Analysis & Evaluation of Costa Linda Condominium Applying New Codes for to Determine Retrofit Requirements for Stability

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Abstract — *This study evaluate the capacity of a 10-story existing building designed in 1972 to determine its capacity to resist new loads according with present codes. A structural model was developed using a modern engineering software and existing new codes loads were applied. After entering all data into the model and running the software, columns, shear wall punching shear and story drift were evaluated according with the output and compared with present building codes. As expected, some structural elements presented inadequate design and possible failure. Recommendations to increase the elements' capacity for structure stability are provided.*

Key Terms — *Columns Resistance, Punching Shear, Shear Walls Capacity, Structure Stability.*

INTRODUCTION

Costa Linda Condominium is an existing building located at the seafront on Isla Verde Avenue, Carolina, PR. It consists of a 10-story apartment building. The construction drawings were approved by the Puerto Rico Planning Board on April, 1972. The construction code for PR at that time was the *Reglamento de Edificación #7* approved in 1967.

This building main structure system consists of a Flat Plate with 6-in posttensioned concrete slabs with no drop panel, 12 in x 24 in concrete columns, and shear walls on elevator and stair main shaft. Is important to mention that actual codes no longer permit this system [1].

A flat plate is a one- or two-way system usually utilizing a slab of uniform thickness and supported directly on columns (as shown in Figure 1) or load bearing walls. The principal feature of the flat plate floor is a uniform or near-uniform thickness with a flat soffit which requires only simple formwork and is easy to construct.

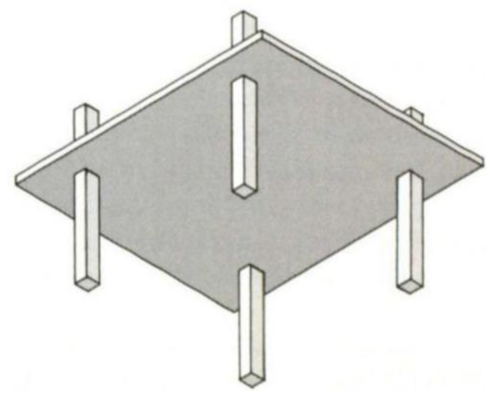

Figure 1
Flat Slab

The floor allows great flexibility for locating horizontal services above a suspended ceiling or in a bulkhead. The economical span of a flat plate for low to medium loads is usually limited by the need to control long-term deflection and may need to be sensibly pre-cambered or prestressed.

The advantages of the flat slab system are [2] [3]:

- Simple formwork and suitable for direct fix or sprayed ceiling.
- No beams—simplifying under-floor services.
- Minimum structural depth and reduced floor-to floor height.

The disadvantages of the flat slab system are:

- Punching shear limitation.
- Medium spans.
- Limited lateral load capacity as part of a moment frame.
- May need shear heads or shear reinforcement at the columns or larger columns for shear.
- Long-term deflection may be controlling factor
- May not be suitable for supporting brittle (masonry) partitions.

- May not be suitable for heavy loads.

Construction codes change an update over time due to better knowledge on structure behavior due to disaster like earthquake and other natural events. New investigation and studies also help to improved structure strength capacity with new technology application. Code revision provide new design parameter for better structural stability to resist natural event to preserve human life. In this project, Costa Linda Condominium will be evaluated according to current codes.

Research Objective

The objectives of this project are to identify the structural elements of Costa Linda Condominium that fail when analyzing the building using the applicable loads according to current building codes and to provide recommendations for improvements to increase its structural capacity and structural stability.

Methodology

This section presents in details the methodology used for this research.

Building Codes

The Building Codes and Design Manual used to create the model were the following:

- International Building Code 2009 [4]
- Minimum Design Load for Building and Other Structure ASCE 7-05 & 10 [5][6]
- Building Code Requirements for Structural Concrete - ACI 318-14 [7]

Computer Program

The structural system was modeled using ETABS 2016. Also, SAFE 2016 was used to perform punching shear analysis of the slabs. Both are commercial software created by Computers and Structures, Inc.

Model

Costa Linda building has a 90'-6" height, divided on 10 floors plus 2 additional levels for elevator shafts. There are 14 concrete columns with a resistance of 5,000 psi from floor 1 to floor 4; 4,000 psi from floors 5 to floor 6 and 3,000 psi from floors 7 to the top. There are posttensioned concrete slabs with 6 in of thickness and a resistance of 4,000 psi on each floor. There are shear walls with a thickness of 6 in on the elevator and stair area. Figure 2 presents the typical floor plan view, while Figure 3 shows the 3D Model Elevation created using ETABS.

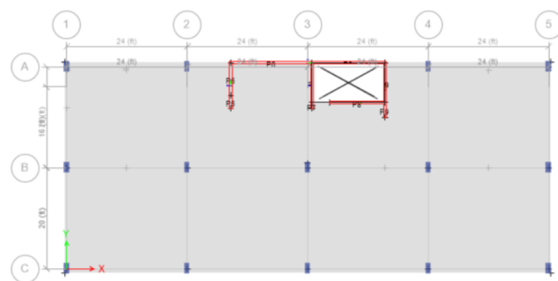

Figure 2
Plan View

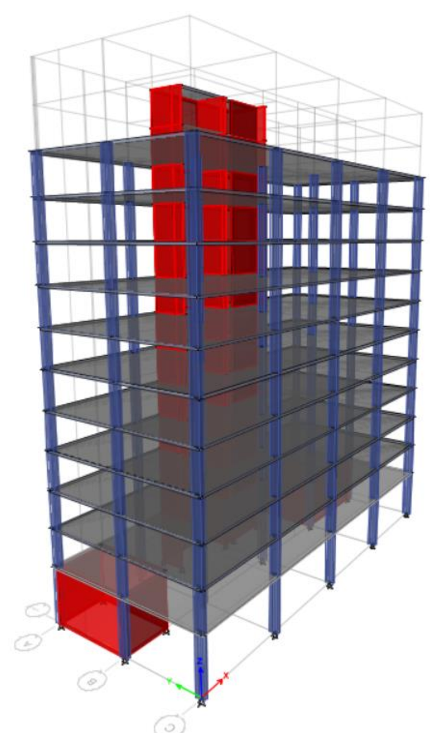

Figure 3
Elevation-3D Model

Dead load, live load, wind load parameters applied to the model are sumarized on Tables 1 to 2.

Table 1
Loads

Load Type	Level	Load
Dead Load	All Floors	50 psf
Live Load	All Floors	40 psf

Table 2
Wind Load Parameters

Wind Speed	145 mph
Exposure Type	B
Category	III
Importance Factor	1.15
Topographic Factor (K_{ZT})	1
Gust Factor (Gx)	1.6
Gust Factor (Gy)	1.52
Directional Factor	0.85
Winward Coefficient (Cpw)	0.8
Leeward Coefficeint (Cpl)	0.5
e1 Ratio	0.15
e2 Ratio	0.15

Table 3
Seismic Load Parameters

Occupation Category	III
Importance Factor:	1.25
$S_{S:}$	0.99
$S_{1:}$	0.39
Site Class:	D
Fa:	1.1
Fv:	1.24
$S_{DS:}$	0.73
$S_{D1:}$	0.32
R:	5
$T_{a:}$	0.67
$h_{n:}$	108.83
$C_{t:}$	0.02
x:	0.75
$T_{S:}$	0.44
$T_{L:}$	12
$C_{S:}$	0.120
Seismic Design Category	D

To determine the building natural frequency Equation 1 from ASCE [6] was used:

$$n_a = 43.5/n^{0.9} \qquad (1)$$

For the building of this project, the natural frequency is 0.69 Hertz. Since this values is less than 1, the structure is considered flexible. To determine Gust Factor parameters, Equation 2 from ASCE [6] was used:

$$G_f = 0.925\left(\frac{1+1.7I_{\bar{z}}\sqrt{g_Q^2Q^2+g_R^2R^2}}{1+1.7g_vI_{\bar{z}}}\right) \qquad (2)$$

Seismic load parameters applied to the model are sumarized on Table 3.

RESULTS

In this section, the output of the analysis is presented. It is divided on four mayor areas: column response, shear walls response, evaluation of slab punching shear and story drift analysis.

Columns

According with the design check, 9 concrete frames failed. Columns failure occurred on floors 1 2 and 3. See Figures the 4 to 6 for columns location.

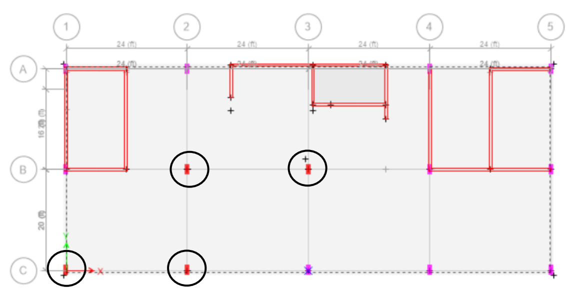

Figure 4
Columns Failure - Floor 1

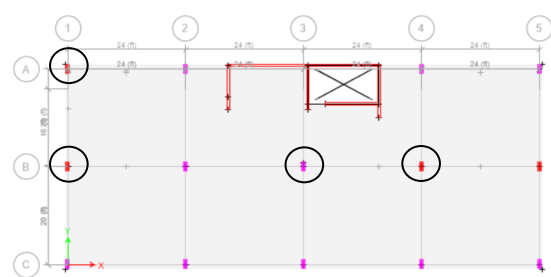

Figure 5
Columns Failure - Floor 2

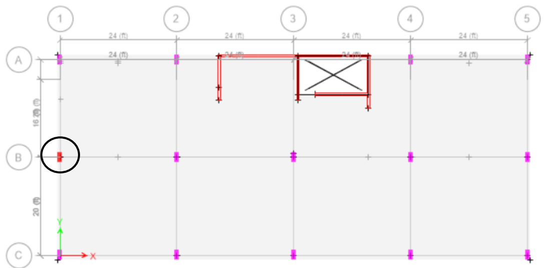

Figure 6
Columns Failure - Floor 3

For the evaluation of concrete section design for column C8, the output analysis from ETABS is summarized on Table 4.

Table 4
Column C8 Design Output

Column C8		
Design Pu	753.151	kip
Design Mu2	234.4842	kip-ft
Design Mu3	-62.43	kip-ft
Minimum M2	60.252	kip-ft
Minimum M3	82.85	kip-ft
Rebar Area	28.44 (O/S#2)	in2
Rebar %	9.88 (O/S#2)	%

Actual column rebar for this column consist on 12 No. 11 or a rebar area of 18.72 in^2. According to the design analysis, an additional reinforcement of 9.72 in^2 is required to resist the load combinations.

Walls

According to ETABS, some shear walls present overstress. Flexural design for wall on axis A at the 4th floor is summarized on Table 5.

Table 5
Wall P6 - Design Output

P6 Story 4					
Station	D/C	Flexural	Pu (kip)	Mu2 (kip-ft)	Mu3 (kip-ft)
Top	1.774	DWal12	-2338.54	-11.76	-35687.37
Bottom	1.862	DWal12	-2312.87	-42.53	-38039.9

The wall pier demand/capacity ratio is a factor that gives an indication of the stress condition of the wall with respect to the capacity of the wall. See Figure 7 [8] presents a graphical representation of this ratio. In this figure:

- If OL = OC (or D/C = 1), the point (P_f, M_{3f}) lies on the interaction curve and the wall pier is stressed to capacity.
- If OL < OC (or D/C < 1), the point (P_f, M_{3f}) lies within the interaction curve and the wall pier capacity is adequate.
- If OL > OC (or D/C > 1), the point (P_f, M_{3f}) lies outside of the interaction curve and the wall pier is overstressed.

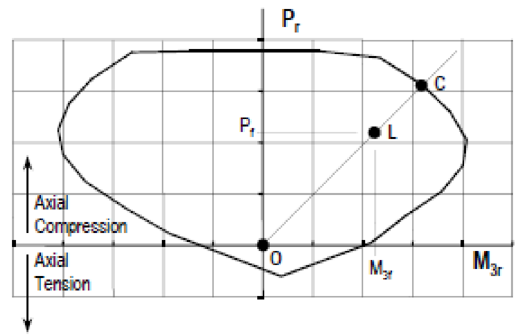

Figure 7
Two-Dimensional Wall Pier Demand/Capacity Ratio

The demand/capacity for this pier is between 1.774 and 1.86 indicating that the wall is overstressed to almost the double of its capacity. Additional reinforcement or increasing wall thickness is required.

Slab-Punching Shear

For punching shear analysis, a model was developed on SAFE 2016 structural software, as shown in Figure 8. The deformed shape for dead load only is as show on Figure 9. Punching shear analysis were developed and the results are shown on Figure 10. As shown on Figure 10, ratios are greater than 1, indicating low capacity for resist punching shear. Higher slab thickness is required.

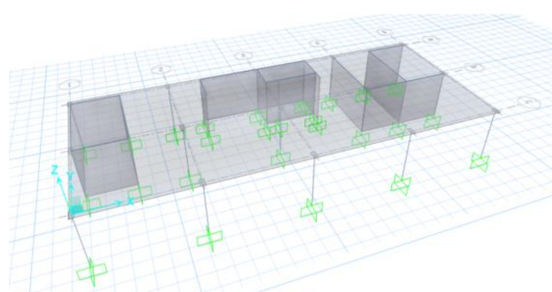

Figure 8
Slab Model on SAFE 2016

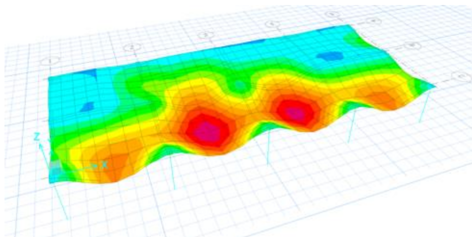

Figure 9
Deformed Shape – Dead Load

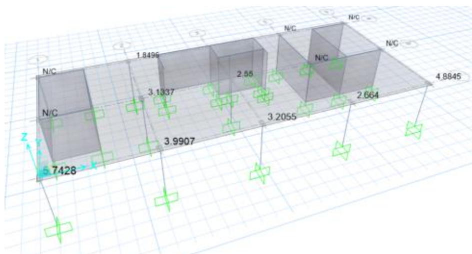

Figure 10
Punching Shear Capacity Ratios/Shear Reinforcement

Story Drift Limits

For story drift limits evaluation, the Allowable Story Drift from ASCE [6] were used according to FEMA [9]. Limits for risk category III and all other structures identification, the maximum drift limits is $0.015h_{sx}$, were h_{sx} is the height story x; this limit may be thought of as 1.5% of the story height. To determine the Magnified story drift, Equation 3 from ASCE 7 [6] was used.

$$\delta_M = \frac{C_d \delta_{max}}{I_e} \tag{3}$$

Table 6
Story Drift Determination- Seismic Load X Direction

X- direction — Story Drift (Δ) — Allowable story drift

Story	Displacement (in)	Story Drift (in)	Cd	I_E	Magnified Story Drift	Magnified Drift Ratio	Story Drift Limit	Check
11	19.667	2.586	5	1.25	10.344	0.100	0.015	Over Limit
10	17.081	1.2915	5	1.25	5.166	0.050	0.015	Over Limit
9	14.498	1.2705	5	1.25	5.082	0.049	0.015	Over Limit
8	11.957	1.221	5	1.25	4.884	0.047	0.015	Over Limit
7	9.515	1.1185	5	1.25	4.474	0.043	0.015	Over Limit
6	7.278	1.018	5	1.25	4.072	0.040	0.015	Over Limit
5	5.242	0.923	5	1.25	3.692	0.036	0.015	Over Limit
4	3.396	0.772	5	1.25	3.088	0.030	0.015	Over Limit
3	1.852	0.584	5	1.25	2.336	0.023	0.015	Over Limit
2	0.684	0.3225	5	1.25	1.29	0.013	0.015	OK
1	0.039	0.0195	5	1.25	0.078	0.000	0.015	OK
Base	0							

Y - direction — Story Drift (Δ) — Allowable story drift

Story	Displacement (in)	Story Drift (in)	Cd	I_E	Magnified Story Drift	Magnified Drift Ratio	Story Drift Limit	Check
11	22.98	3.05	5	1.25	12.2	0.118	0.015	Over Limit
10	19.93	1.5205	5	1.25	6.082	0.059	0.015	Over Limit
9	16.889	1.4945	5	1.25	5.978	0.058	0.015	Over Limit
8	13.9	1.4325	5	1.25	5.73	0.056	0.015	Over Limit
7	11.035	1.3085	5	1.25	5.234	0.051	0.015	Over Limit
6	8.418	1.1895	5	1.25	4.758	0.046	0.015	Over Limit
5	6.039	1.0775	5	1.25	4.31	0.042	0.015	Over Limit
4	3.884	0.899	5	1.25	3.596	0.035	0.015	Over Limit
3	2.086	0.6735	5	1.25	2.694	0.026	0.015	Over Limit
2	0.739	0.3625	5	1.25	1.45	0.014	0.015	OK
1	0.014	0.007	5	1.25	0.028	0.000	0.015	OK
Base	0							

Table 7
Story Drift Determination- Seismic Load Y Direction

X- direction — Story Drift (Δ) — Allowable story drift

Story	Displacement (in)	Story Drift (in)	Cd	I_E	Magnified Story Drift	Magnified Drift Ratio	Story Drift Limit	Check
11	1.311	0.198	5	1.25	0.792	0.008	0.015	OK
10	1.113	0.096	5	1.25	0.384	0.004	0.015	OK
9	0.921	0.0905	5	1.25	0.362	0.004	0.015	OK
8	0.74	0.083	5	1.25	0.332	0.003	0.015	OK
7	0.574	0.072	5	1.25	0.288	0.003	0.015	OK
6	0.43	0.064	5	1.25	0.256	0.002	0.015	OK
5	0.302	0.0565	5	1.25	0.226	0.002	0.015	OK
4	0.189	0.0468	5	1.25	0.1872	0.002	0.015	OK
3	0.0954	0.0346	5	1.25	0.1384	0.001	0.015	OK
2	0.0262	0.0081	5	1.25	0.0324	0.000	0.015	OK
1	0.01	0.005	5	1.25	0.02	0.000	0.015	OK
Base	0							

Y - direction — Story Drift (Δ) — Allowable story drift

Story	Displacement (in)	Story Drift (in)	Cd	I_E	Magnified Story Drift	Magnified Drift Ratio	Story Drift Limit	Check
11	19.667	13.392	5	1.25	53.568	0.520	0.015	Over Limit
10	6.275	0.424	5	1.25	1.696	0.016	0.015	Over Limit
9	5.427	0.4175	5	1.25	1.67	0.016	0.015	Over Limit
8	4.592	0.4055	5	1.25	1.622	0.016	0.015	Over Limit
7	3.781	0.385	5	1.25	1.54	0.015	0.015	OK
6	3.011	0.3525	5	1.25	1.41	0.014	0.015	OK
5	2.306	0.319	5	1.25	1.276	0.012	0.015	OK
4	1.668	0.2845	5	1.25	1.138	0.011	0.015	OK
3	1.099	0.2385	5	1.25	0.954	0.009	0.015	OK
2	0.622	0.186	5	1.25	0.744	0.007	0.015	OK
1	0.25	0.125	5	1.25	0.5	0.003	0.015	OK
Base	0							

Seismic response on x and y direction for maximum displacement is sumarized on Tables 6 and 7.

Results indicate that critical case is on seismic load on the x direction. The story drifts from the 2nd to the 11th floor are over the 1.5% limit. To reduce story drift, building lateral resistance should be evaluated; the shear wall on elevator and stair case is not enough.

Conclusion and Recommendation

In general, the building's structural performance applying present codes demonstrate a lack of structural stability. Is important to mention that some columns and walls failed the analysis for some load combination. Punching Shear and drift limits area exceeded on basic loads. Structural analysis validate no permitting any more the flat slab structural system.

Following are recommendations for increasing structural capacity and lateral stability. Is important to mention that only some general recommendations are given. Final design and specific details are not part of the scope of this project.

Columns

Increasing column strength capacity can be achieved using various methods. One of them is to wrap columns with carbon fiber. Carbon fiber increase ductility and increase energy absorption capacity. Other alternatives are the installation of steel jacket around columns or reinforce columns with concrete jacket adding additional rebar to increase column strength capacity. These methods are shown in Figure 11.

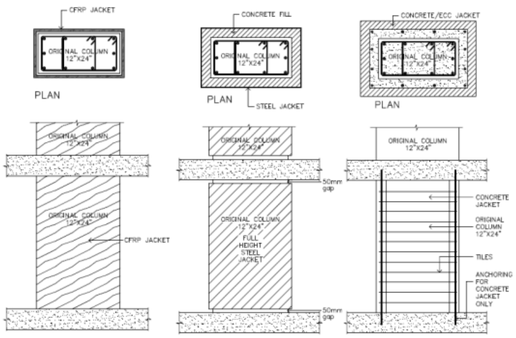

Figure 12
Increasing Column Capacity Methods

Walls

Increasing wall capacity can be achieved by wrapping them with carbon fiber or installing steel plates around. Examples of these are shown in Figure 12 & 13.

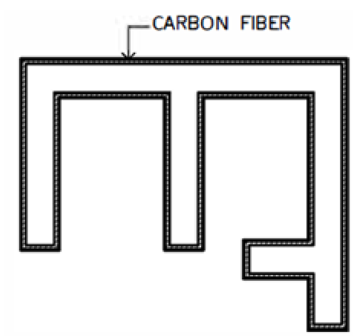

Figure 12
Increasing Wall Capacity Methods Carbon Fiber

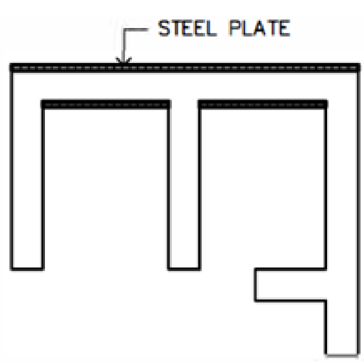

Figure 13
Increasing Wall Capacity Methods Steel Plate

Slab Punching Shear

One of the disadvantage of flat slab is the low capacity to resist punching shear stresses. Increase strengthening of flat slab can be provided throughout various methods. Some of the alternatives are the following:

- Shear bolt reinforcement – Bolts are anchored at each face of the slab surface of the column, as shown in Figure 14. Bolts intercept punching cone, restricting its development as tension member.

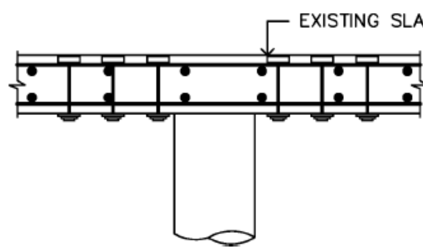

Figure 14
Shear Bolt Reinforcement

- Post-installed shear reinforcement – Consists of bars installed into inclined holes of 45 degrees and bonded with high-performance epoxy, as shown in Figure 15.

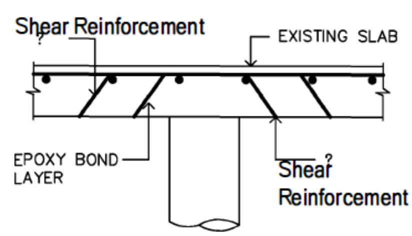

Figure 15
Post-Installed Shear Reinforcement

- Widening column – Consists of increasing colums dimension to reduce punshing shear, as shown in Figure 16.

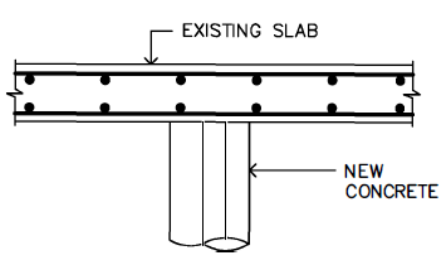

Figure 16
Widenng Column

Story Drift

As shown on Tables 6 and 7, story drift limits are exceeded. To reduce building displacement, additional lateral resistance should be added. To achieve this, additional walls could be added. Walls can be designed on concrete or use a steel bracing system (as seen in Figure 17). Architectural considerations should be evaluated for the aesthetics of the original structure.

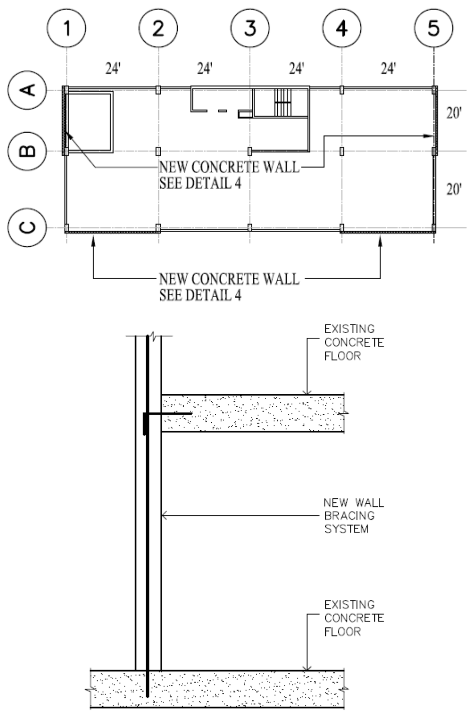

Figure 17
Concrete Walls to Increase Lateral Resistance

Closing

Implementation alternatives for retrofitting on this type of building, were people are presently living, is one of the biggest challenges to consider. Another important issue to be considered is that the architectural finishes can't affect the architectural aesthetics of the building. Construction procedure and cost should be evaluated as part of the design. Implemented this changes to increase structural capacity involve great effort and possible high cost.

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