Modeling of a Residential Building to Compare the Structural Performance of Structural Concrete Insulated Panels vs. Reinforced Concrete

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***Abstract* ––** *This paper presents the modeling of a 4 story residential building currently under development in San Juan, P.R., to compare the structural performance of structural concrete insulated panels (SCIP) vs. reinforced concrete (RC). The core walls on the building consist of bearing wall system also acting as shear walls. Models analysis and design were performed according to the IBC 2009, ASCE7-05 and ACI318-08 and using ETABS, SAFE and PILOTYN7 computer programs. The objectives of this work were to find if the models with structural concrete insulated panels (SCIP) and reinforced concrete (RC) were capable to perform under the codes limits and compare their results. The main conclusion of this work was to find that both the RC and SCIP models performed adequately under gravity and seismic loads, assuming that SCIP recommended strength models are appropriate.*

Key Terms –– *Deflection, Design, Drift, SCIP Panels.*

INTRODUCTION

This paper presents the modeling of a 4 story residential building currently under development in San Juan, P.R., to compare the structural performance of structural concrete insulated panels, from now on referred to as (SCIP) vs. reinforced concrete, from now on referred to as (RC).

The manufacturers of the SCIP compare their system with conventional structural systems, such as RC, stating the following:

- Structural Capacity**:** Because SCIP is a highly resistant mortar-coated tridimensional structure, the result is a very light and compact section wall, with a bearing capacity similar to, and in some cases, higher than the one obtained by the use of conventional systems.
- Safety**:** Its performance in earthquakes is excellent because inertia forces are proportional to the mass and, as a result, to the weight of the building. SCIP lightness makes it an excellent alternative to build optimal and safe structures with the capacity to dissipate energy.

  The shear stress supported by SCIP is much higher than the one supported by a conventional system.

With this in mind, the work started by analyzing gravity, wind and seismic loads applying the ones that governs to both model buildings under the requirements of the IBC 2009, ASCE7-05 and ACI318-08. A three-dimensional analysis of the building was performed in both the N-S (Y) and E-W (X) direction for seismic forces using ETABS and SAFE programs. In the models, rigid diaphragms were assigned to each floor level. P-delta effects were also considered in the analysis.

After that, the following verifications and designs were performed on the models:

- Lateral Drift on X and Y direction.
- Deflection on floors.
- Floor design for gravity loads.
- Shear wall design for seismic lateral loads.

Finally SCIP and RC model results were compared and conclusions were developed.

DESCRIPTION OF MODEL BUILDING AND SCIP SYSTEM

The selected building is one of twenty-one (21) buildings of the multifamily housing complex

Gardens of Monte Carlo. The complex is located at the Monte Carlo Ave. B Street, Bo. Sabana Llana, San Juan; Puerto Rico (see Figure 1). The proposed structure is a four story building of 65’-4” by 35’-6” of plan dimensions, see Figure 2. The building elevation is 32’-0”, each storey of 8’-0”. There are two (2) housing units on each floor. Each housing unit consists of three (3) bedrooms, one (1) bathroom, living room, kitchen, dining room and a balcony area. Housing units from the second floor to the fourth will have access via a stairway, located at the center of the building.

Figure 1
Site Map

The core walls on the building consist of bearing wall system. Bearing walls provide support for all or most of the gravity loads. Resistance to lateral loads is provided by the same bearing walls acting as shear walls. The RC building walls are 6 in. thick in the X direction and 5 in. thick in the Y direction. The floors are 6 in. thick.

SCIP System

The SCIP panel for the support walls, not including the mortar, is 4 ft. wide and 8 ft. high; its 3.75 in. thickness, includes the double mesh. The core of the panel is a corrugated plate of expanded polystyrene (foam), reinforced by an electro-welded mesh placed on each face and connected to each other by galvanized wires which penetrate through the foam and are welded to each mesh. The electro-welded mesh consists of smooth wires of galvanized steel with 0.1378 in. of diameter in the longitudinal direction and 0.0991 in. in the transversal direction. The wires are 2.56 in. apart each way. The links are also smooth galvanized steel, 0.1181 in. diameter. After applying a layer of shotcrete 1.5 in. thick on each face of the wall panel, the panel itself becomes 6 in. thick and acquires a weigh of 37.5 psf.

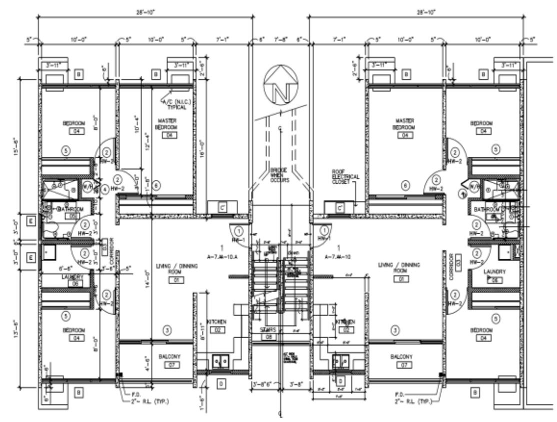

Figure 2
Typical Floor Plan

The floor panels are similar to the ones described for the walls, with the following differences: the base of the floor panel receives a layer of shotcrete 3000 psi 1 in. thick, while the upper face receives a layer of simple 3000 psi concrete, 2.5 in. thick, therefore forming a floor that is 6.5 in. thick, with an average weight of 43.75 psf, not including finishing materials or overload.

LOAD ANALYSIS

This section presents the gravity, wind and seismic loads analysis.

Gravity Loads

The following gravity loads are applicable for both buildings:

- Live loads:
  Roof 40 psf per new 2011 PR CODE
  Floor 40 psf per IBC
- Dead loads for RC building:
  Roof self weight is 75 psf
  Floor self weight is 75 psf
- Dead loads for SCIP building:
  Roof self weight is 43.75 psf
  Floor self weight is 43.75 psf

Wind Loads

According to IBC 1609.1.1, wind loads shall be determined in accordance with Chapter 6 of ASCE 7 [1]. The analytical procedure (Method 2) of ASCE 6.5 is used to determine the wind forces on the building in the N-S and E-W directions.

Basic wind speed V, is equal to 145 mph for Puerto Rico.

The wind directionality factor *Kd*, may be used as 1 for main wind-force-resisting systems on concrete buildings.

Importance factor *Iw*, is equal to 1.0 for Category II occupancy according to Table 6-1 of ASCE7.

Velocity pressure exposure coefficient, *Kz* can be determined from ASCE7 Table 6-3 by linear interpolation. Values of *Kz* or *Kh* are summarized in Table 1 at the various story heights for the model building.

Table 1
Velocity Pressure Exposure Coefficient

Level	Height *z*, ft.	*Kz=Kh*
4	32	0.712
3	24	0.652
2	16	0.58
1	8	0.57

Topographic factor *Kzt*, is to be determined in accordance with ASCE7 6.5.7. The building is situated on level ground and not on a hill, ridge, or escarpment, *Kzt* is equal to 1.

Gust effect factors *G* and *Gf*, depends on whether a building is Rigid or flexible. A rigid building has a fundamental natural frequency, n_1 greater than or equal to 1 Hz.

An approximate fundamental period, *Ta* is determined using Eq.12.8-7 of ASCE7. The natural frequency is computed by taking the inverse of the period (1).

$$Ta = C_T (hn)^{3/4} \qquad (1)$$

Where, C_T is the building period coefficient and for these types of building system is 0.02.

Then, $Ta = 0.02(32')^{3/4} = 0.2691$sec. And n_1=1/0.2691=3.7161 Hz, since $n_1 > 1$ Hz the building is rigid and *G* is taken as 0.85.

This building is Partially Enclosed because it complies with both of the following conditions:

- Ao > 1.10Aoi.
- Ao > 4 sq ft or > 0.01Ag, whichever is smaller, and Aoi/Agi < 0.20.

Where, Ao are open wall areas. Ag are gross area of wall. Aoi is the sum of the areas of openings in the building envelope (walls and roof) not including Ao in sq ft. Finally, Agi is the sum of the gross surface areas of the building envelope (walls and roof) not including Ag in sq ft.

The critical direction of the building is N-S. In this direction Ao is 1406.4 sq ft, Ag is 2077 sq ft., Aoi is 1238 sq ft. and Agi is 6276 sq ft.

Then, 1406.4 > 1.10(1238) =1362 is adequate. Also, 1406.4 > 4 sq ft or > 20, whichever is smaller, and 1238/6275= 0.19 < 0.20 is adequate.

The External Pressure Coefficients *Cp*, for main wind force resisting systems are taken from Figure 6-6 of ASCE7 for this building.

For wind in the N-S direction:

- Windward wall: C*p* = 0.8
- Leeward wall: C*p* = -0.5

For wind in the E-W direction:

- Windward wall: C*p* = 0.8
- Leeward wall: C*p* = -0.334

Velocity pressure *qz*, at height *z* is determined from Eq. 6-15 in ASCE7.

$$qz = 0.00256\, Kz\, Kzt\, Kd\, V^2\, I \qquad (2)$$

Where, all terms have been defined previously. Table 2 contains a summary of the velocity pressures for the model building.

The design pressure *p*, on the main wind-force-resisting systems of a partially enclosed building are determined in accordance with (3).

$$p=qGCp \qquad (3)$$

Tables 3 and 4 contain summaries of design pressures and forces, respectively, for wind in the N-S direction. It has been assumed that the design wind pressure is constant over the tributary height of the floor level. Tables 5 and 6 contain the pressures and forces for wind in the E-W direction, respectively.

Table 2
Velocity Pressure *qz*(V=145*mph*)

Level	Height *z*, ft.	*Kz=Kh*	*qz(psf)*(2)
4	32	0.712	38.32
3	24	0.652	35.1
2	16	0.58	31.2
1	8	0.57	30.7

Table 3
Design Pressure (N-S)

	Level	Height *z*, ft.	*qz* *(psf)* (2)	*G*	*Cp*	*qzGCp* *(psf)*
Wind-ward	4	32	38.32	0.85	0.8	26
	3	24	35.1	0.85	0.8	23.9
	2	16	31.2	0.85	0.8	21.2
	1	8	30.7	0.85	0.8	20.1
Lee-ward	n/a	All	38.32	0.85	-.5	-16.3

For the SCIP building the wind loads are the same as the ones calculated for the RC building.

Seismic Loads

Seismic loads are determined by chapters 11 and 12 of ASCE7 [6], referred there by the IBC. For San Juan, Puerto Rico Ss = 0.9 and S_1 = 0.31. The importance factor, I is 1 for occupancy II.

The maximum considered earthquake spectral response accelerations for short periods S_{MS} and at 1 second period S_{M1} are determined from (4) and (5), respectively:

$$S_{MS} = Fa\ Ss = 1.14(0.9) = 1.026\text{g} \quad (4)$$

$$S_{M1} = Fv\ S_1 = 1.78(0.31) = 0.55\text{g} \quad (5)$$

Where, Fa and Fv are contained in ASCE7 Table 11.4-1 and Table 11.4-2, respectively. Once S_{MS} and S_{M1} have been determined, S_{DS} and S_{D1} are computed from (6) and (7):

$$S_{DS} = 2/3\ S_{MS} = 2/3\ (1.026) = 0.68\text{g} \quad (6)$$

$$S_{D1} = 2/3\ S_{M1} = 2/3\ (0.55) = 0.37\text{g} \quad (7)$$

Table 4
Design Force (N-S)

Level	Height *h, ft.* 1	*qzwGCp* *(psf)* 2	*qzlGCp* *(psf)* 3	*L* *ft.* 4	*V* *(Kips)* 5
4	8	26	-16.3	64.9	21.9
3	8	23.9	-16.3	64.9	20.9
2	8	21.2	-16.3	64.9	19.5
1	8	20.1	-16.3	64.9	18.9

Force, 5= (2-3) x 1 x 4/1000 Total 81.2

Table 5
Design Pressure (E-W)

	Level	Height *z*, ft.	*qz* *(psf)* (2)	*G*	*Cp*	*qzGCp* *(psf)*
Wind-ward	4	32	38.32	0.85	0.8	26
	3	24	35.1	0.85	0.8	23.9
	2	16	31.2	0.85	0.8	21.2
	1	8	30.7	0.85	0.8	20.1
Lee-ward	n/a	All	38.32	0.85	-.3	-10.9

Table 6
Design Force (E-W)

Level	Height *h, ft.* 1	*qzwGCp* *(psf)* 2	*qzlGCp* *(psf)* 3	*L* *ft.* 4	*V* *(Kips)* 5
4	8	26	-10.9	35.5	10.5
3	8	23.9	-10.9	35.5	9.9
2	8	21.2	-10.9	35.5	9.1
1	8	20.1	-10.9	35.5	8.8

Force, 5= (2-3) x 1 x 4/1000 Total 38.3

Once these have been computed the seismic design category is determined with Tables 11.6-1 and 11.6-2 of ASCE7. For this building the design category is D.

The seismic base shear, V is computed from (8):

$$V = CsW \quad (8)$$

Where, *Cs* is the seismic response coefficient determined in accordance with ASCE7 12.8 and *W* is the effective weight of the structure. For the member sizes and above dead load, *W* = 979 Kips.

In both directions, a bearing wall system with special reinforced concrete shear walls is utilized, which is permitted for structures assigned to category D with a height less than or equal to 160ft. The response modification coefficient, *R* is 5 and the deflection amplification factor, *Cd* is 5 both of them are taken from ASCE7 Table 12.2-1.

The approximate period, *Ta* was previously computed with (1). The period for the model building is 0.27 sec.

The seismic response coefficient, *Cs* is determined from (9):

$$C_s = \frac{S_{D1}}{\left(\frac{R}{I}\right)\times T} = \frac{0.37}{\left(\frac{5}{1}\right)\times 0.27} = 0.2741 \quad (9)$$

The value of *Cs* needs not to be more than results from (10):

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I}\right)} = \frac{0.68}{\left(\frac{5}{1}\right)} = 0.136 \quad (10)$$

Also, *Cs* must not be less than (11):

$$Cs = 0.044\, S_{DS}\, I = 0.044 \times 0.68 \times 1 = 0.029 \quad (11)$$

Then, *Cs* is 0.136.

The seismic base shear for the RC system is *V*= (0.136) 979 = 133 Kips. The base shear is the same in both directions.

The vertical distribution of the seismic forces is compute from (12) and (13).

$$Fx = Cvx\, V \quad (12)$$

$$Cvx = \frac{Wxhx}{\sum_{i=1}^{n} Wihi} \quad (13)$$

The lateral load forces per floor are contained in Table 7.

Seismic lateral load forces for the SCIP building are calculated in the same form as previous calculations except for the effective weight of the structure. For the member sizes and above dead load, *W* = 573 Kips. Then seismic base shear is *V*= (0.136) 573 = 78 Kips. The base shear is the same in both directions. The lateral load forces per floor are contained in Table 8.

Then for the analysis and design of both model buildings, gravity and seismic load are used. Seismic loads govern over wind for the RC and SCIP building.

Table 7
Seismic Lateral Load Forces RC

Level	Height *hx*, ft.	*Wx Kip*	*Wxhx Kip-ft*	*Fx Kip*
4	32	205	6560	46
3	24	258	6192	43.5
2	16	258	4128	29
1	8	258	2064	14.5
SUM		979	18944	133

LOAD COMBINATIONS

The following load combinations are applicable:

- 1.4D
- 1.2D + 1.6L + 0.5Lf
- 1.2D + 1.6Lf + 0.5L
- 1.2D + E + 0.5L
- 0.9D + E

Table 8
Seismic Lateral Load Forces SCIP

Level	Height *hx*, ft.	*Wx Kip*	*Wxhx Kip-ft*	*Fx Kip*
4	32	120	3840	27
3	24	151	3624	25.5
2	16	151	2416	17
1	8	151	1208	8.5
SUM		573	11088	78

The seismic load effect E, which is the combined effect of horizontal and vertical earthquake induced forces need to be taken into account in load combination from (14):

$$E = \rho Q_E \pm 0.2 S_{DS} D \quad (14)$$

Where, Q_E is effect of horizontal seismic forces. Rho ρ, is redundancy coefficient equal to 1.3 determined as per ASCE7 12.3.4.2. Substituting S_{DS} = 0.68 and ρ = 1.3 into (14) and then substituting (14) into load combinations with E results in the following:

- 1.33D + 0.5L + 1.3E
- 1.06D + 0.5L + 1.3E
- 1.03D + 1.3E
- 0.76D + 1.3E

ANALYSIS AND DESIGN OF RC MODEL

A three-dimensional analysis of the building was performed in both the N-S (Y) and E-W (X) direction for seismic forces using ETABS and SAFE programs, see Figure 3. In the model, rigid diaphragms were assigned at each floor level. P-delta effects were also considered in the analysis. For a more accurate analysis, the cracked section property of the shear walls was taken as $I_{eff} = 0.35 I_g$ where I_g is the gross moment of inertia of the section. The compressive strength $f'c$, of concrete is 3000 psi and the reinforcement yield strength fy, is 60000 psi.

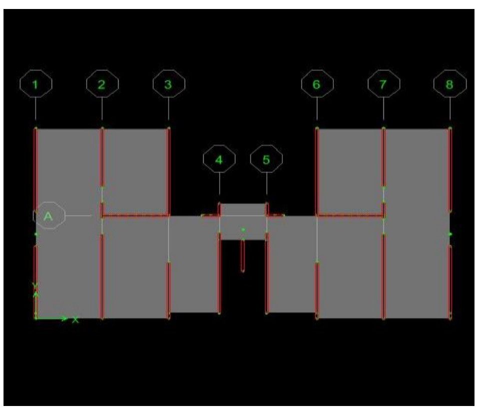

Figure 3
ETABS MODEL

Story Drift

The deflections of Level x at the center of the mass shall be determined in accordance with the following:

$$\delta x = \frac{Cd\,\delta xe}{I} \qquad (15)$$

Where, Cd is the deflection amplification factor. Cd for this building is 5. δxe is the deflections determined by ETABS analysis and I is the importance factor. I for this building is 1.0.

Interstory drift is calculated by the following:

$$\Delta = \delta x - \delta_{x-1} \qquad (16)$$

This interstory drift must not be larger than the allowable story drift $\Delta a = 0.02\, h = 0.02(8)12 = 1.92$ in. Lateral drifts calculations are shown in Tables 9 and 10 for all stories in the N-S and E-W directions.

Table 9
Lateral Displacement and Drift E-W

Story	δxe in.	δx in.	Drift in.
4	0.32	1.6	0.35
3	0.25	1.25	0.4
2	0.17	0.85	0.5
1	0.07	0.35	0.35

Table 10
Lateral Displacement and Drift N-S

Story	δxe in.	δx in.	Drift in.
4	0.08	0.4	0.10
3	0.06	0.3	0.10
2	0.04	0.2	0.15
1	0.01	0.05	0.05

Floor Deflection

The limit deflections for floors are found in Table 1604.3 of IBC [2]. Design deflections for the model building are from SAFE program analysis. Table 11 shows deflection limits from IBC and maximum deflections from model building, for live and dead loads. The longest span in the model is 120 in.

Floor Design

From ETABS the maximum positive moment, Mu = 1.44 K-ft and maximum negative moment Mu = -1.81 K-ft were obtained. The distance from top of floor to center of tension bar, is d = 6 in – 1 in = 5 in.

The coefficient of resistance is calculated from the following:

$$Rn = \frac{Mn}{bd^2} = \frac{1.44/_{0.9} \times 12000}{12 \times 5^2} = 64psi \tag{17}$$

Then stress ratio is computed from:

$$m = \frac{fy}{0.85f'c} = 23.53 \tag{18}$$

Percentage of steel require ρ is then calculated:

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2 \times m \times Rn}{fy}}\right) = 0.0011 \tag{19}$$

Now the area of steel is $As = \rho bd$ = 0.0011(12)5= 0.07 sq in. Then minimum steel is $Amin$= .0018bt = 0.0018(12)6 = 0.129 sq in. The area of steel use for design is $Amin$. Finally, the following formula is used for bar spacing:

$$s = \frac{12 \times A_{bar}}{As} \tag{20}$$

Recommendations for design are to use #3 @ 10".

The same procedure is used for the negative moment. As= 0.0905 sq.in. and $Amin$ = 0.129 sq in. As in previous calculations, $Amin$ governs. Then, recommendations for design are #3 @ 10" for negative steel.

Critical Shear Wall Design Line 2 N-S

From ETABS analysis axial load Pu=139.56K, moment Mu=202.95K-ft and shear Vu= 16.65K were obtained. The wall dimensions are l=189 in., h=8 ft per story or a total h=32 ft and b=5 in. Area gross, $Ag = l \times b$ = 945 sq in. Design is per ACI 318 Chapter 21 [3].

Reinforce requirements are determined by the following: the minimum reinforcement ratio ρ, in both directions is 0.0025, unless the design shear force Vu, is less than or equal to $f'c^{1/2}Ag = (3000)^{1/2}$ 945 = 51.75K. In this case Vu is less, so vertical rho, ρv is 0.0012 and horizontal rho, ρh is 0.0020. Then minimum vertical reinforcement area = 0.0012 x 12 x 5 = .072 sq in. per ft., from (20) we get #3 @ 18". The minimum horizontal reinforcement area = 0.0020 x 12 x 5 = .12 sq in. per ft., from (20) we get #3 @ 11".

Two curtains of steel are not needed to be provided because 2 $f'c^{1/2}Ag = 2(3000)^{1/2}$ 945 = 103.5K is more than Vu.

Shear strength verification is determined by calculating the nominal shear Vn, from (21):

$$ØVn = ØAg(\alpha\,(f'c)^{1/2} + \rho t\, fy) \tag{21}$$

Table 11
RC Floor Deflections

	Live Load	Live + Dead Load
IBC Limit	L/360	L/240
IBC Limit(in)	0.33	0.5
SAFE Max Deflection(in)	0.02	0.08

Where, α=2 when h/l=32/15.75=2.032 > 2. Horizontal rho is 0.0020. Then, $ØVn$= 0.85(945)(2(3000)$^{1/2}$ + 0.0020(60000))=184K > Vu, stating that #3 @ 11" recommendation is adequate. A second verification is needed, Vn shall not be larger than 8$f'c^{1/2}Ag$ = 414K, wish is adequate.

Boundary elements are needed if critical stress exceeds stresses calculated from (22).

$$f = \frac{Pu}{Ag} + \frac{6Mu}{bD^2} = \frac{139.5}{945} + \frac{6(202.95)12}{5(187)^2} = 0.23 \tag{22}$$

Then f < 0.2$f'c$= 0.600 Ksi. So no boundary is needed.

Design for flexure and axial load are determined by the following: the tension, Tu= (202.95 x 12)/ (0.9 x 187) = 14.47K. Then with ultimate steel stress of $Ø\,Fy$ = 0.9 x 60 = 54 Ksi. The require steel, $As = Tu/ Ø\,Fy$ = 0.27 sq in., say 1# 5 on both ends of wall and #3 @ 18". With this information, program PILOTYN7 was used to create and interaction diagram for the final verification and it was adequate, see Figure 4 above.

Critical Shear Wall Design Line A W-E

From ETABS analysis the axial load Pu=116.6K, moment Mu=371.89K-ft and shear Vu= 72.13K were obtained. The wall dimensions are l=125 in., h=8 ft per story or a total h=32 ft and b=6 in. Area gross, $Ag = l \times b$ = 750 sq in. Design is per ACI 318 Chapter 21 [3].

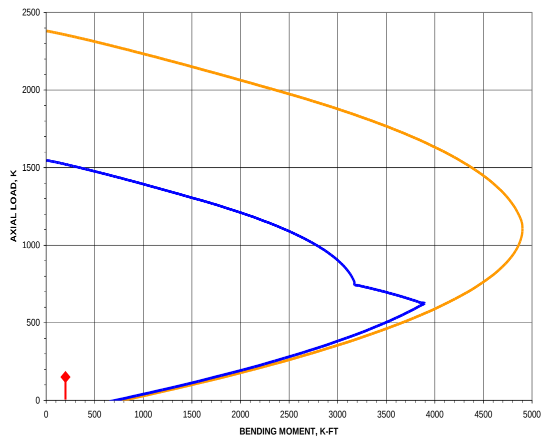

Figure 4
Interaction Diagram for Shear Wall Line 2

Reinforce requirements are determined by the following: the minimum reinforcement ratio ρ, in both directions is 0.0025, unless the design shear force is less than or equal to $f'c^{1/2}Ag$ = $(3000)^{1/2}$ 750 = 41.07K. In this case Vu is larger, then vertical rho and horizontal rho are ρ=0.0025. Then, minimum reinforcement area is 0.0025 x 12 x 6 = .18 sq in. per ft., from (20) we get 2#3 @ 14" e.w.

Two curtains of steel are not needed to be provided because 2 $f'c^{1/2}Ag$ = $2(3000)^{1/2}$ 750 = 82.15K is more than Vu.

Shear strength verification is determined by calculating the nominal shear Vn, from (21):

$$ØVn= ØAg(\alpha\ (f'c)^{1/2} + \rho t\ fy) \tag{21}$$

Where, α=2 when h/l=32/10.4=3 > 2. Horizontal rho is 0.0026. Then, $ØVn$ =0.85 (750) $(2(3000)^{1/2}$+ 0.0026(60000)) = 176.9K > Vu, stating that 2#3 @ 14" recommendation is adequate. A second verification is needed, Vn shall not be larger than 8 $f'c^{1/2}Ag$ = 328.6K, wish is adequate.

Boundary elements are needed if critical stress exceeds stresses calculated from (22).

$$f = \frac{Pu}{Ag} + \frac{6Mu}{bD^2} = \frac{116}{750} + \frac{6(371.8)12}{6(125)^2} = 0.44 \tag{22}$$

Then $f < 0.2f'c$= 0.600 Ksi. So no boundary is needed.

Design for flexure and axial load are determined by the following: tension, Tu= (371.8 x 12)/ (0.9 x 123) = 40.30K. Then with ultimate steel stress of $Ø\ Fy$ = 0.9 x 60 = 54 Ksi. The require steel, $As = Tu/\ Ø\ Fy$ = 0.74 sq in., say 4 # 4 on both ends of wall and 2#3 @ 14". PILOTYN7 program was use for final verification (adequate), see Figure 5.

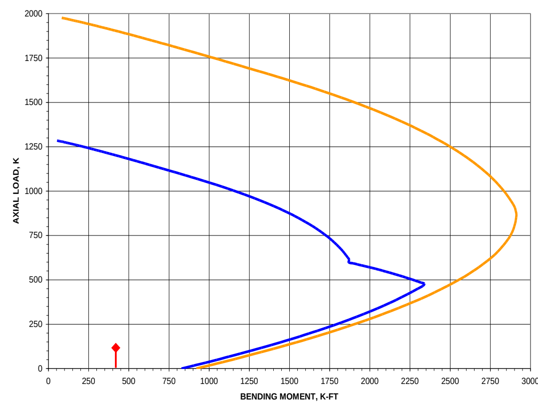

Figure 5
Interaction Diagram for Shear Wall Line A

Analysis and Design of SCIP Model

A three-dimensional analysis of the building was performed in both the N-S (Y) and E-W (X) direction for seismic forces using ETABS and SAFE programs. In the model, rigid diaphragms were assigned at each floor level. P-delta effects were also considered in the analysis. For a more accurate analysis as recommended by references [4] [5][7][8], the walls were modeled considering a thickness of 3 in. (effective RC thickness) and the Modulus of elasticity for concrete, Ec = 426,700 psi (1/5 of Ec). Poisson modulus is equal to 0.15. For floors an equivalent thickness of 3 in. was used.

Story Drift on SCIP

The deflections of Level x at the center of the mass shall be determined in accordance with the following:

$$\delta x = \frac{Cd\,\delta xe}{I} \tag{15}$$

Where, *Cd* is the deflection amplification factor. For this building is 5. δxe is the deflections determined by ETABS analysis and *I* is the importance factor. *I* for this building is 1.0.

Interstory drift is calculated by the following:

$$\Delta = \delta x - \delta_{x-1} \quad (16)$$

This interstory drift must not be larger than the allowable story drift $\Delta a = 0.02\ h = 0.02(8)12 = 1.92$ in. Lateral drifts calculations are shown in Tables 12 and 13 for all stories in the N-S and E-W directions.

Table 12
Lateral Displacement and Drift E-W

Story	δxe in.	δx in.	Drift in.
4	0.46	2.3	0.55
3	0.35	1.75	0.7
2	0.21	1.05	0.615
1	0.087	0.435	0.35

Floor Deflection on SCIP

The limit deflections for floors are found in Table 1604.3 of IBC [2]. Design deflections for the model building are from SAFE program analysis. Table 14 shows deflection limits from IBC and maximum deflections from model building, for live and dead loads. The longest span in the model is 120 in.

Table 13
Lateral Displacement and Drift N-S

Story	δxe in.	δx in.	Drift in.
4	0.1	0.5	0.135
3	0.073	0.365	0.155
2	0.041	0.21	0.135
1	0.015	0.075	0.075

SCIP Floor Design

From ETABS the positive moment, Mu = 0.96K-ft and negative moment Mu = -1.46K-ft were obtained. Distance from top of floor to the center of tension bar, d = 5.82 in.

The coefficient of resistance, *Rn* is calculated from (17) and is 31.49 psi.

Then the stress ratio is computed from (18) an is 23.53.

Table 14
SCIP Floor Deflections

	Live Load	Live + Dead Load
IBC Limit	L/360	L/240
IBC Limit(in)	0.33	0.5
SAFE Max Deflection(in)	0.06	0.2

Percentage of steel require, ρ is then calculated from (19) and is 0.0005 < than ρ min.

Now the area of steel is $Amin$= 0.0018bt = 0.0018(12)2.5 = 0.054 sq in. The SCIP panels have an area of steel equal to 0.069 sq in. per ft., it is adequate.

The same procedure is used for the negative moment, but d = 4.32 in. As= 0.0778 sq in. and $Amin$ = 0.054 sq in., *As* governs. Then SCIP area of steel 0.069 sq in. is subtracted from *As*. Area *As* not covered is 0.02, so use 1#3 @ 18”.

SCIP Critical Shear Wall Design Line 2 N-S

From ETABS analysis the axial load Pu=101.67K, moment Mu=175.63K-ft and shear Vu= 9.64K were obtained. The wall dimensions are l=189 in., h=8 ft per story or a total h=32 ft and b=3 in. Area gross, $Ag = l \times b$ = 567 sq in. Design is per ACI 318 Chapter 21 [3].

Reinforce requirements are determined by the following: the minimum reinforcement ratio in both directions is 0.0025, unless the design shear force is less than or equal to $f'c^{1/2}Ag = (3000)^{1/2}\ 567$ = 31.05K. In this case Vu is less, then vertical rho, ρv=0.0012 and horizontal rho, ρh=0.0020. Vertical reinforcement area, Asv = 0.0012 x 12 x 3 = .043 sq in. per ft., the panel vertical steel is 0.1397 sq in. per ft., so is adequate. The minimum horizontal reinforcement area, Ash = 0.0020 x 12 x 3 = 0.072 sq in. per ft., the panel horizontal steel is 0.0737 sq in. per ft. so is adequate.

Shear strength verification is determined by calculating the nominal shear Vn, from (21):

$$ØVn = ØAg(\alpha\ (f'c)^{1/2} + \rho t\ fy) \quad (21)$$

Where, α=2 when h/l=32/15.75=2.032 > 2. Horizontal rho is 0.0020. Then, $ØVn$=0.85(567) (2 $(3000)^{1/2}$ + 0.0020(60000)) =110K > Vu, stating that SCIP panel is adequate. A second check is needed, Vn shall not be larger than 8 $f'c^{1/2}Ag$ = 248.4K, wish is adequate.

Boundary elements are needed if critical stress exceeds stresses calculated from (22).

$$f = \frac{Pu}{Ag} + \frac{6Mu}{bD^2} = \frac{101.6}{567} + \frac{6(175.63)12}{3(187)^2} = 0.29 \qquad (22)$$

Then $f < 0.2f'c$= 0.600 Ksi. So no boundary is needed.

Design for flexure and axial load are determined by the following: tension, Tu= (175.6 x 12)/ (0.9 x 187) = 12.53K. Then with ultimate steel stress of Ø Fy = 0.9 x 60 = 54 Ksi. The require steel, As = Tu/ Ø Fy = 0.23 sq in., say 1# 3 on both ends of wall and steel from panel. PILOTYN7 program was used for final verification (adequate), see Figure 6.

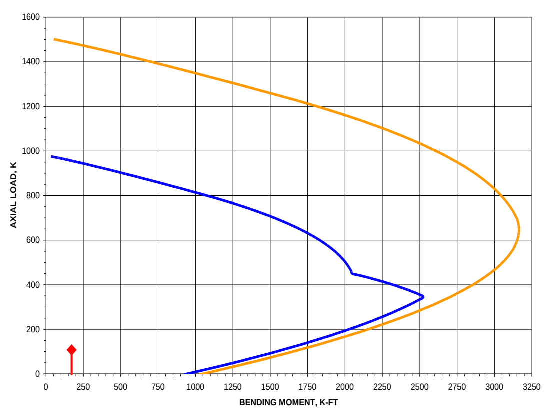

Figure 6
SCIP Interaction Diagram for Shear Wall Line 2

SCIP Critical Shear Wall Design Line A W-E

From ETABS analysis the axial load Pu=65.4K, moment Mu=274.92K-ft and shear Vu= 45.54K. The wall dimensions are l=125 in., h=8 ft. per story or a total h=32 ft. and b=3 in. Area gross, Ag =l x b = 375 sq in. Design is per ACI318 Chapter 21 [3].

Reinforce requirements are determined by the following: the minimum reinforcement ratio in both directions is 0.0025, unless the design shear force is less than or equal to $f'c^{1/2}Ag$ = $(3000)^{1/2}$ 375 = 20.5K. In this case Vu is larger, then vertical rho and horizontal rho are ρ=0.0025. Then minimum reinforcement area = 0.0025 x 12 x 3 = .09 sq in. per ft., panel vertical steel is 0.1397 sq in. per ft., so is adequate. Panel horizontal steel is 0.0737 sq in. per ft., so is below require by 0.0163 sq in. By (20) we get #3 @ 18".

Shear strength verification is determined by calculating the nominal shear Vn, from (21):

$$ØVn= ØAg(\alpha\ (f'c)^{1/2} + \rho t\ fy) \qquad (21)$$

Where, α=2 when h/l=32/10.4=3 > 2. Horizontal rho is 0.0045. Then, $ØVn$=0.85(375) (2 $(3000)^{1/2}$ + 0.0045(60000)) =120.9K > Vu, stating that SCIP panel steel with #3 @ 18" is adequate. A second verification is needed, Vn shall not be larger than 8 $f'c^{1/2}Ag$ = 164.31K, wish is adequate.

Boundary elements are needed if critical stress exceeds stresses calculated from (22).

$$f = \frac{Pu}{Ag} + \frac{6Mu}{bD^2} = \frac{65.4}{375} + \frac{6(274.9)12}{3(125)^2} = 0.59 \qquad (22)$$

Then $f < 0.2f'c$= 0.600Ksi. So no boundary is needed.

Design for flexure and axial load are determined by the following: tension, Tu= (274.9 x 12)/ (0.9 x 123) = 29.79K. Then with ultimate steel stress of $Ø Fy$ = 0.9 x 60 = 54 Ksi. So require steel, $As = Tu/ Ø Fy$ = 0.55 sq.in., say 1# 3 on both ends of wall and steel from panel. PILOTYN7 program was use for final verification (adequate), see Figure 7.

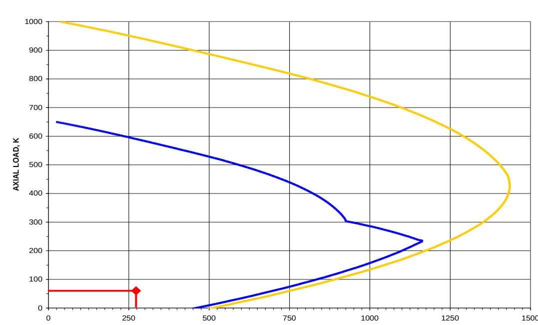

Figure 7
SCIP Interaction Diagram for Shear Wall Line A

CONCLUSIONS AND RECOMMENDATIONS

Comparing the drift of SCIP with RC, it was found that drifts in both directions were in the range of 65% larger in SCIP than in RC. The deflections of the floor for live and dead load are in the range of 200 % larger in SCIP than in RC. Nevertheless, these values of the SCIP were below the limits of the codes requirements. If strength models proposed in the literature are applicable, the composite action is effective, and the response is ductile, then the SCIP system could perform under code requirements, although being significantly more flexible than the RC system. It is important to point out that the normal stress due to axial load and bending moment in the SCIP panels almost reached code limits, while the RC walls were far from the limit. This result strongly suggests that the SCIP system would not perform adequately on taller buildings.

The lack of extensive documentation on SCIP systems strongly suggests the following research topics as possible future work:

- Investigate strength models for shear and bending on SCIP systems (by performing laboratory, analytical, and numerical studies)
- Investigate the effective composite action between reinforced concrete and foam layers on the SCIP system.
- Study the long term load effects in floor deflections of SCIP systems.
- Study the behavior of diaphragms on SCIP buildings.
- Investigate the proper inertia (I) and concrete modulus of elasticity (E) values to be used in the analysis of these systems.
- Investigate the ability of SCIP system to withstand large deformations, and provide a ductile behavior equivalent to RC walls.

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