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Construction history of the composite framed tube structural system

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This paper examines the construction history of tall buildings designed during the latter portion of the twentieth century. Fazlur Khan was able to capitalize on the inherent strengths of steel and concrete by using them in conjunction with a framed tube system. This idea involves a novel construction process, which takes advantage of the virtues of structural steel and reinforced concrete. Early applications of this system in the mid-1960's were in the 35–story Gateway III Building in Chicago, Illinois and the 25 story CDC Building in Houston, Texas. Composite construction is now being used more frequently in the design of high rise buildings. This paper will look closely at the construction history of the 52–story One Shell Square Building in New Orleans, which was completed in 1971 and still stands as the tallest composite building in the world, at 700'. In One Shell Square, light steel framing is erected first. Temporary stays are used to position the steel members and then left in place for lateral resistance in the exposed steel frame. Placement of concrete filled metal deck follows closely behind erection of structural steel. Placement of rebar cages and forms around the perimeter steel columns and spandrels follows erection by 6 to 8 floors. Concrete is then cast to form a solid concrete tube around the light steel frame. The resulting structure has 40' of rentable floor space and minimal foundation requirements due to the light steel frame. The concrete perimeter provides exceptionally high lateral resistance, some thermal insulation, and a degree of fire protection for the primary lateral structural system. As a result, the potential applications for composite construction are numerous, especially with the renewed awareness of fire protection concerns in high-rise structures.

Study of such innovations allows us to see how they came into being, how individual engineers contributed to their success, and how the local construction constraints stimulated the search for such new ideas.

Introduction

Born in Dacca, Bangladesh, Fazlur Rahman Khan received his Bachelor of Engineering degree from the University of Dacca. Khan then taught at the University of Dacca for two years before coming to the United States. He studied at the University of Illinois, Champaign/Urbana, where he earned MS degrees in structural engineering and theoretical applied mechanics and a Ph.D. in structural engineering. Immediately after receiving his Ph.D. in 1955, Dr. Khan joined the Chicago office of the architectural/engineering firm of Skidmore, Owings & Merrill (SOM) and was made a general partner in charge of structural engineering in 1970. While working with SOM, Dr. Khan also taught and served as a research advisor at the Illinois Institute of Technology (IIT).[1]

Fazlur Khan introduced a series of innovations that changed the way engineers' viewed tall concrete building structures. He could not have done this without the close collaboration with his partners and colleagues at Skidmore, Owings, and Merrill. Especially the architect Bruce Graham and the architect-engineer Myron Goldsmith were crucial to Khan's success. Also there were excellent structural engineers who worked with him, most prominently Hal Iyengar and John Zils. Khan was also fortunate to be able to work closely with developers and builders as he thought through his new ideas. Fazlur Khan's experience of working with developers and builders allowed him to develop new systems. The resulting structures, which integrated engineering and construction practice, could be built economically. He died in mid-career with many plans uncompleted but he nevertheless achieved the status of structural artist whose works will be studied long after the twentieth century.[2]

Structural Tube System

In 1961, Dr. Fazlur Khan began to seek structural systems that would allow for construction of taller buildings without paying the «premium for height» which resulted from the wind loadings. He recognized that overcoming this premium would require new systems and new construction practices, in the tradition of the technology-oriented style commonly referred to as the Chicago School of Architecture.[3] This led him to the creation of the framed-tube, in which exterior closely spaced columns and deep spandrels provide the entire lateral resistance. The World Trade Center employs this scheme on a grand scale. The exterior columns have narrow spacing and the windows are recessed, creating the illusion of solid tubes.

Khan first used the framed-tube in the 43–story DeWitt Chestnut Apartment Building in Chicago, completed in 1965. Here, perimeter columns are spaced at 5.5 feet on centers and the spandrels between columns at each level are about 2 feet deep. The close column spacing expresses the idea of a solid tube perforated by holes that create the windows.

Structurally, the framed-tube is superior to a rigid frame because it places material on the exterior of the building, where it will contribute most to the moment of inertia and maximize the lateral stiffness of the building. The DeWitt Chestnut Building also has a relatively small floor plan, so by moving the lateral system from the core to the exterior of the building valuable floor space is freed. The entire interior structural system is secondary —designed to carry only gravity loads to the ground level.

In 1963 Khan started to contemplate a new system in which lateral forces were carried primarily by an interior shear wall core. The resulting 38–story Brunswick Building in Chicago was designed with a substantial concrete shear wall core. Much of the building's service machinery were also located in the core. Perimeter columns were placed at 9 feet 4 inches on centers and tied by deep spandrel beams at each floor. In making a quick check of the design calculations it became evident to Khan that the exterior framing was nearly as stiff as the shear wall core acting as a vertical cantilever.[4]

Khan had created an efficient system in which lateral loads are carried both by the exterior frame and interior shear wall core through shear wall-frame interaction. According to Khan, «One of the advantages of the shear wall frame interaction system is that it exists in every reinforced concrete building with shear walls, whether the frame is deliberately designed for it or not, simply because every joint in a reinforced concrete structure when cast monolithically acts as in a rigid frame».[5] In the Brunswick Building, this system also created 38–feet of unobstructed space by allowing the interior columns all to be placed in the building's core.

Composite Construction

By employing various structural systems in buildings of steel and buildings of reinforced concrete, Khan was able to gain a clear sense of the advantages and disadvantages of each material. He then developed a system to take advantage of a steel structure combined with one of a reinforced concrete. This is the basis for what became known as the SOM-Composite System.[6]

For framed tube structures the system consists of an exterior frame in reinforced concrete. The interior floor framing is erected entirely with structural steel. Concrete offers properties for inherent fire protection with out resorting to the separate cladding used in fire protected steel surfaces. Under normal service the concrete tube will serve as a layer of insulation causing a reduction of heating and cooling loads. The lateral stiffness of the concrete tube is often adequate to carry the wind loads, while the steel floor framing carries most of the gravity dead loads, except for the weight of the concrete tube itself.

This also avoids the disadvantages of concrete construction. Namely: interior concrete shear walls contribute greatly to the loss of floor efficiency and concrete construction is typically much slower than steel erection. In most high-rise structures, floor framing contributes to 75–80% of the total material in the structure.[7] Composite construction practices can result in construction cycles that are as fast as structural steel erection, demonstrated first in the 50–story CDC Building in Houston, Texas where the entire steel frame for five stories was erected before concrete casting commenced. This separated the two work forces necessary for the two materials and allowed a fast construction pace. Precast modular window framing units were attached during steel erection and then used as forms for the casting of the exterior concrete frame. The construction cycle was 3 days per floor for all 50 floors. Khan designed similar composite construction projects at One Shell Square in New Orleans and Union Station in Chicago where construction cycles ere on par with those for steel erection. Since normal concrete construction cycles can be in excess of 7 days per floor, the savings in time allowed by composite construction reduced cost as much as 33%.[8]

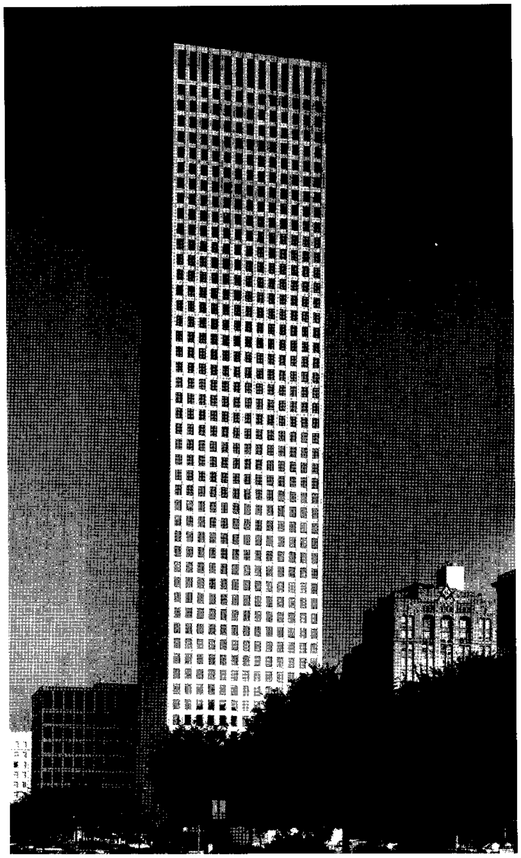

Figure 1
Facade of One Shell Square, New Orleans

One Shell Square

Completed in 1970, One Shell Square remains the tallest building in New Orleans at 52 stories. Fazlur Khan designed it for the developer Gerald D. Hines. One Shell Square is most interesting from a structural perspective because of innovative use of steel and concrete. The composite system that was developed by Khan is efficient because it capitalizes on the inherent strengths of each material. Material selection was driven by a desire to maximize the rentable floor space and overall stiffness under lateral loads while minimizing the dead weight and total cost per area of the building.

One Shell Square was designed with a composite structural system that combined steel floor framing and a steel core with concrete encased steel columns on the periphery of the building. Efficiency was gained through the use of steel in the floor frame because the ductile metal is approximately 9 times stronger than concrete and has a higher modulus of elasticity (29,000 ksi for steel vs. 3,200 ksi for concrete) so more compact sections could be used to achieve the same strength. Compact sections are essential in steel frame design because steel is a very dense material. However, well-designed steel floor beams are able to span long distances. In One Shell Square the floor-framing scheme allowed for 40' of column free space around the buildings braced steel core. Steel floor framing was also desirable because it could be erected quickly when compared to a concrete frame.[9] The World Trade Center Towers were marvelous examples of a framed tube made entirely of structural steel.

In very tall buildings, like One Shell Square, lateral loads dictate design. Lateral loads can result from seismic activity like an earthquake but in New Orleans the critical lateral loads applied to structures are wind. Structures must be designed with adequate stiffness to counteract the overstressing moment resulting from wind. A high premium would have to be paid for the additional material to stiffen a steel structure of this height. In the World Trade Center mass tuned dampers were used at each story to minimize lateral sway from wind, which contributed to the structure's $700 million price tag.

The use of reinforced concrete on the perimeter of One Shell Square allowed designers to overcome this premium. The stiff concrete columns and spandrels formed a braced tube, which provided lateral resistance for the entire structure. A concrete framed tube is ideal for composite systems because it places large heavy sections on the perimeter of the building, as far as possible from the centroid, where they will contribute most to the moment of inertia of the structure. The moment of inertia, modulus of elasticity, and unbraced length determine the stiffness of a structure. The perimeter composite columns of One Shell Square have a close spacing of 10'. This gives the spandrels a shorter unbraced length, further stiffening the structure against lateral loads.

Construction Sequencing

The construction sequencing of One Shell Square was an most innovative aspect of the composite design. Steel erection and concrete casting had to be kept separate if they were to occur on the same site. Each requires a specialized labor force and equipment. Interference between the two could cause complications and result in delays. To avert this problem construction of One Shell Square began with traditional steel frame erection.

Steel sections were positioned using temporary $^3/_4$" cable stays. The stays were left in place during construction of the steel frame to provide additional stiffness to the compact sections. In this initial phase the braced core carried all lateral loads so the perimeter columns were sized to carry minimal gravity loads. The largest steel section used on the perimeter was W8 × 67, which can be compared to the large W14 × 550 and built-up sections used in the core of the structure. Figure 2 shows the dimensions of one built-up column at the base of the structure. The core was stiff enough to allow erection of 10 to 12 stories of exposed steel. Placement of metal deck and casting of the floor slab follow closely behind erection of the frame.

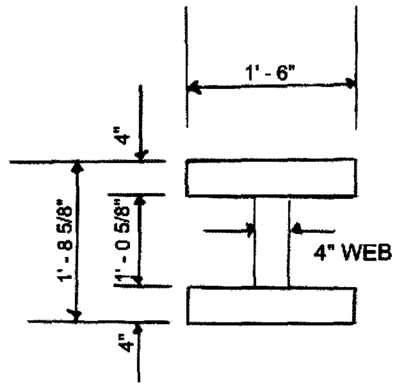

Figure 2
Construction detail of One Shell Square steel perimeter column

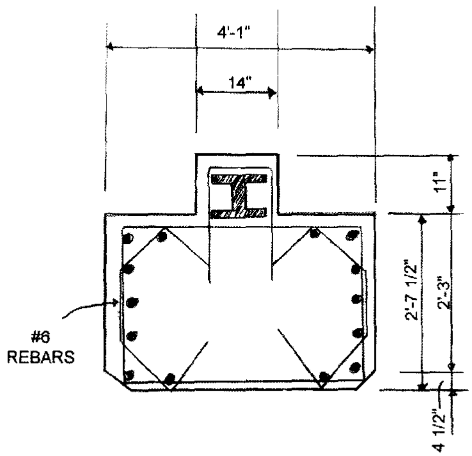

Figure 3
Construction detail of One Shell Square composite perimeter column

Once steel erection had progressed to a prescribed height of 9 stories, concreting of the building's perimeter began. Rebar cages and forms were attached directly to the exterior steel frame sections. Concrete was then poured into the forms, completely encasing the exterior light steel frame (Figure 3). Composite action begins once the columns become monolithic with the floor slab and spandrels. Casting of concrete and steel erection then proceeded simultaneously until both reach the completed building height of 52 stories.

Structural Analysis

The efficiency gained by the composite system can be easily observed through an analysis of the construction sequence. Construction was divided into 5 phases in order to identify critical points in the sequence. SAP2000 was used to create a 2–D model of the structure. Figure 4 shows the proportions of the SAP models that were used for the 5 phases of construction. The section properties and dimensions for the columns used in each model appear in Table 1. The model takes the equivalent moment of inertia, cross-sectional area, and dead weight from one bay of the buildings frame and applies them to a planar frame with equivalent properties.

The effective area of one bay is outlined in Figure 5, which shows the actual floor-framing plan for One Shell Square. Each construction phase was subjected to a combination of wind, dead, and live load. The output, however, has been formatted to show the building's response from each load separately. In this manner, it can be determined which load must have governed design. As expected the building's response to wind load was the most critical, so that is what is discussed here.

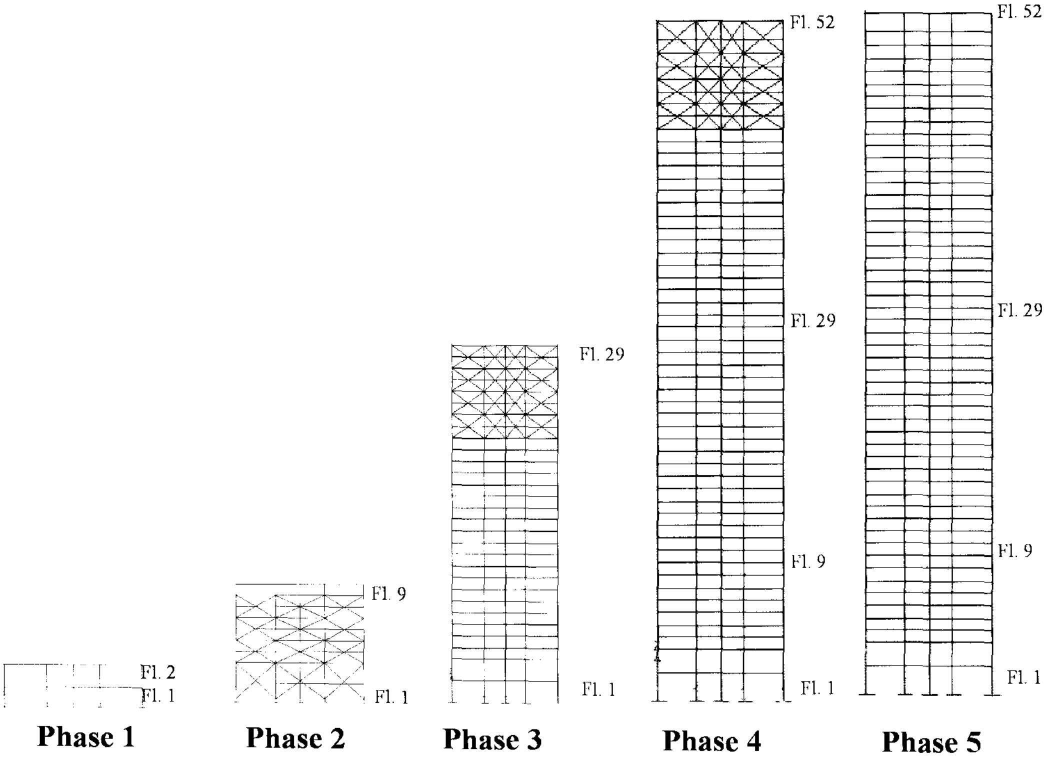

Figure 4
One Shell Square construction phases

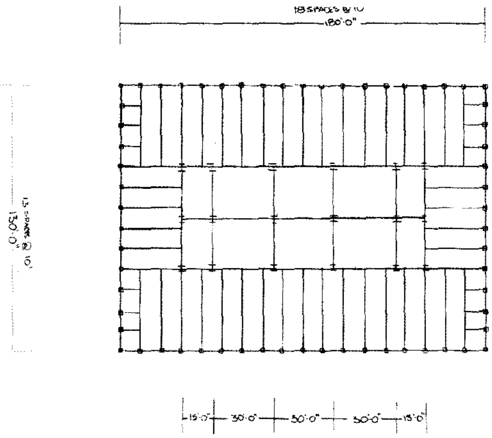

Figure 5
One Shell Square floor framing plan

The wind pressure profile used in this analysis was generated using the method prescribed by the American National Standard's Institute. In this method, wind pressure varies with wind velocity, which is a function of elevation. New Orleans falls in a region that is subject to hurricanes on a regular basis so a basic wind speed of 140 mph was used to determine wind pressure. The wind pressure for any given story was then multiplied by the total facing area of that story to obtain the design wind load. The following equation is used to determine the design wind pressure at all elevations;

$$p = q_z G C_f$$

where:

q_z = velocity pressure evaluated at height z
G = Gust factor based on exposure category
C_f = pressure coefficient

A gust factor (G) of .8 was used for this exposure category. The number is less than 1, indicating that the surrounding terrain will probably reduce, rather than amplify, the effect of the wind. The pressure coefficient C_f is based on the inclination of the roof. A value of 1 was used here. The velocity pressure changes with height. It is based on the expression:

$$q_z = .00256\, K_z V_x^2 I$$

where:

K_z = velocity pressure exposure coefficient
V_x = mean wind speed at roof height
I = importance factor

The mean wind speed at roof height was 140 mph. The importance factor was 1.0 for this multi-use structure and the velocity pressure exposure coefficient varied with height between .32 and 1.46. At the 29th story a K_z of 1.12 was used, resulting in a design pressure of 44.96 psf.

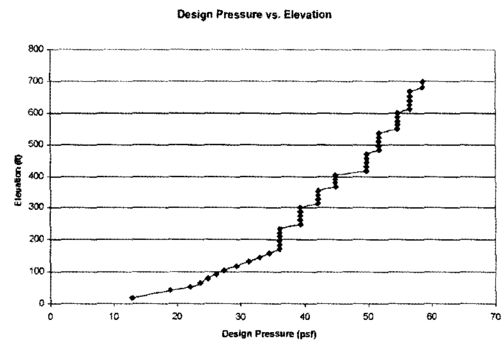

Figure 6
Design wind pressure profile

Wind Load Analysis for Construction Phases

The resulting wind pressure profile is shown in Figure 6. It is anticipated that wind will govern in the latter phases of construction when the building has reached its maximum height but still has exposed steel in the upper levels.

Phase 1

The first phase analyzed considers two floors of steel framing under wind, dead, and live load. Figure 7 shows a free body diagram of Phase 1 under wind load only. A joint on Column Line B has been circled in red. Moment equilibrium for this joint is checked in Figure 8. Note that the core columns (with the exception of the central column, which carries none

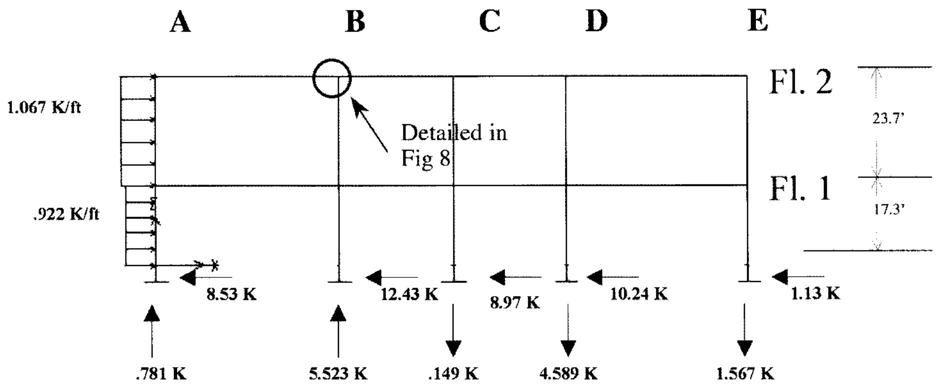

Figure 7
Construction phase 1 wind load analysis

of the wind load as an axial force) take a larger portion of the axial and shear load due to their high stiffness. The stiffest members will attract the greatest proportion of the load. In this early phase of construction design wind pressure is small and the height of the structure is low so the member stresses and displacements are well within the acceptable range for the material. Floor 1 has a height of 17.33' and Fl. 2 has a height of 23.73. The resulting horizontal wind load is then:

$$(1.067 \text{ K/ft} * 23.73') + (.922 \text{K/ft} * 17.33') = 41.3 \text{ K}$$

This is equal and opposite to the sum of the horizontal reactions:

$$(-8.53\text{K}-12.43\text{K}-8.97\text{K}-10.24\text{K}-1.13\text{K}) = -41.3\text{K}$$

So that:

$$\Sigma F_x = 41.3 \text{ K} - 41.3 \text{ K}$$

The structure does not project a large area to the wind with only 2 floors of exposed steel framing erected so it is not surprising that the deflections are very small. For tall structures The Uniform Building Code dictates that the lateral deflections must be less than L/360; where L is the total height of the structure. The floors in Phase I have a total height of 43.06 ft. so L/360 for this structure is 1.44 in. The actual maximum deflection is .1407 in. at joint 11. 55 ksi steel was used in the construction of One Shell Square. All the stresses are in the elastic range under this load.

$$\boxed{\begin{aligned}\Sigma M_z &= M_1 + M_2 + M_3 \\ &= (-3.19 - 34.4 + 37.4) \text{ K-ft} \\ &\approx 0\end{aligned}}$$

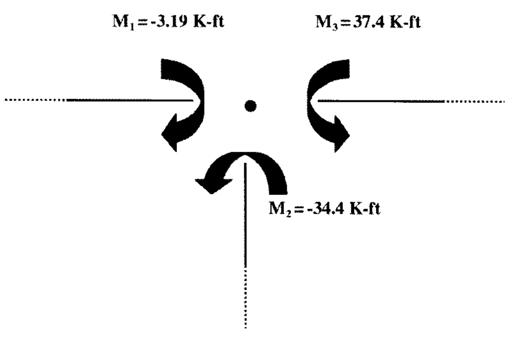

Figure 8
Moment equilibrium for joint in phase 1

Phase 2

Construction Phase 2 represents the limit of exposed steel erection in One Shell Square. After the 9th floor is erected, concreting of the external spandrels and columns begins. Temporary cable stays are used to position steel members and stiffen the exposed steel structure under lateral loads. Floor 3 of Phase 2 has a height of 13'. Floors 4 through 9 each have a height of 26'.

Phase II is a critical phase in the construction sequence because it is the phase during which the maximum amount of steel has been left exposed. The temporary cable stays are essential at this point to limit lateral deflections under the wind load. The lateral deflections of the columns are greatest in the windward columns and decrease slightly with increasing distance from the windward face. Phase II is 134 ft tall. L/360 is slightly less than 5 inches for this phase. A maximum horizontal deflection of 1.48» at the 9th floor occurred in Phase II. This is acceptable. The vertical deflections under wind are practically negligible

Figure 9
Construction photo of One Shell Square (phase 3)

Phase 3

Phase 3 signifies the point at which the structure begins to behave like a composite tube. Construction Phase 3 is a 29–story model of the building.

Figure 9 is a photograph of One Shell Square that was taken during this phase of construction. It is evident that the columns and spandrels of the bottom 21 floors have been encased in concrete and act as composite sections with large cross sectional area and moment of inertia. The resulting structure is stiffer than the first two phases, however the top 7 stories are exposed steel frame. This creates a more critical scenario than Phase 2 because the least stiff members in the system, the exposed steel frame, are at the top of the structure where wind loads are largest.

The reactions at the base of the structure reflect the high stiffness of the exterior columns. While the elastic modulus of concrete is less than that of steel, the moment of inertia of the composite perimeter columns is far greater than any of the other columns. The stiffer columns attract more of the lateral load in this framed tube. The fl» cable bracing becomes essential in this phase to limit deflections and insure that the structure remains safe in its vulnerable state. The equations of equilibrium can once again be used to verify that the base reactions are equivalent to the external wind load that has been applied to the structure. The vertical reactions due to wind load become more complex to calculate in this phase because the section properties of the columns vary greatly between the top and bottom of the structure.

One Shell Square begins to act like a composite tube at its base in Phase III. This is evident in the structures response. The lateral deflections in the lower levels of Phase III are actually less than the deflections in Phase II. At the second level, Phase III had a maximum horizontal deflection of .63 in., which can be compared to .72 in. in Phase II. Phase III is also a taller structure and projects a greater area to the wind so the overturning moment at the second level in Phase III (1,567.49 kip-ft) is nearly three times as large as the overturning moment in the same location in Phase II (554 kip-ft). The maximum deflection at Level 9 in Phase III was 1.61 in., which can be compared to 1.48 in. for Phase II.

In Phase II this deflection results from a bending moment of only 25 kip-ft, while in Phase III the structure is responding to a moment of 575 kip-ft. The stiff behavior in Phase III is directly attributable to the composite action of the columns and spandrels.

The axial stresses in the columns are also reduced as a result of the additional concrete in the lower levels. From Phase II to Phase III the maximum axial stress at level 2 dropped from .819 ksi to .148 ksi. The drop in stress is due to the far larger cross-sectional area of the composite section. The large cross-sectional area, while creating an efficient lateral system, is also necessary for carrying axial loads. Concrete is ultimately not as strong as steel so the stresses in the composite sections must be less than the stresses in the all steel section in order for the system to be safe and efficient.

Phase 4

In Phase 4 concreting has progressed 45 stories, leaving 8 stories of exposed steel at the very top of the structure where the highest wind loads act. This leaves One Shell Square highly vulnerable to lateral deflections in the upper levels, where only temporary stay cables brace exposed steel. The maximum lateral deflection in this phase of 19 in. occurs at the roof level. The structure is at its full height of 691 ft in this phase. The criterion of L/360 provides a maximum deflection of 23 in. Under hurricane force winds, the exposed steel at the top of the structure will not exceed this deflection criterion.

The transition from composite sections to exposed steel framing occurs at the 45th floor. As a result, the axial stresses at this level are higher than at any other point in the model at 1.9 ksi, under wind load only (Table 1). This is still well below the ultimate strength of steel. Note that the maximum stresses that have been highlighted on the other levels are lower than this one. This indicates the effectiveness of the composite system.

Phase 5: Full Composite Structure

Phase 5 is not actually a construction phase —but a model of the completed structure. In this model, the

Table 1. Member force summary for phase IV under wind load

CONSTRUCTION PHASE	FLOOR	COL. LINE	FRAME	LOAD	P (Kip)	Area (in ^{2})	Axial Stress (ksi)	V_x (Kip)	M_z (Kip-ft)
PHASE IV	1	A	1	WIND	3269.32	5908.00	**0.553**	**310.31**	**4470.34**
	1	B	53	WIND	3.91	270.46	0.014	194.31	1776.36
	1	C	105	WIND	0.04	218.50	0.000	143.25	1294.02
	1	D	157	WIND	-4.24	270.46	-0.016	193.82	1772.35
	1	E	209	WIND	-3269.03	5908.00	-0.553	300.71	4424.38
	2	A	2	WIND	3106.55	5908.00	**0.526**	**287.36**	**3273.95**
	2	B	54	WIND	43.56	270.46	0.161	206.61	2463.70
	2	C	106	WIND	-0.11	218.50	-0.001	150.96	1794.71
	2	D	158	WIND	-43.10	270.46	-0.159	206.59	2463.13
	2	E	210	WIND	-3106.90	5908.00	-0.526	274.91	3237.89
	9	A	9	WIND	2318.69	5908.00	**0.392**	202.80	**1282.90**
	9	B	61	WIND	53.19	270.46	0.197	**215.60**	1404.66
	9	C	113	WIND	0.00	218.50	0.000	177.44	1153.08
	9	D	165	WIND	-53.21	270.46	-0.197	215.58	1404.53
	9	E	217	WIND	-2318.66	5908.00	-0.392	193.93	1263.75
	45	A	45	WIND	55.35	47.20	1.173	21.78	90.56
	45	B	97	WIND	65.75	35.00	**1.878**	30.79	229.45
	45	C	149	WIND	-0.04	32.70	-0.001	**34.50**	**242.61**
	45	D	201	WIND	-64.94	35.00	-1.855	30.44	227.05
	45	E	253	WIND	-55.93	47.20	-1.185	8.94	61.13
	52	A	52	WIND	-0.38	47.20	-0.008	**2.83**	**6.75**
	52	B	104	WIND	5.19	17.90	**0.290**	-0.22	-1.83
	52	C	156	WIND	-0.67	17.90	-0.037	0.62	4.26
	52	D	208	WIND	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

Table 2. Member force summary for phase V under wind load

CONSTRUCTION PHASE	FLOOR	COL. LINE	FRAME	LOAD	P (Kip)	Area (in ^{2})	Axial Stress (ksi)	V_x (Kip)	M_z (Kip-ft)
PHASE V (FULL COMP.)	1	A	1	WIND	3269.37	5908.00	**0.553**	**310.31**	**4470.34**
	1	B	53	WIND	3.79	270.46	0.014	194.31	1776.36
	1	C	105	WIND	0.04	218.50	0.000	143.25	1294.03
	1	D	157	WIND	-4.13	270.46	-0.015	193.82	1772.36
	1	E	209	WIND	-3269.07	5908.00	-0.553	300.71	4424.38
	2	A	2	WIND	3106.60	5908.00	0.526	287.34	3273.94
	2	B	54	WIND	43.44	270.46	0.161	206.61	2463.71
	2	C	106	WIND	-0.12	218.50	-0.001	150.96	1794.72
	2	D	158	WIND	-42.99	270.46	-0.159	206.59	2463.13
	2	E	210	WIND	-3106.94	5908.00	-0.526	274.91	3237.88
	9	A	9	WIND	2318.78	5908.00	0.392	202.79	1282.83
	9	B	61	WIND	52.99	270.46	0.196	215.60	1404.68
	9	C	113	WIND	-0.01	218.50	0.000	177.45	1153.16
	9	D	165	WIND	-53.03	270.46	-0.196	215.58	1404.56
	9	E	217	WIND	-2318.73	5908.00	-0.392	193.92	1263.69
	44	A	45	WIND	74.32	5908.00	0.013	85.18	444.08
	44	B	97	WIND	33.46	35.00	0.956	21.16	137.49
	44	C	149	WIND	0.09	32.70	0.003	22.71	147.97
	44	D	201	WIND	-33.33	35.00	-0.952	21.16	137.44
	44	E	253	WIND	-74.54	5908.00	-0.013	72.64	416.53
	52	A	52	WIND	-3.07	5908.00	-0.001	26.13	41.33
	52	B	104	WIND	12.75	17.90	0.713	1.88	18.03
	52	C	156	WIND	-0.19	17.90	-0.011	2.54	24.18
	52	D	[illegible]	WIND	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

exterior columns and spandrels have been completely encased in concrete for the entire structure. No exposed steel framing remains on the exterior of the structure and all of the perimeter elements act compositely.

Nearly the entire lateral load is carried in the stiff perimeter columns. This is the behavior that Fazlur Khan was seeking in this framed tube design. This is the most stable model, with the lowest element stresses and displacements at the top of the structure. The maximum displacements at level 45 have dropped 3 inches from Phase 4. This number also considers the fact that the composite structure projects a large area to the wind, resulting in a higher bending moment. The maximum moment at roof level in Phase 4 was 6.8 kip-ft (Table 1). The maximum moment at roof level in Phase 5 was 6 times larger at 41 kip-ft (Table 2). It will be observed that both values are of lower magnitude because the stiffness and cross-sectional area have both been increased significantly with the addition of concrete encasement.

Phase 5 shows clearly that the composite structural system is efficient under lateral loads. The success of One Shell Square as a structure in the New Orleans's windy environment emphasizes the effectiveness of the composite system.

Conclusion

The design innovations discussed in this thesis represent the works of the last century that allowed tall buildings to become skyscrapers without paying an exorbitant premium for height. The innovations represent the creative genius of engineers like Fazlur Khan, Myron Goldsmith, and Leslie Robertson who devised structural systems that had functional superiority and were financially competitive.

The structural tube form that all of these designers are masters of, provides lateral resistance through closely —spaced large columns and spandrels on the perimeter of the building. This creates a tube, which is stiff enough to resist bending in all directions.

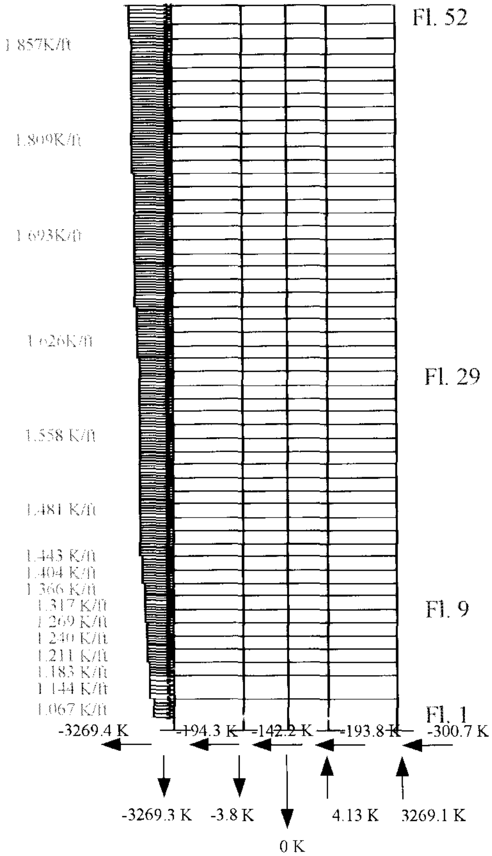

Figure 10
Construction phase 5 wind load analysis

Some framed-tube structures also have a core within the exterior tube to carry some portion of the lateral load or gravity load. The stiffness of any member is proportional to its moment of inertia and inversely proportional to its unbraced length. The framed tube gives short spans or unbraced lengths to the perimeter columns and spandrels, resulting in a high stiffness. The fact that the tube is on the perimeter of the building means that it is as far as possible from the centroid of the structure and has the highest moment of inertia. The Brunswick Building, World Trade Center, and Dewitt-Chestnut Apartments are early examples of framed-tube structures.

In the late 1960's another advancement was made when Fazlur Khan designed One Shell Square in New Orleans. The structure was to be the tallest framed-tube ever but it was also to be one of the first structures to directly benefit from the strengths of both steel and concrete. Steel would be used for its ease of erection and ability to span long distances in floor framing. A steel core was erected and a lightweight frame was assembled around it. Cable stays were used to position the steel and left in place for temporary wind support. The core carried all lateral loads during construction while sharing gravity loads with the perimeter columns.

After erection of 8 to 10 floors the perimeter columns and spandrels were encased in concrete. Concrete is a lighter material than steel so larger cross-sections can be used for sections before self-weight becomes a limiting factor. Concrete sections can therefore have larger moments of inertia than steel sections of the same weight. Placing these large sections far from the centroid of a structure serves to further increase their moment of inertia and their contribution to the stiffness of the structure as a whole.

The combination of these design innovations has allowed engineers to build structures more quickly that are taller and more cost effective. Composite construction not only fully utilizes the strengths of steel and concrete —it minimizes their weaknesses, while the framed-tube system provides higher stiffness than is possible in a traditional frame. These innovations have truly allowed us to overcome the premium for height. Study of such innovations allows us to see how they came into being, how individual engineers contributed to their success, and how the local construction constraints stimulated the search for such new ideas.

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