Reinforced Concrete vs Structural Concrete Insulated Panels Performance and Cost Comparison for 4-story Building

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Abstract *– This paper presents the modeling of a 4-story building to compare the structural behavior of reinforced concrete (RC) versus the structural concrete insulated panels (SCIPS) under the code required loads and deflection requirements. The model analysis has been performed according to IBC 2018, ASCE7-16, and ACI318-14. Both models were created on ETABS to be evaluated under the gravity, wind, and seismic loads. The objective is to evaluate if the RC and SCIPS can perform under the code requirements and compare the behavior and cost of both systems. A grey shell cost calculation was performed to include on the evaluation the cost difference for both systems. As a conclusion both systems can perform under the code prescribed loads, but they have a cost difference being the SCIPs slightly more expensive.*

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

INTRODUCTION

This paper presents the modeling of a 4-story structure for residential use. The evaluated structure was created for the purpose of this study and does not represent a real structure that will be built. On this evaluation, the behavior and performance of the reinforced concrete structure was compared with the behavior and performance of the structural concrete insulated panels.

Structural concrete insulated panels, also referred to as SCIPs, are becoming a popular alternative to traditional reinforced concrete. Obtained from [1, SCIPs system was created in Italy by Mr. Angelo Candiracci in 1981 and, since its creation, the system has been optimized along its 42 years of research and development. SCIPs system is composed of wall and floor panels. The wall panels have an expanded polystyrene (EPS) core in the center that varies in thickness depending on the selected panel (Figure 1). On both sides of the EPS, the system has an electro welded wire mesh that is connected from one side to the other ensuring that wire mesh behaves as a single unit. On site structural mortar is applied to both sides to have the final structural element. The floor panels are installed horizontally with traditional concrete poured on top and structural mortar applied in the bottom (Figure 2). Currently there are SCIPs manufacturing plants in 35 countries including two plants in USA. One common comment from designers is that the system needs more documentation about code compliance and structural behavior.

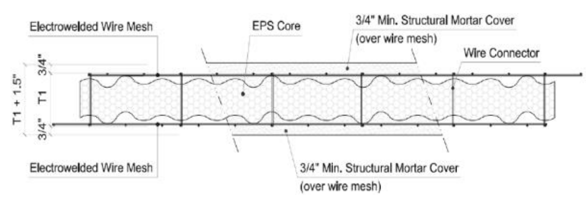

Figure 1

PSM Wall Section

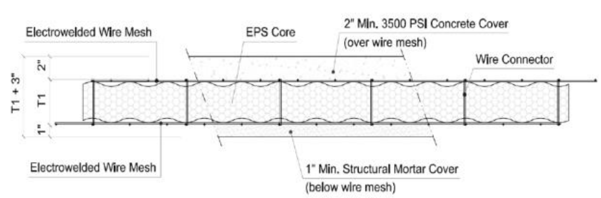

Figure 2

PSS Floor Slab Section

The purpose of this paper is to present an example on how SCIPs system can be modeled and to add more technical information about the system and its behavior compared to reinforced concrete. A cost analysis is also included in the evaluation and comparison of both structural systems. The SCIP system manufacturer indicates that the system can perform under the loads, including seismic prone areas, but also the system has incorporated a high thermal insulation performance that will drastically reduce the air conditioning and heating costs. The SCIPs manufacturer also states that the system has a reduction in construction time of 40%, which will result in a labor and equipment cost reduction. The systems were evaluated by applying gravity, wind, and seismic loads according to code requirements of the [2], [3], and [4]. A three-dimensional analysis was performed in both X and Y directions using ETABS. Rigid diaphragms were assigned to each floor and roof to distribute the lateral loads. After running the model on ETABS, manual calculations of seismic loads, wind loads, and reinforcement ratios for shear wall and slabs were performed to verify the model.

Building and model descriptions

The proposed structure is a 68'-9" by 41'-3" on each floor. It consists of two three-bedroom apartments per floor (Figure 3). Each apartment has one master bedroom, with a private bathroom and walk-in closet, and two additional bedrooms. Each apartment also has a shared bathroom, kitchen, and living room. The units above the first floor are accessed via stairways located in the center of the building. On the reinforced concrete model, all walls and the slabs are designated as six inches cast in place concrete. On the SCIPS system the walls are comprised of PSM80 wall panel with an EPS core of 315" thickness. The panel has two electro welded wire mesh on both sides. The distance from one wire mesh to the other is 4.61" with a finish panel thickness of 5.32" and a self-weight of 34.85 pounds per square feet. Both wire mesh is secured together by Gauge 11 wire connectors. The principal longitudinal wire is Gauge 11, with a spacing of 3". The transversal reinforcement is a Gauge 12.5 wire with a uniform spacing of 2.6". The wall panels are covered on both sides with a structural mortar with a minimum compressive strength of 4,000 psi at 28 days. The slab panels are PSS120 with an EPS core of 4.72" and a wire-to-wire distance of 4.76". The panel is covered on the top with 2" cast in place concrete layer of 3,500 psi and in the bottom with structural mortar. The PSS120 slab panel has a finished thickness of 8.55" and a self-weight of 47.38 psf. As per [5] and [6] for the ETABS model, an equivalent thickness of 2.96" was used for all PSM80 walls. For the floor and roof panels on the ETABS model an equivalent of 3" was used.

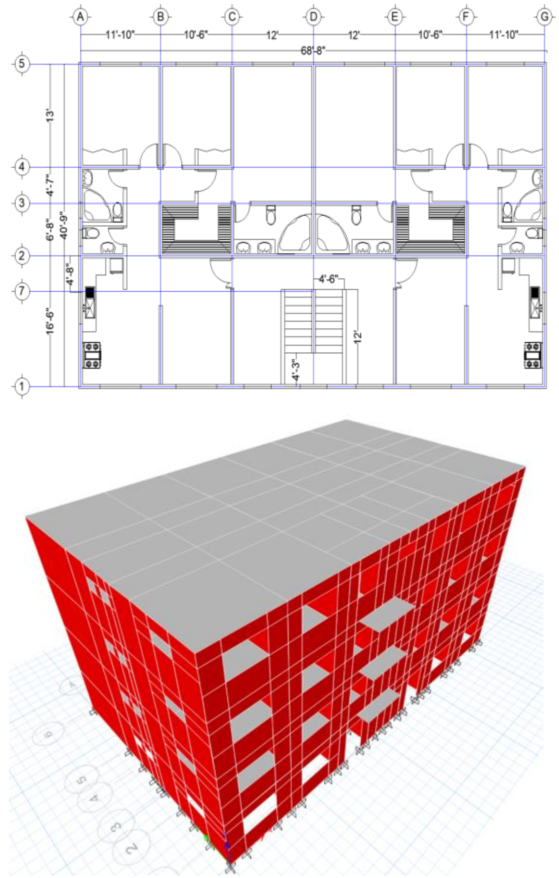

Figure 3

Typical Floor Plan and 3D model image

Live load was obtained from ASCE7 Table 4.3-1. For Residential use, the table provides a live load of 40 psf to be applied to all floors. Roof live load as per ASCE7 Table 4.3-1 is 40 psf for both reinforced concrete and SCIP systems.

Dead Load for reinforced concrete system:

- For Walls is 75 psf.
- For floor slabs is 75 psf.
- For roof slabs is 75 psf.

Dead Load for Structural Insulated Concrete system:

- For Wall with panel PSM 80 is 34.85 psf.
- For floor slabs PSS 120 is 47.38 psf.
- For roof slabs PSS120 is 47.38 psf.

Load Combinations IBC 2018 Section 1605:

Notations:

D= Dead Load

E= Combined effect of horizontal and vertical earthquake forces

F= Load due to fluids

F_a= Flood Load

H= Load due to lateral earth pressure, ground water pressure or pressure of bulk materials

L= Roof live load greater than 20 psf and floor live load

L_r= Roof live load of 20 psf or less

R= Rain load

S= Snow load

W= Load due to wind pressure

Basic Load Combination:

1.4(D + F)

1.2(D + F) + 1.6(L + H) + 0.5(L_r or S or R)

1.2(D + F) + 1.6(L_r or S or R) + 1.6H + (f_1L or 0.5W)

1.2(D + F) + 1.0W + f_1L + 1.6H + 0.5(L_r or S or R)

1.2(D + F) + 1.0E + 1.6H

0.9D + 1.0W + 1.6H

0.9(D + F) + 1.0E + 1.6H

There are seven basic load combinations; however only two combinations include seismic load effects, E. To consider the maximum gravity effect load combination will be modified as per (1).

$$(1.2 + 0.2S_{DS})D + 1.2F + \rho Q_E + f_1 L + 1.6H + f_2 S \qquad (1)$$

On (1) $0.2S_{DS}$ represents the vertical seismic load effects. Also, the term ρQ_E represents the effect due to horizontal seismic forces. Since the project does not have flood loads or earth pressure equation (1) is modified as (2).

$$1.36D + 1E_X + 0.5L \qquad (2)$$

The second combination that includes seismic loads is (3) and represents the minimum gravity effect where the seismic effect is in the opposite direction to gravity.

$$(0.9 - 0.2S_{DS})D + 0.9F + \rho Q_E + 1.6H \qquad (3)$$

On (3) $-0.2S_{DS}$ represents the vertical seismic load effects. Also, the term ρQ_E represents the effect due to horizontal seismic forces with a sign that opposes the sign of D. Since the project does not have flood loads or earth pressure (3) is modified as (4).

$$0.74D - 1.0E_X \qquad (4)$$

Both (2) and (4) were included in the load combinations that were used for the analysis.

A three-dimensional analysis was performed in both directions X and Y using ETABS to determine the gravity, wind, and seismic loads distribution through the structure. At each floor level and roof a rigid diaphragm was defined (Figure 4). To account for plastic behavior, the gross inertia was reduced to $0.35I_g$ as per Table 6.6.3.1.1.(a) on ACI318 code. The compressive strength of concrete is taken as f'_c 4,000 psi and the reinforcement yielding strength is taken as 60,000 psi. Two parallel models were created, one for the reinforced concrete system and the other for the SCIPs.

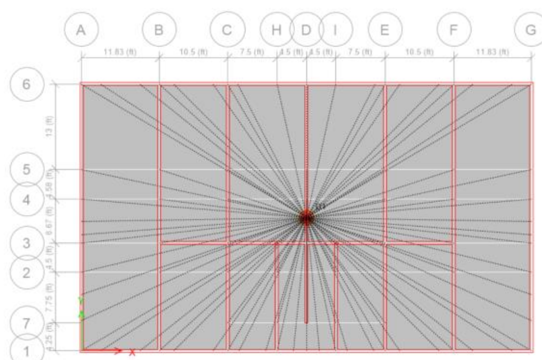

Figure 4

Rigid Diaphragm Definition

Wind Loads

According to IBC2018 Section 1609 WIND LOADS on every building or structure shall be determined in accordance with Chapters 26 to 30 of [4]. For this study, ASCE7-16 was used to calculate the design wind pressure. The structure use is residential apartments that will fall under Risk Category II, as per Table 1604.5 of IBC 2018. Using ASCE7 Figure 26.5-1B, the Basic Wind Speed was obtained for different areas of Puerto Rico. For this study, the building is in the northwest area of Puerto Rico that has a Basic Wind Speed of 150 mph. The wind directionality factor K_d is determined from Table 26.6-1 on ASCE 7. For the building's main wind force resisting system (MWFRS), the factor K_d is 0.85. Exposure category is determined as follows. For this study, the structure is assumed to be located in an urban area. According to Section 26.7.2 of ASCE 7, for urban areas the Surface Roughness is Category B. For the exposure category, the conditions of the study building do not meet the requirements for Exposure B or D; in that case Exposure C applies. Directional procedure will be used to calculate the wind loads on the MWFRS. For this study, the structure will be located on level ground where the features of hills, ridge or escarpment are not present; therefore, the Topographic Factor K_{zt} is equal to 1. As per section 26.9 of ASCE 7 and Note 1 of Table 26.9-1, conservatively a ground elevation factor K_e of 1 will be used. Per Section 26.11.1, the gust effect factor for a rigid building or other structure is permitted to be taken as G = 0.85.

All openings on the structure are covered and protected by wind rated doors and windows; therefore, the area of openings is less than 0.01 A_g placing the structure under the classification of enclosed building. The internal pressure is moderate with internal pressure coefficients of GC_{pi} =+0.18 and -0.18. The velocity pressure is calculated using (5).

$$q_z = 0.00256 K_z K_{zt} K_d K_e V^2 \left(\frac{lbs}{ft^2}\right) \tag{5}$$

Where:

K_z = velocity pressure exposure coefficient

K_{zt} = topographic factor

K_d = wind directionality factor

K_e = ground elevation factor

V = basic wind speed

q_z = velocity pressure at height z

From Table 26.10-1, the applicable pressure coefficients K_h and K_z for exposure category B are presented on Table 1.

Table 1

Velocity Pressure Exposure Coefficient

Level	**Height z, ft**	**$K_h = K_z$**
4	40	0.76
3	30	0.70
2	20	0.62
1	10	0.57

Wind Loads for the main wind force resisting system is calculated as follows. External pressure coefficients were obtained from ASCE 7 Figure 27.3-1. For windward wall C_p is 0.8 and for leeward wall is -0.5. Table 2 presents the velocity pressures for the different levels of the structure. The design pressure is calculated using (6) and results are presented on Table 3. The wind design force applied on each diaphragm corresponding to each floor is calculated with (7). Wind design pressure forces calculation results are shown on Table 4 and Table 5 for both directions N-S and E-W, respectively.

$$p = q * G * C_p \tag{6}$$

$$V = (q_{zw} GCp - q_{zL} GCp) * h * L \tag{7}$$

Table 2

Velocity Pressure qz for V=150 mph

Level	Height z (ft)	K_z=K_h	q_z (psf)(5)
4.0	40	1.04	51.09
3.0	30	0.98	48.09
2.0	20	0.90	44.16
1.0	10	0.85	41.56

Table 3

Design Pressure

	L	**Height z (ft)**	**q_z (psf)**	**G**	**C_p**	**q_zGC_p (psf)**
Wind-ward	4	40	51.09	0.85	0.8	34.74
	3	30	48.09	0.85	0.8	32.70
	2	20	44.16	0.85	0.8	30.02
	1	10	41.56	0.85	0.8	28.26
L-ward			37.21	0.85	-0.42	-13.28

Table 4

Design Force Wind N-S, Y direction

Level	Height h, ft	$q_{zw}GCp$ (psf)	$q_{zL}Cp$ (psf)	L (ft)	V (Kips)
4	5	34.74	-13.28	40.75	11.25
3	10	32.70	-13.28	40.75	21.55
2	10	30.03	-13.28	40.75	20.30
1	10	28.26	-13.28	40.75	19.47

Table 5

Design Force E-W, X direction

Level	Height h, ft	$q_{zw}GCp$ (psf)	$q_{zL}Cp$ (psf)	L (ft)	V (Kips)
4	5	34.74	-13.28	68.66	18.96
3	10	32.70	-13.28	68.66	36.31
2	10	30.03	-13.28	68.66	34.20
1	10	28.26	-13.28	68.66	32.80

Seismic Loads

Seismic design criteria are determined as per ASCE7-16 Chapter 11. From ASCE7 Figure 22-6, the 0.2-second spectral response acceleration of S_s=1.02 and the 1-second spectral response acceleration of S_1=0.47 were obtained. For this structure, a site class D was assumed. The site coefficients F_a=1.2 and F_v=1.83 were obtained from ASCE 7 Tables 11.4-1 and 11.4-2 respectively. The spectral response acceleration parameters S_{MS}=1.22 and S_{M1}=0.86 were calculated using (8) and (9) respectively.

$$S_{MS} = F_a * S_s \quad (8)$$

$$S_{M1} = F_v * S_1 \quad (9)$$

The designs spectral acceleration parameters S_{DS}=0.816 and S_{D1}=0.57 are calculated using (10) and (11), respectively.

$$S_{DS} = \frac{2}{3} * S_{MS} \quad (10)$$

$$S_{D1} = \frac{2}{3} * S_{M1} \quad (11)$$

Using the design spectral acceleration parameters and codes, a seismic design category D was determined. For this study, a Risk Category I and an Importance Factor of 1 were selected. The seismic load has been calculated using the equivalent lateral load procedure. The approximate fundamental period T_a=0.318 seconds has been calculated with (12). Parameters C_t and x were obtained from the code and h_n is the structural height.

$$T_a = C_t * h_n^x \quad (12)$$

The seismic response coefficient C_s=0.163 is calculated using (13), where R=5 is the response modification factor and I_e=1 is the importance factor. From ASCE 7, a long period transition period T_L=12 was obtained.

$$C_s = \frac{S_{DS}}{\frac{R}{I_e}} \quad (13)$$

The seismic response coefficient C_s has upper and lower limits that need to be verified. The upper limit is 9.013 and the lower limit is 0.036. The upper limit and lower limit were calculated with (14) and (15) respectively.

$$C_{s\ upper} = \frac{S_{D1}}{T_a * \frac{I_e}{R}} \quad (14)$$

$$C_{s\ lower} = 0.044 * S_{DS} I_e \quad (15)$$

Reinforced Concrete Seismic Response

For the reinforced concrete system, the minimum slab thickness shall not be less than 1/24. The biggest span distance on the structure is 12ft; therefore, the minimum slab thickness is 6”. For uniformity, all the slabs on the structure were 6” thick. The minimum wall thickness for reinforced concrete bearing walls is the greater of 4” or 1/25 times the lesser unsupported length and unsupported height. The unsupported height of the structure is 10 ft; therefore, the minimum thickness is 4.8”. For the model for this study, 6” were conservatively used for all concrete walls. The calculated seismic weight W is 1,946 kips. The seismic base shear V=318 kips is calculated with (16) using the equivalent lateral force procedure.

$$V = C_s * W \quad (16)$$

The vertical distribution of seismic forces was calculated with formulas (17) and (18). The parameter k for structures that have a period of 0.5 second or less is equal to 1.

$$C_{vx} = \frac{W_x * h_x^k}{\sum_{i=1}^{n} W_i * h_i^k} \quad (17)$$

$$F_x = C_{vx} * V \quad (18)$$

The vertical distribution of seismic forces for the reinforced concrete structure are presented on Table 6 and can be seen graphically in Figure 5 and Figure 6.

Table 6

Seismic Forces Vertical Distribution RC

Level	Weight (kips)	Height (feet)	w x h	C_{vx}	F_x (kips)
4	392	40	15680	0.34	106.50
3	518	30	15540	0.33	105.55
2	518	20	10360	0.22	70.36
1	518	10	5180	0.11	35.18
Total	**1,946**	---	**46,760**	**1.00**	**318**

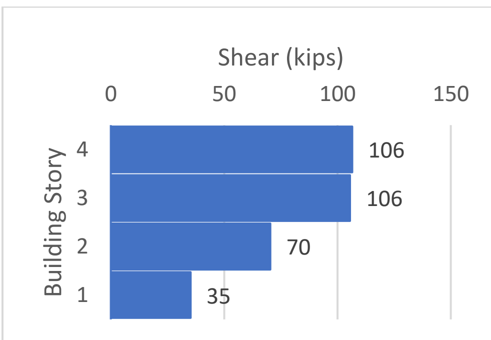

Figure 5

Vertical Seismic Force Distribution RC

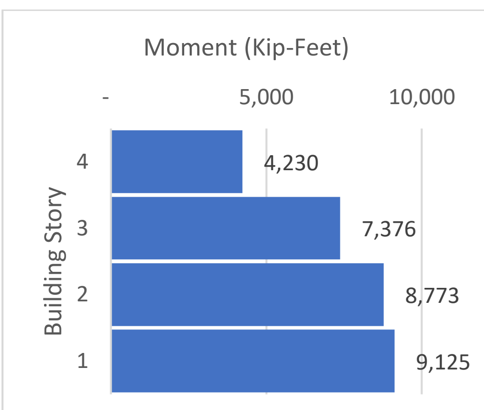

Figure 6

Overturning Moment per story RC

Structural Concrete Insulated Panels (SCIPs) Seismic Response

As per manufacturer's technical evaluation report, the allowable axial service load capacity of a PSM80 panel at 10 feet of height is 11,815 pounds per linear foot of wall. It was estimated that the axial load on the first floor is around 4,200 pounds per linear foot of wall by performing a simple load calculation on a center wall. Therefore, the PSM80 panel was preliminary selected for all the walls on the structure. The PSM80 panel once finished with structural mortar has a thickness of 6.11", that is a similar thickness as the reinforced concrete structure. This way the interior space on both structures is very similar. The self-weight of a finished PSM80 wall is 34.85 psf.

For the slab preliminary selection as per manufacturer's technical specifications for a 12-foot span, a PSS120 slab panel can carry 45 psf of live load. The live load limit is controlled by deflection. A PSS120 slab panel once finished with mortar on the bottom face and concrete on the top has a self-weight of 47.38 psf. The seismic weight of the SCIPs structure is estimated as W = 1,155 kips. With similar calculation as the reinforced concrete structure, a base shear V = 188.496 kips was calculated. The vertical distribution of the seismic forces is presented on Table 7 and can be seen graphically in Figure 7 and Figure 8.

Table 7

Seismic Forces Vertical Distribution SCIPs

Level	Weight (kips)	Height (feet)	w x h	C_{vx}	F_x (kips)
4	240	40	9586	0.34	65
3	305	30	9158	0.33	62
2	305	20	6105	0.22	41
1	305	10	3053	0.11	21
Total	**1,155**	---	**27,902**	**1.00**	**188**

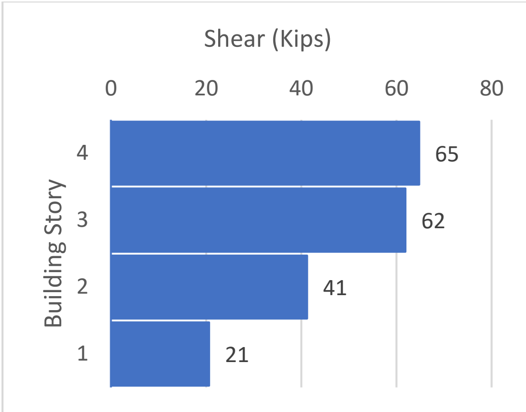

Figure 7

Vertical Distribution of Seismic Base Shear SCIPs

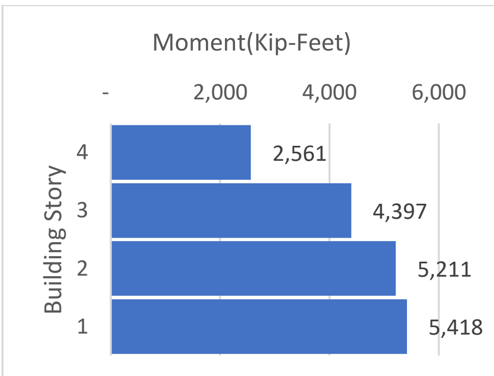

Figure 8

Overturning Momen per Story SCIPs

Story drift

Story drift is defined as the difference of deflections at the center of mass at the top and bottom of the story under consideration. To allow or account for inelastic deformations, story drift is determined using the deflection amplification factor C_d and considering the Importance Factor I_e. At story x, the story drift is calculated from (19).

$$\Delta_x = \delta_x - \delta_{x-1} \tag{19}$$

Where δ_x is the design displacement of the structure, which is the actual anticipated inelastic response displacement caused by the design lateral forces calculated using (20).

$$\delta_x = \frac{C_d * \delta_{xe}}{I_e} \tag{20}$$

Where δ_{xe} is the theorical deflection calculated from an elastic analysis ant level *I* under code prescribed seismic forces, F_x. Aplification factor C_d is obtained from Table 12.2-1 on ASCE7-16 code. Importance factor I_e for this structure is 1.0. Deflection values have been obtained from ETABS analysis and presented on Table 8 and Table 9 for the critical direction X on both structural systems.

Table 8

Story Drift on Reinforced Concrete Structure

Story	hx (ft)	δxe (in)	C_d	δx (in)	Limit (in)	Check
4	10	0.379653	5	1.898267	2.4	OK
3	10	0.283539	5	1.417697	2.4	OK
2	10	0.189942	5	0.949709	2.4	OK
1	10	0.090516	5	0.45258	2.4	OK

Table 9

Story Drift on SCIPS Structure

Story	hx (ft)	δxe (in)	C_d	δx (in)	Limit (in)	Check
4	10	0.338242	5	1.69121	2.4	OK
3	10	0.273292	5	1.36646	2.4	OK
2	10	0.180307	5	0.901535	2.4	OK
1	10	0.072758	5	0.36379	2.4	OK

Model Verification

To confirm the reinforced concrete shear wall design provided by ETABS, a manual calculation has been performed for a shear wall axis G of the reinforced concrete model. From the ETABS model analysis, the subject wall has an axial load P_u = 39.827 kips and a moment M_u = 101.374 kip * ft. The shear wall length is 35”, and its height is 10 ft. Vertical reinforcement provided is 6 #6 bars @15.93 inches providing a steel area of 2.64 in^2. The reinforcement density ρ = 0.01257. As per ACI318 code, the minimum spacing is 18”. By providing the minimum spacing, the reinforcement density adjusted for 18” is ρ = 0.0111. Reinforcement is adequate when compared to the minimum reinforcement density required by the code of 0.0025. As per manual calculation with the proposed reinforcement the moment capacity of the shear wall is M_n = 340.88 kip*ft which is greater than the actual moment M_u = 101.374 kip*ft. ETABS design and proposed reinforcement is adequate. The moment capacity M_n was calculated using (21). The shear wall interaction diagram was created using the section designer on ETABS and can be seen in Figure 9.

$$M_n = T * \left(\frac{l_{wflexural}}{2}\right) + N_u * \left(\frac{l_{wflexural-c}}{2}\right) \quad (21)$$

The shear capacity was also verified by manual calculations. For shear reinforcement ETABS design provided 2 #4 bars spaced at 18 inches. This provides a shear reinforcement area of 0.4 square inches. The reinforcement density ρ = 0.00556 which is greater than the minimum required by code of 0.0025. The calculated shear capacity ∅Vn is 34.443 kips and it take in to account the contribution of the concrete and reinforcement steel. The shear capacity of the wall was calculated with (22), (23) and (24). The shear demand Vu =33.32 kips obtained from ETABS is smaller than the shear capacity of the wall therefore ETABS shear wall design is adequate.

$$\emptyset V_n = \emptyset V_c + \emptyset V_s \quad (22)$$

$$V_c = (2 * \left(1 + \frac{N_u}{500 * A_g}\right) * \lambda * \sqrt{f'c} * h * d \quad (23)$$

$$V_s = \frac{A_v f_{yt} d}{s} \quad (24)$$

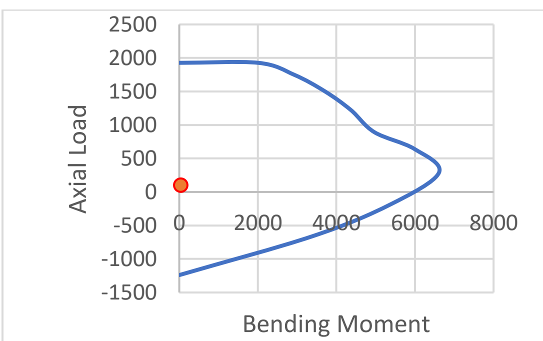

Figure 9

Reinforced Concrete Wall Interaction Diagram

In the same manner as the reinforced concrete structure, to confirm the SCIPs system shear wall design provided by ETABS a manual calculation has been performed for a shear wall on axis G of the reinforced concrete model. From the ETABS model analysis the subject wall has an axial load P_u = 9.35 kips and a moment M_u = 1.49 kip * ft. The shear wall length is 34.96 inches, and the height is 10 ft. Vertical reinforcement provided is 2 #6 bars @17.4 inches providing a steel area of 0.88 in^2. The reinforcement desinty ρ=0.0085. As per ACI318 code, the minimum spacing is 18”. By providing the minimum spacing the reinforcement density adjusted for 18” is ρ=0.0075. Reinforcement is adequate when compared to the minimum reinforcement density required by the code of 0.0025. As per manual calculation with the proposed reinforcement the moment capacity of the shear wall is M_n=137.94 kip*ft which is greater than the actual moment M_u = 1.49 kip*ft. ETABS design and proposed reinforcement is adequate. The moment capacity M_n was calculated using (21). The interaction diagram for the SCIPs shear wall shown on Figure 10 was obtained from [3].

The shear capacity was also verified by manual calculations. For shear reinforcement ETABS design provided 2 #4 bars spaced at 18 inches. This provides a shear reinforcement area of 0.4 in^2. The reinforcement density ρ = 0.00278 which is greater than the minimum required by code of 0.0025. The calculated shear capacity $\emptyset V_n$ is 23.7 kips and it take in to account the contribution of the concrete and reinforcement steel. The shear capacity of the wall was calculated with (22), (23) and (24). The shear demand V_u =0.338 kips obtained from ETABS is smaller than the shear capacity of the wall therefore ETABS shear wall design is adequate.

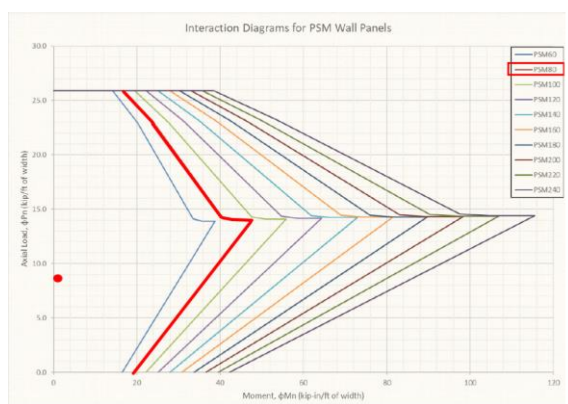

Figure 10

SCIPs Wall Interaction Diagram

Slab Verification

A critical slab has been selected for manual calculation verification. For the reinforced concrete structure, the slab span is 11.83 ft, and the design was based on a 12-in strip. The concrete cover for the slab is 1 inch. The reinforcement density required for the slab calculated using (25) and (26) is A_s = 0.416 in^2. The provided reinforcement area is greater than the minimum required by code of 0.2 square inches. As per ETABS model and design, an area of 0.4228 in^2 of reinforcement has been provided, therefore as compared to the manual calculations the model design for positive moment is adequate. For the negative moment, similar calculations were made and the required reinforcement area for negative moment is 0.723 in^2. As per ETABS negative moment reinforcement, an area of 0.7243 in^2 is provided. Therefore, according to the manual calculations, the reinforcement provided by the ETABS model design is adequate.

$$a = d - \sqrt{d^2 - \frac{2*M_u}{0.85*\phi*f'c*b}} \tag{25}$$

$$A_{sreq'd} = \frac{M}{\phi*fy*(d-\frac{a}{2})} \tag{26}$$

Maximum deflection permitted by code for live load and live plus dead load has been calculated and compared with maximum deflection from ETABS model and the results are shown on Table 10.

Table 10

Reinforced Concrete Floor Deflections

	Live Load	**Live + Dead Load**
IBC Limit	L/360	L/240
IBC Limit (in)	0.394	0.592
ETABS Max Deflection (in)	0.04	0.156

In the same manner as the concrete structure, for the SCIPs system a critical slab was selected for manual calculation verification. The slab span is 11.83 ft, and the design was based on a 12-in strip. The concrete cover for the slab is 1 in. The reinforcement area required for the slab calculated using (25) and (26) is A_s = 1.955 in^2. The provided reinforcement area is greater than the minimum required by code of 0.08 in^2. As per ETABS model and design, an area of 2.006 in^2 of reinforcement has been provided, therefore as compared to the manual calculations the model design is adequate. For the negative moment, similar calculations were made and the required reinforcement area for negative moment is 2.6 in^2. As per ETABS negative moment reinforcement, an area of 2.63 in^2 is provided. Therefore, according to the manual calculations, the reinforcement provided by the ETABS model design is adequate.

Maximum deflection permitted by code for live load and live plus dead load has been calculated and compared with maximum deflection from ETABS model and the results are shown on Table 11.

Table 11

SCIPS Floor Deflections

	Live Load	**Live + Dead Load**
IBC Limit	L/360	L/240
IBC Limit (in)	0.394	0.592
ETABS Max Deflection (in)	0.073	0.3

Grey Shell Cost Evaluation

A cost estimate was performed for both reinforced concrete and structural concrete insulated panels to compare the grey shell finished cost on both cases. Foundations, finishes, electricity, and plumbing costs were not included in this study since these elements are likely to be the same for both systems and will not add to the cost difference. It was found that the SCIP system structure cost 10.5% more than the reinforced concrete structure. Even though the SCIPs structure cost more than the reinforced concrete, there is other advantages during operation that might balance the difference. One of the differences is the thermal insulation provided by the wall panels and roof panels that will decrease the air conditioning load significantly. Other advantage to consider is that the manufacturer information points out that the construction with the SCIPS is around 40% faster than the traditional reinforced concrete construction that will result in additional savings during construction and a building that can be occupied earlier.

Table 12

Grey Shell Cost

Reinforced Concrete	$ 256,406.11
SCIPs	$ 286,364.00

Conclusions And Recommendations

After comparing the results of the reinforced concrete and SCIPs structures, it was found that both systems can perform under the code prescribed loads of gravity, wind, and seismic. However, the SCIPs system required double the length of shear wall to keep the story drift within the code restrictions. SCIPs system structure total weight is 60% as compared to the total weight of the reinforced concrete structure which could tend to point to lower lateral seismic forces if compared with the concrete structure. However, being more flexible, the lateral displacements were higher than the reinforced concrete structure which required a longer shear wall to control the lateral drift. Also, the additional flexibility could contribute to greater ductility of the SCIPs system compared to the reinforced concrete. The SCIPs system costs approximately 10.5% more than the reinforced concrete but has additional advantages that might counter act or balance the additional cost, such as the savings in energy bills due to the thermal insulation on wall and roof that will reduce drastically the size and consumption of the air conditioning units or heating units. Other advantages include the durability of the reinforcement because all wire mesh in all panels is hot dip galvanized, adding an additional layer of protection, and increasing the use life of the structure. There is also savings associated with a reduced duration of construction, that will reduce the labor time and equipment rental time.

A more precise model can be evaluated for the SCIPs system if on the finite element software, the wall and slab panels are modeled with the current configuration of the wire mesh layers with mortar and concrete. Also, the laboratory tests results presented on the technical evaluation report can be used to calibrate the elements on the model to obtain a more precise behavior. A soil-foundation interaction can be added to the model and analysis to consider the difference in the weight of the structure when using the SCIPs system.

References

[1] M2 EMMEDUE Advanced Building System, "Company and Values, Identity and History," *M2 EMMEDUE Advanced Building System* [Online]. Available: https://www.mdue.it/en/35-years-of-history/. [Accessed: Oct. 13, 2023].

[2] IBC International Building Code, 2018 International Code Council Inc, Country Club Hills, IL. [Online]. Available: https://codes.iccsafe.org/content/IBC2018P6. [Accessed: Sept. 15, 2023].

[3] ACI Committee 318, "Building Code Requirements for Structural Concrete ACI 318", *American Concrete Institute*, Farmintong Hills, MI, 2018.

[4] American Society of Civil Engineers, "Minimum Design Loads and Associated Criteria for Buildings and Other Structures", *ASCE7*, Reston, VI, 2016.

[5] Dr J Engineering, "Technical Evaluation Report TER 2110-01, Insulated Concrete Panels in Roof, Floor, and Wall Application," Madison, WI, Sept 2022.

[6] G. Lacayo, "Manual Técnico Sistema Constructivo Emmedue", *SUMINSA*, Managua, Nicaragua, 2014.