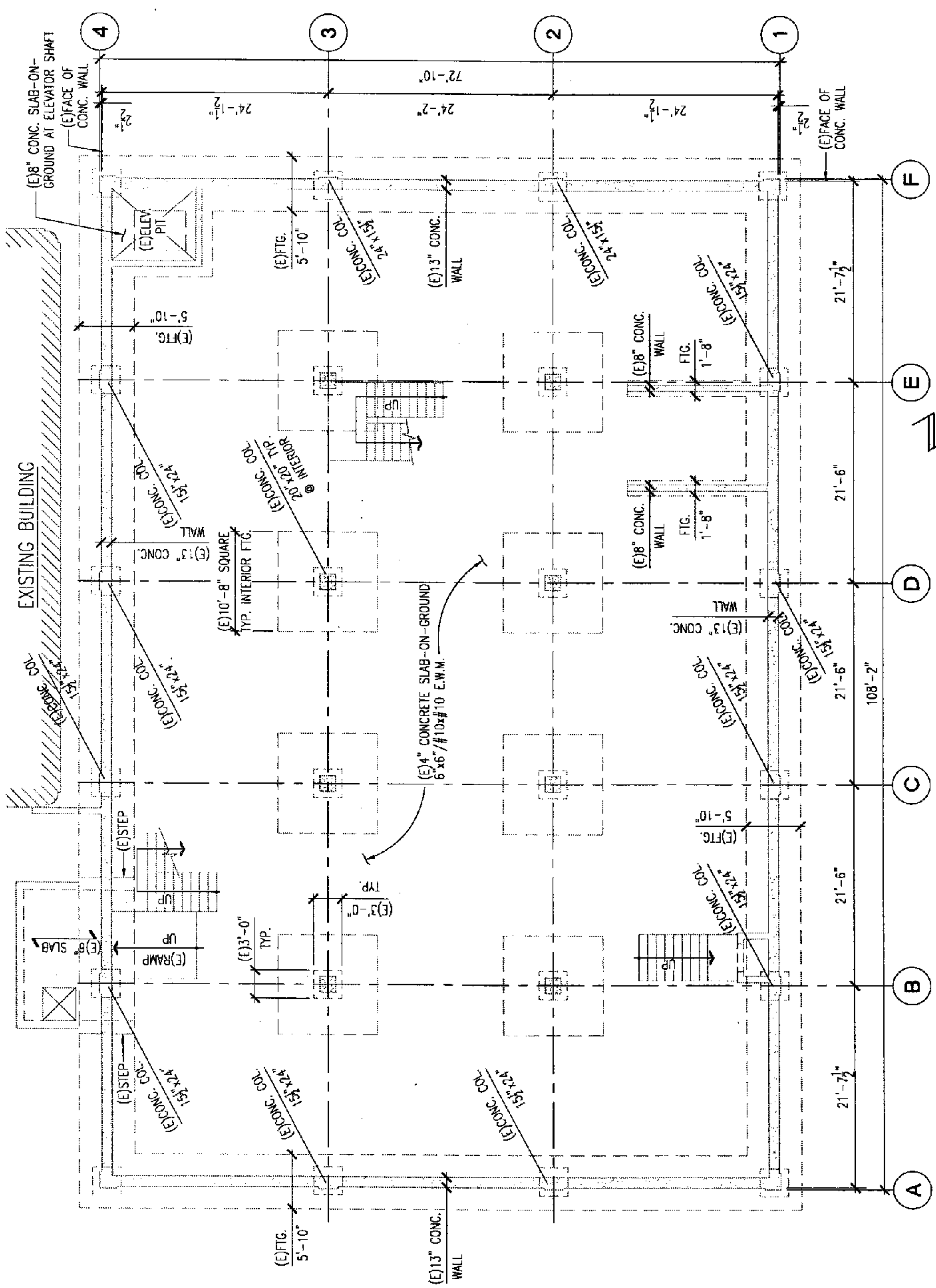
Picture 19 – Water-damaged interior partition



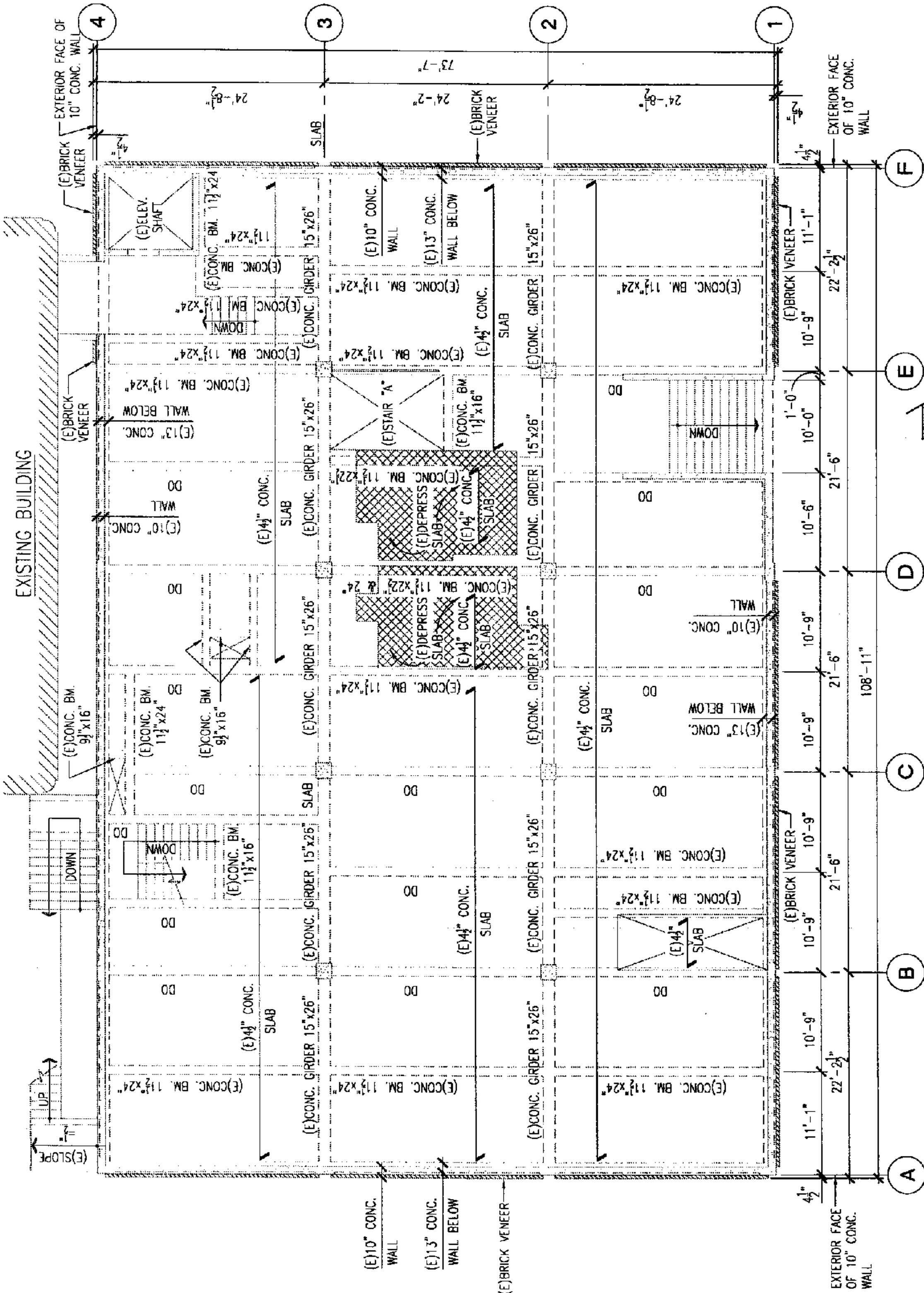
Picture 20 – Water-damaged interior partition



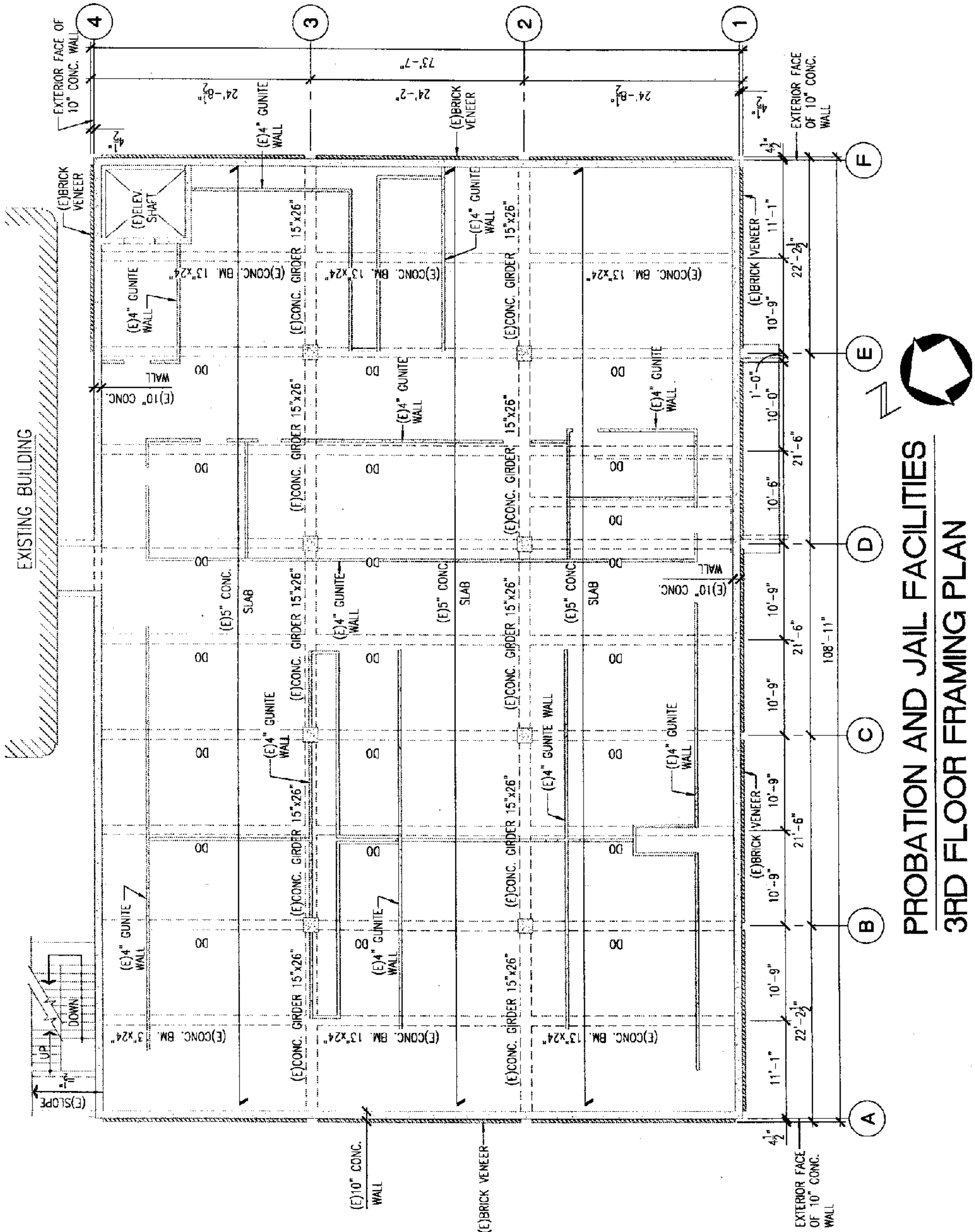
Picture 21 – Water-damaged ceiling and wall



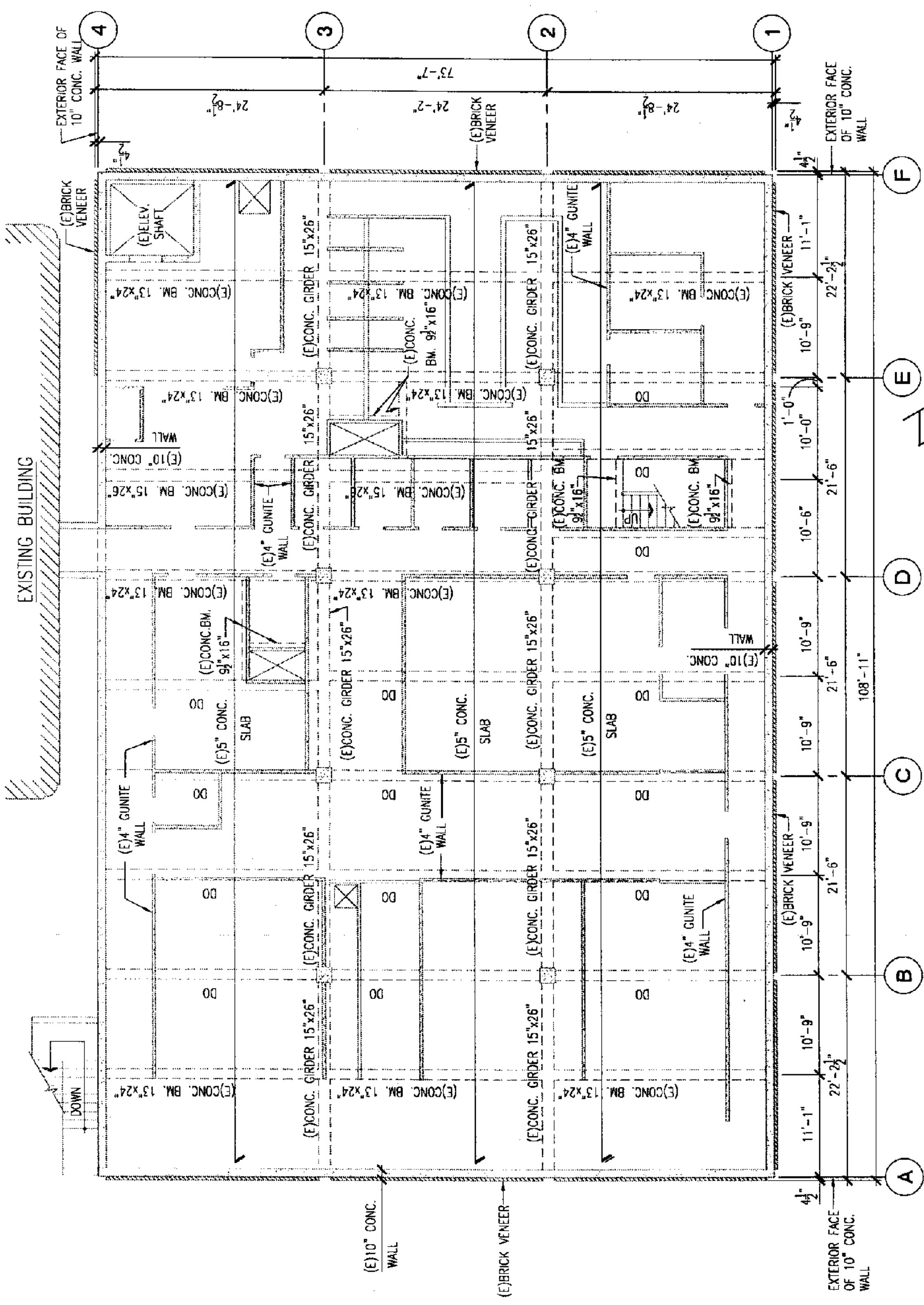
PROBATION AND JAIL FACILITIES
FOUNDATION PLAN
FIGURE 1



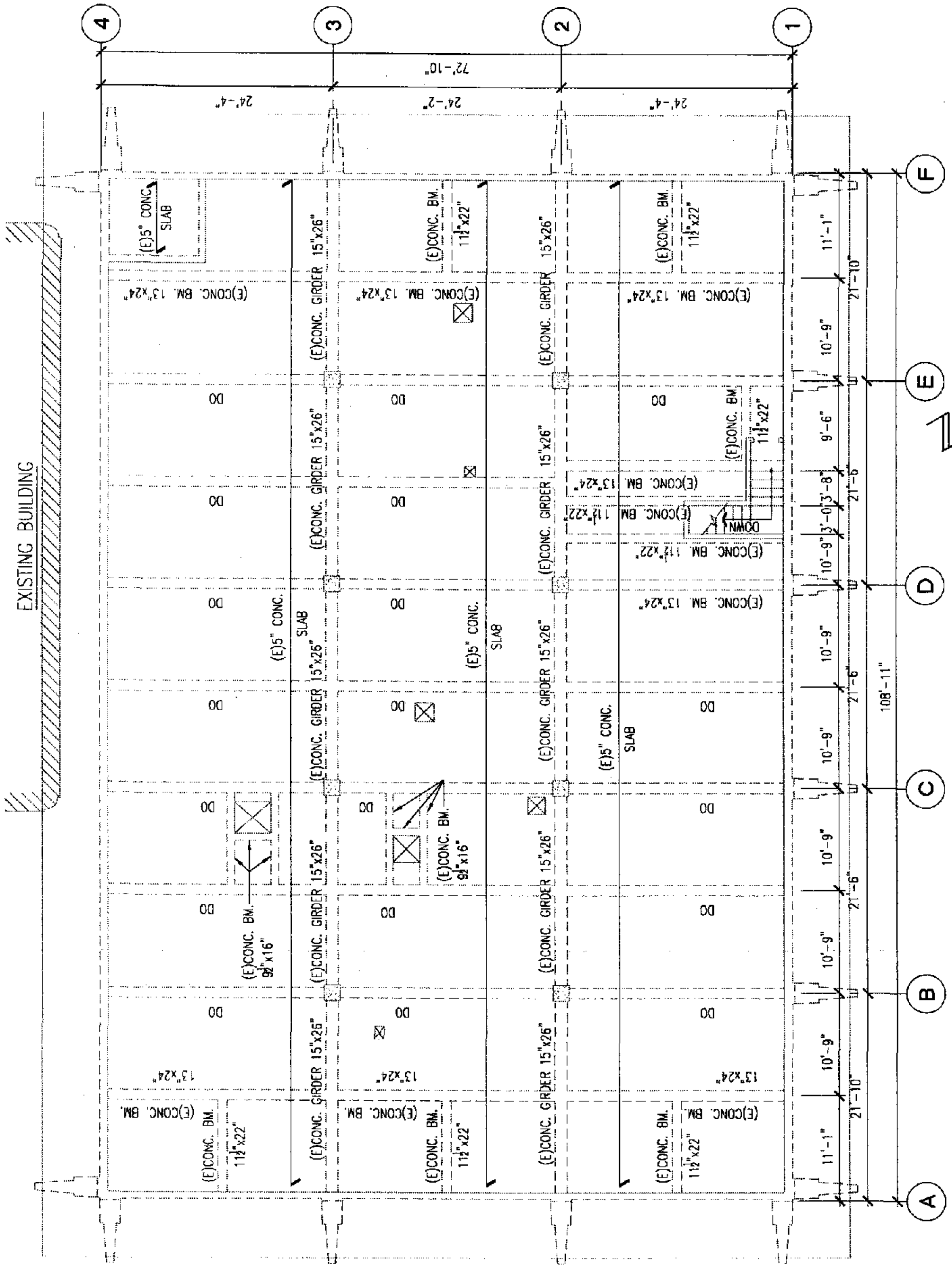
PROBATION AND JAIL FACILITIES
2ND FLOOR FRAMING PLAN
FIGURE 2



PROBATION AND JAIL FACILITIES
3RD FLOOR FRAMING PLAN
FIGURE 3



PROBATION AND JAIL FACILITIES
4TH FLOOR FRAMING PLAN
FIGURE 4



PROBATION AND JAIL FACILITIES
ROOF FRAMING PLAN
FIGURE 5

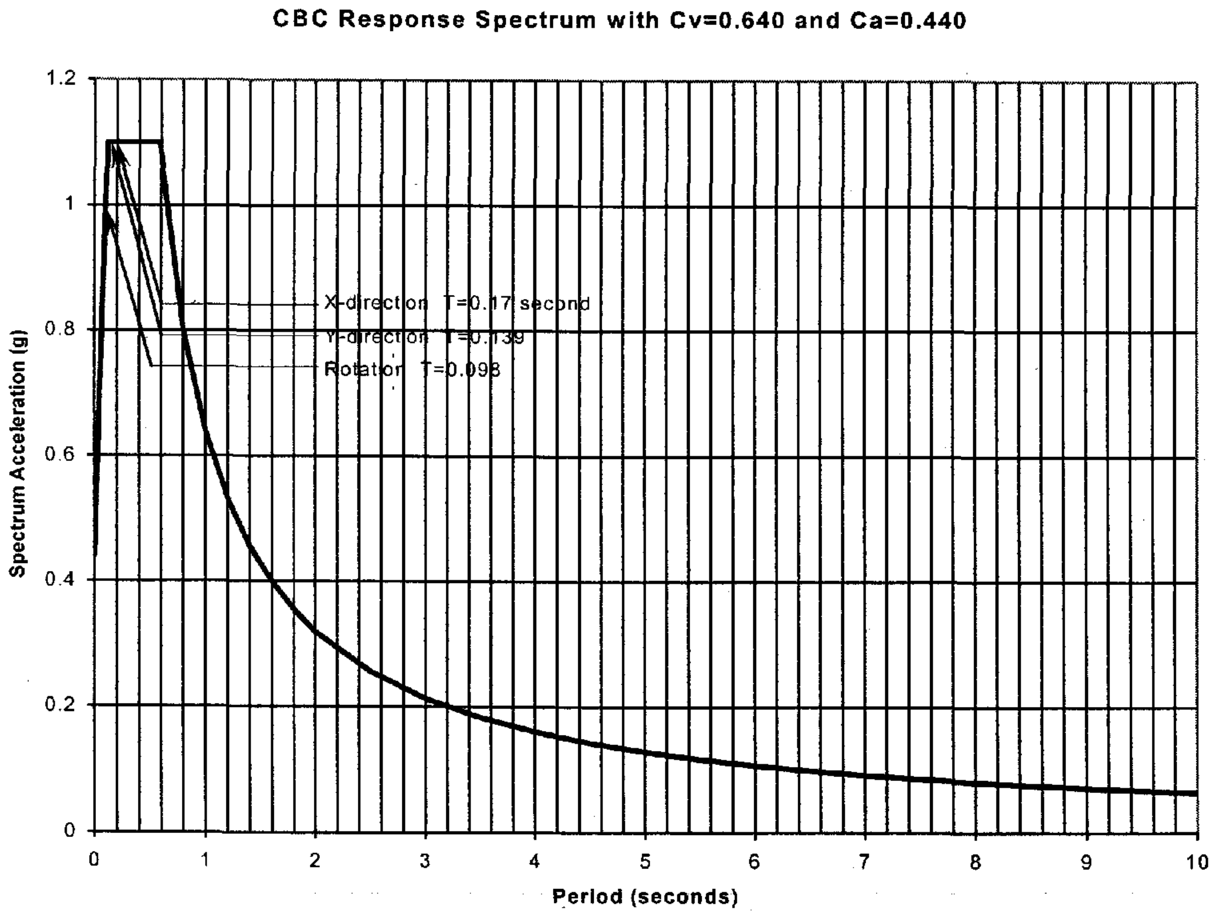
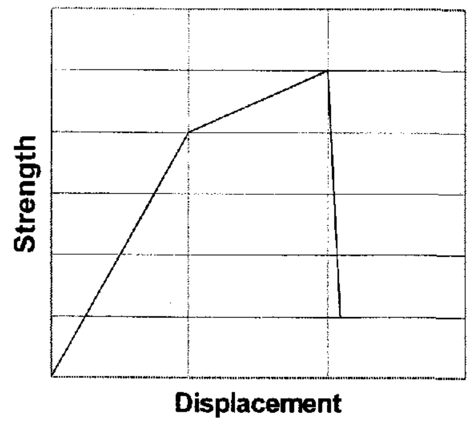
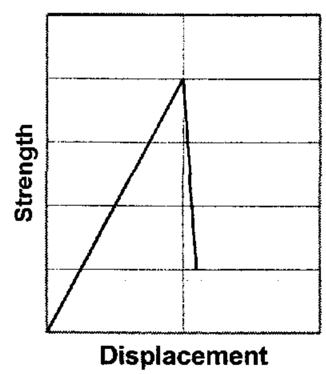


Figure 6 - Response Spectrum used for this Project



(a) Ductile Behavior



(b) Non-ductile Behavior

Figure 7 - Comparison of Material Behavior

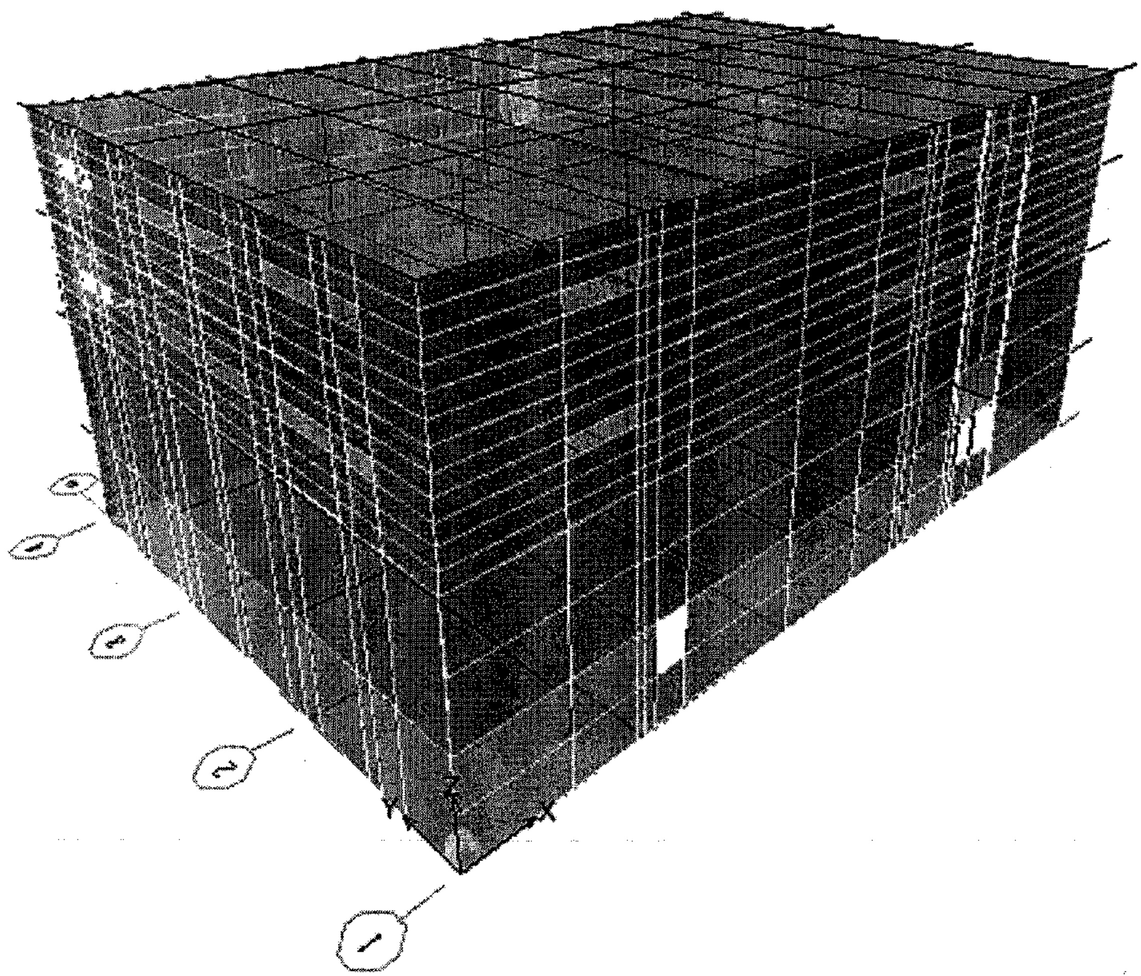
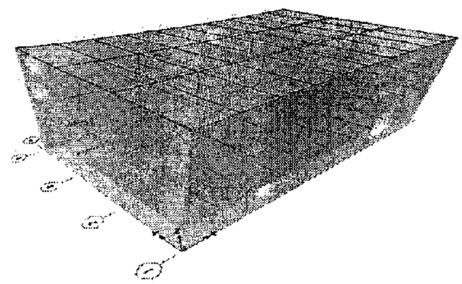
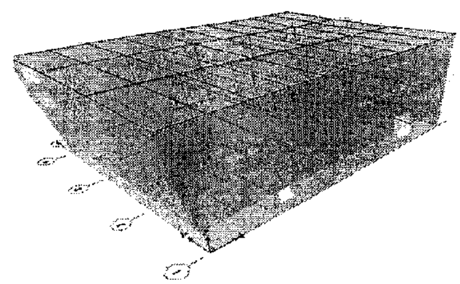


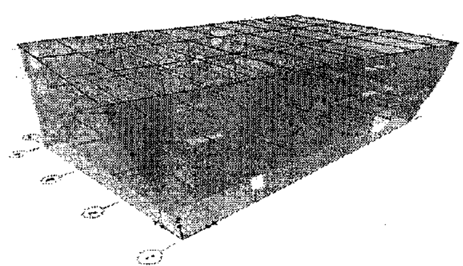
Figure 8 – three-Dimensional Computer Model



(a) First Mode - Y direction translation predominates



(b) Second mode - X-direction translation predominates



(c) Third mode - Rotation predominates

Figure 9 - Fundamental Mode Shapes