Investigation for the Removal of Steel Tie Rods in a Historic Segmental Arch Floor

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ABSTRACT

Gilsanz Murray Steficek, LLP investigated the removal of the tie rods in the floors of the landmarked Metropolitan Life Tower in New York City when the tower was converted for residential use. The typical floor is constructed of segmental concrete arches supported on steel beams with tie rods perpendicular to the beams below the arches. GMS evaluated the removal of the rods by performing linear and nonlinear 3D finite element analysis of the floor system. Load tests were also performed at 5 locations in the building in order to confirm that the floor would perform satisfactorily. Both the analysis and the load test confirmed that the tie-rods could be removed safely.

INTRODUCTION

The Leviev Clock Tower is a 50-story steel frame tower in New York City. Originally built as an office tower for the Metropolitan Life Insurance Company Tower, it was the tallest building in the world when it first opened in 1909. The landmark tower is being converted into a luxury condominium under the direction of new ownership. The building's typical floor system consists of segmental concrete floor arches tied with steel rods. At the request of the owner, Gilsanz Murray Steficek, LLP (GMS) investigated the possibility of removing the tie rods to accommodate the proposed architectural design. As part of the structural system, the tie rods would require fireproofing, which would reduce the ceiling height at every floor and increase the cost of the project. Through the use of finite element analysis and the code required load tests, GMS was able to confirm the safe performance of the floors under the new loadings.

DESCRIPTION OF STRUCTURAL SYSTEM

The typical floor consists of tied segmental cinder concrete arches spanning to beams spaced at 4' to 5'. The arches are 4" thick at mid-span, and at least 8" at the face of the beam. At the bottom of the arch is a perforated metal mesh form with metal strips. ¾" diameter tie rods are installed in the framing bay at approximately 6 feet to 7 feet on center perpendicular to the beams. The steel framing of the floors consists 10" to 20" beams and 15" to 24" girders. The framing is supported by built up column, laid out in a rectangular grid spaced at 14' to 25'. Two inches on cement finish is on top of all the floors as a wearing surface. Figure 1 provides an illustration of the floor system.

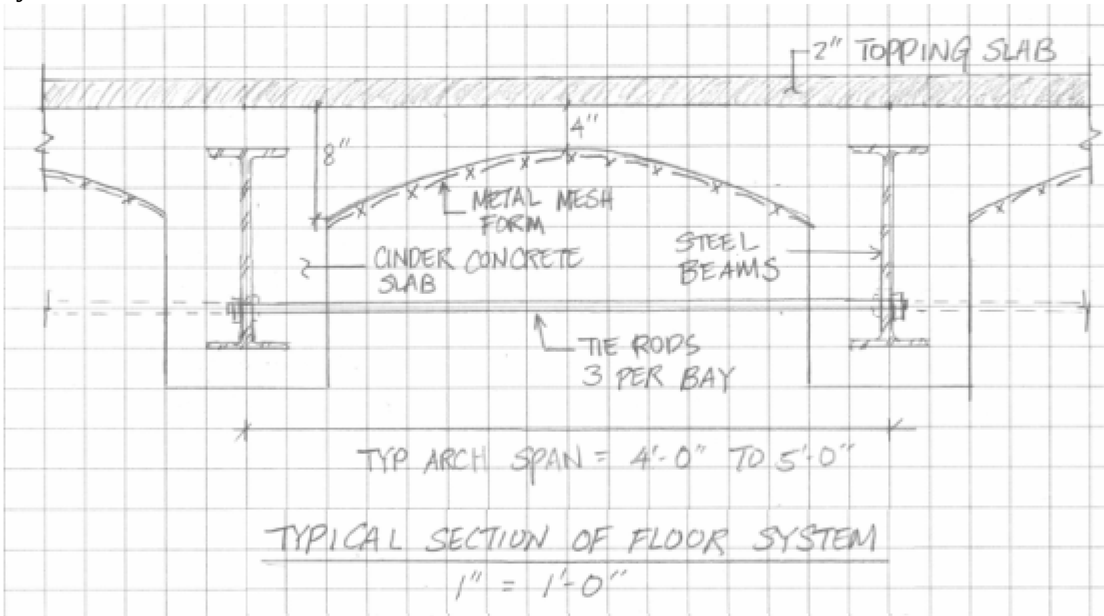

Figure 1: Typical Section of the Floor System

In a tied arch system, the tie rods served two purposes: to provide stability to the steel framing during construction while concrete is poured, and to resist the lateral thrust of the concrete arch. While the tie rods are necessary in the classical tied arch model, GMS observed that the rods are slack or have been cut at several locations in the building due to their interference with existing risers. At these locations, no visible distress is observed in the floor system, suggesting that the tie rods may not be required.

The lateral system of the building consists of a perimeter knee brace frame with 24” to 36” built up girders. At the lower portion of the building, twin plate girders are used to resist the higher cumulative lateral loadings. All steel framing is encased in cinder concrete for fireproofing.

The concrete and steel properties are shown in Table 1.

Table 1: Material Properties

			Design Stress (psi)		
Material	Weight (lb/ft^3)	E (ksi)	f'c	fr	fys
Cinder Concrete	108	860	600	110	
A9 Steel	490	29000			30000

THE LOAD TEST

The test was performed under a two stage loading criteria in accordance with the New York City Building Code (NYCBC) section 27-599 b “Load tests for completed construction,” which specifies both a strength requirement and a deflection requirement. The strength requirement consists of two testing stages.

For the first stage loading, the applied load shall equal 100% of the superimposed dead load (35psf) and 100% of the live load (40psf). At this stage, the deflection is not to "exceed that permitted deflection in the applicable reference standard." For the typical floor framing, the industry accepted deflection standard is L/240 in the gravity direction and, for masonry construction, L/600 in the lateral direction.

For the second stage loading, the load is increased to 150% of the total dead load (100psf self-weight and 35psf superimposed dead load) plus 180% of the live load (40psf). The equivalent superimposed load for this stage is 175psf. This load was maintained on the floor for a minimum of 24 hour to ensure safety under a factored overload condition. In this stage, the residual deflection after removal of load "shall not exceed 25% of the calculated elastic deflection under the superimposed test load." After each stage of the loading, the test areas are inspected for signs of serious distress indicative of a potential failure.

Photo 1 shows the typical test set up.

Photo 1: Typical Test Set Up

THE TEST AREAS

Five representative test areas were selected. All the tested areas were located at the exterior bays of the building, where impact of removing the tie rods would be the most pronounced. As the thrust in the arch would be resisted by the edge framing once the tie rod is removed, the test areas included each of the three different edge conditions in the building. Each test area consisted of a floor bay bounded by column grid lines and the three floor arches. When choosing the test areas, the bays with the smallest girders were typically chosen since they would have the least reserve capacity. A characteristic of each test area is shown in Table 2.

Table 2: Characteristics of the Test Areas

Floor	Side	Bay location	Bay Span	Rows of Rods	Edge condition
7th	West	Corner (N)	25'-1"	3	Twin Girder
9th	West	Side	20'-4"	2	Twin Girder
25th	East	Side	20'-4"	2	Beam + Girder
28th	East	Corner (N)	25'-1"	3	Beam + Girder
37th	West	Side	20'-4"	2	Single Girder

ANALYSIS

A 3-D model of the 28th floor test area was created in SAP2000 to simulate the behavior of the floor during the load test. The bay is framed by a steel wind girder on the exterior edge, and steel edge beams on north and south sides. The typical floor beams span from north to south and are modeled with pins and rollers to represent their simple span condition without the transmission of residual axial forces. The beams are restrained at the web to represent a riveted connection which ties the beam ends to the girders. As shown in Figure 2, the coordinate is defined such that the x-axis is perpendicular to the steel beams and the y-axis is parallel to the beams. Figure 3 is a diagram of the test bay floor plan.

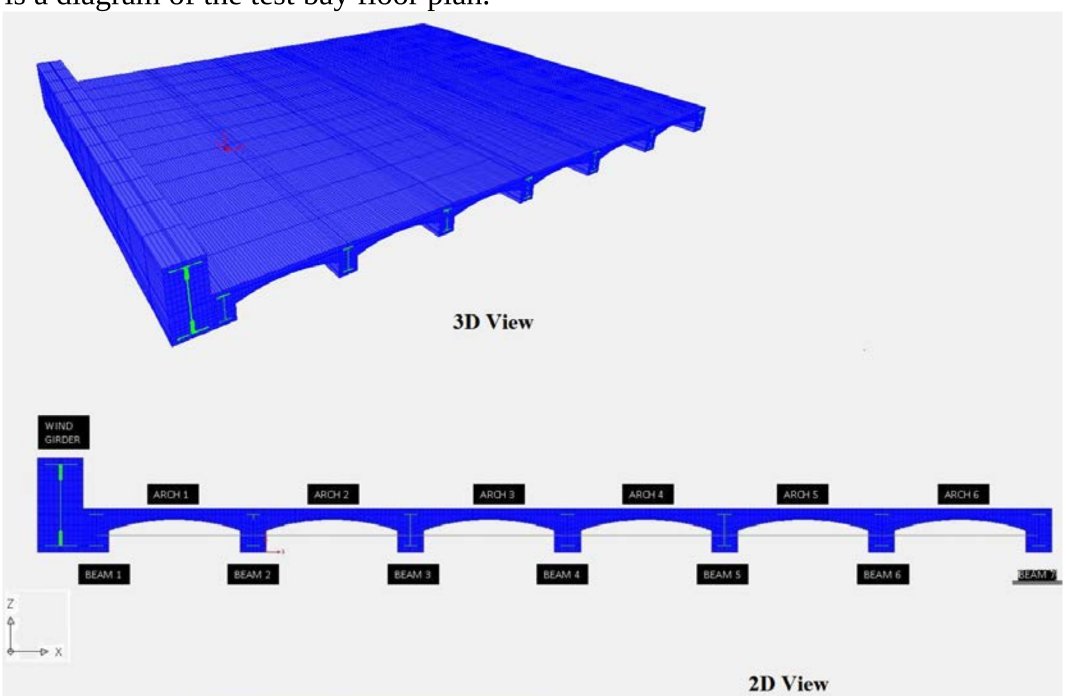

Figure 2: Analysis Model of the 28th floor test area

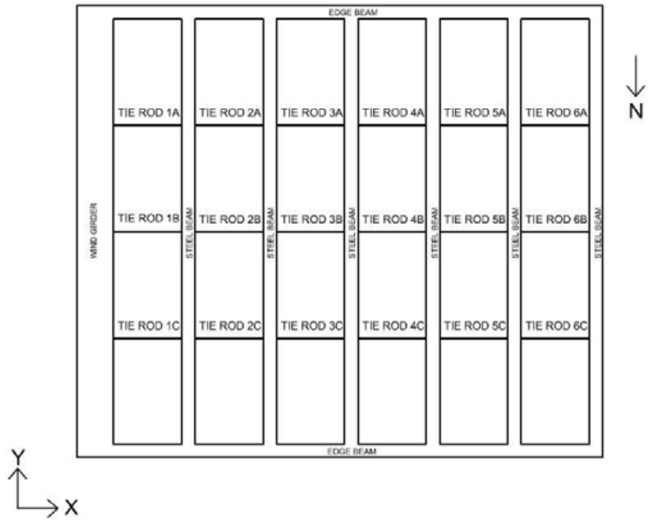

Figure 3: Plan of test bay at the 28th Floor

The cinder concrete arches and steel beams are represented by 3d solid elements. The cross-sectional area of each element is approximately 1.5" by 1.5". The tie-rods are modeled using frame elements and are pinned at both ends. The metal strips and wire mesh on the underside of the arches are modeled as membrane elements. The 2" cement topping is not considered. The mesh sizes were fine-tuned to minimize analysis time while maintaining the accuracy of the results.

The analysis is divided into 3 steps using separate models to represent the different states of the floor system. The results are then superimposed to obtain the final stresses and deflections.

1. Concrete Hardening. This step calculates the stresses in the tie-rods and metal strips induced by the dead weight of the wet concrete. As the concrete is not yet self-supporting, its weight is carried completely by the wire mesh form and the rods. At the end of this step, the concrete is unstressed, the tie rods are in tension, and the metal form, which is in the shape of an arch, is in compression. The tie-rod forces in this stage are tabulated in Table 3.

Table 3: Tie-Rod Forces

Tie Rod	Tension Force (kip)	Tie Rod	Tension Force (kip)	Tie Rod	Tension Force (kip)
1A	1.36	1B	1.77	1C	1.35
2A	1.38	2B	1.91	2C	1.37
3A	.82	3B	1.13	3C	.81
4A	.67	4B	.89	4C	.66
5A	.86	5B	1.12	5C	.85
6A	1.12	6B	1.26	6C	1.11

2. Removal of Tie-rods. This step captures the change in stresses in the floor as a result of removing the tie-rods. First, the tie-rods are removed from the model, and the forces in the rods are added back to the model as point loads. The model is checked to ensure no stress change occurred. The point loads are

then removed from the model. The measured change in stress in the concrete and the steel is less than 0.01 ksi, which is negligible on the overall result.

3. Superimposed Loading. This step simulates the behavior of the floor under stage 1 and 2 load test after the tie-rods are removed. The deflections calculated in this step represent the expected deflections measured during the load test. The stresses from this step, when superimposed with the stresses found in steps 1 and 2, represent the final stresses in the floor. Both linear and nonlinear analyses are performed to account for cracking in the unreinforced cinder concrete. The results are presented below.

Linear analysis. In the linear-elastic model, cracking is modeled by reducing the EI of cinder concrete, incrementally from 100%EI to 20%EI. The deflections obtained are shown in Figure 4. The increase in deflection is not linearly proportional to the reduction of EI, as shown in Figure 5, since the strength of the steel elements are constant.

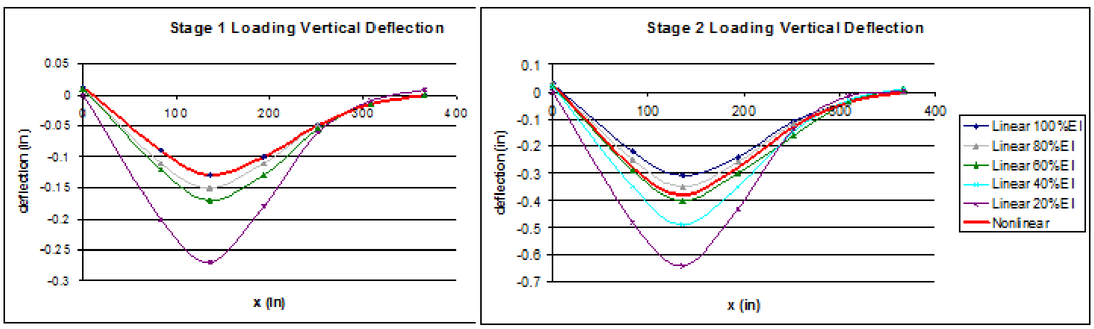

Figure 4: Predicted Deflection under Stage 1 and Stage 2 Loading

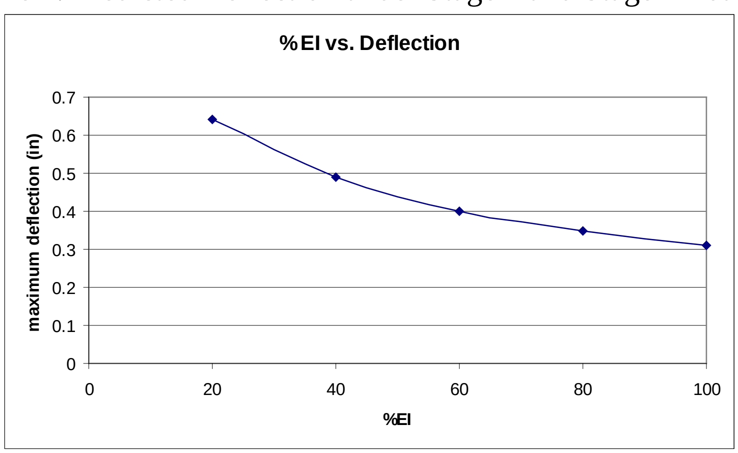

Figure 5: Relationship between Deflection and EI of Cinder Concrete

The lateral displacement of the wind girder is 0.01” under stage loads and 0.03” under stage 2 loads.

Nonlinear analysis. A nonlinear analysis was done to better approximate the effect of cracking. The linear model (with 100% EI) was used as a starting point, and in regions where the concrete stress exceeded the cracking stress of 110 psi, the concrete elements are considered “cracked,” and their elastic modulus was reduced to 5% of the uncracked modulus. The analysis was then performed again on the cracked model. The stresses and deflections typically converge after 5 iterations.

Under Stage 1 loading, cracking is observed only at the bottom face of Beam 3 at mid-span, and results from the cracked analysis are within 1% of those from the elastic analysis. (See Figure 1 for the numbering of the beams and arches.) Under Stage 2 loading, cracking is observed at the bottom face of three beams and at the three arches closest to the wind girder. Figure 6 illustrates the degree of cracking observed. The cracked vertical deflection is 123% of the elastic deflection, and the cracked lateral deflection at the wind girder is 133% of the elastic deflection. Interpolating from the linear analysis results, these deflections correspond to using 68% effective EI. The maximum lateral deflection, which occurs at the mid-span of the wind girder, is 0.01” under Stage 1 loads and 0.04” under Stage 2 loads. Figures 7 and 8 show the deflections under stage 2 loads.

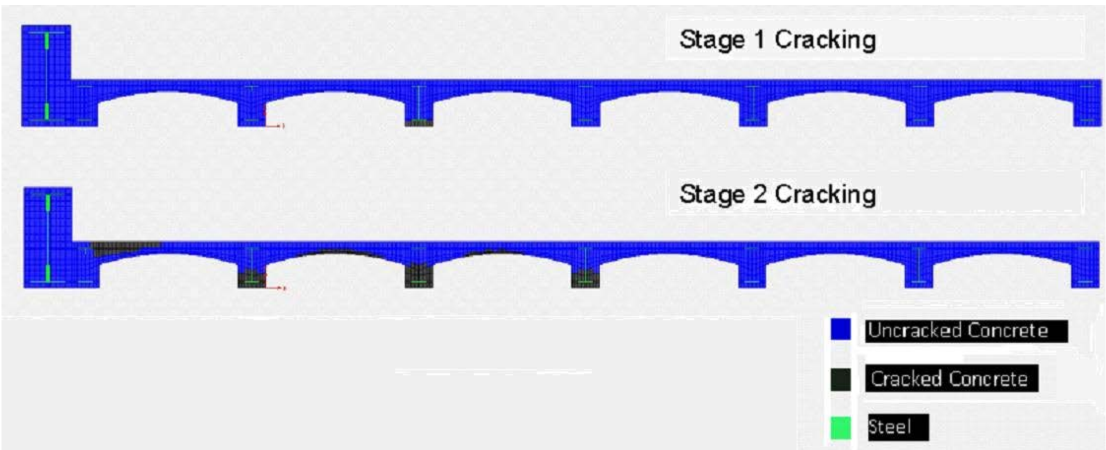

Figure 6: Cracking Observed at Stage 1 and Stage 2

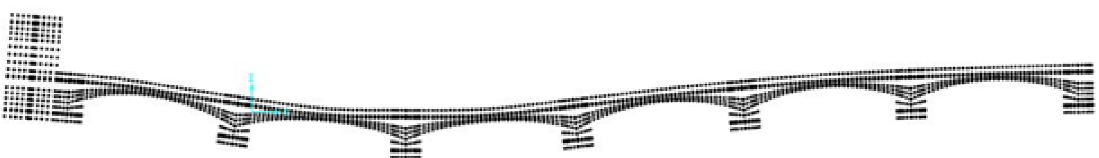

Figure 7: Deflected Shape in X-direction at Stage 2, magnified 50x for illustration

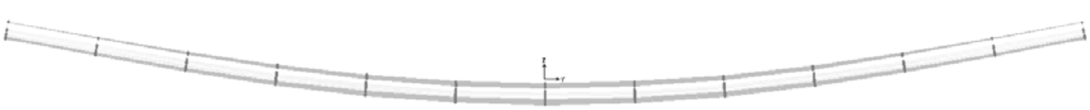

Figure 8: Deflected Shape in Y-direction at Stage 2, magnified 50x for illustration

The deflected shape shows that the floor system is acting with two way action, similar to the behavior as a two-way slab. The moment is largest at the mid-span of Beam 3, which is where the first crack occurs. Under both loading conditions, the concrete does not fail in compression and the steel stresses are always below the maximum allowable stress. The maximum stresses are tabulated in Table 4.

Table 4: Maximum stress in structural elements under stage 1 and stage 2 loads

Member	Max. Stress under Stage 1 Loading		Max. Stress under Stage 2 Loading	
	Tension	Compression	Tension	Compression
Concrete	0.11 ksi	-0.12 ksi	0.11 ksi	-0.35 ksi
Steel Beam	8.1 ksi	-4.4 ksi	14 ksi	-7.0 ksi
Wind Girder	2.4 ksi	-2.8 ksi	3.6 ksi	-3.6 ksi
Metal Strip	4.6 ksi	-3.9 ksi	12 ksi	-7.0 ksi

To assess the difference in behavior with and without tie rods, the behavior of the floor with tie rods was also analyzed. The difference in maximum deflections, both vertical and lateral, is within 5% of each other for both Stage 1 and 2 loads, which shows that removing the tie rods will not significantly change the behavior of the floor slab. Under Stage 1 loading, the maximum tension in the tie rod is 3.7 kips. Under Stage 2, the maximum tension is 7.3 kips.

LOAD TEST PROCEDURES

The test areas were prepared by installing shoring frames underneath test areas and screw jacks under the floor below to engage an additional floor level in case of any serious structural distress. Dial gages with a precision of .001" were installed to measure the horizontal and vertical displacements of the floor under the loaded condition. The gages were placed to measure the vertical deflection of the floor at mid-span of the beams and arches and the horizontal deflection at each of the tie rod locations.

Gages and rods were epoxied to the underside the concrete encasement. After the gages were installed and zeroed, the tension rods were loosened by pulling the beam flanges until the rod is slack. The transferred tension could be measured in a dynamometer attached to the pulling rig. Once the rod was slack, the nuts at the end of the rods were loosened to allow a deflection of at least a half inch. This would allow the beams and the floors deflect without the influence of the rod up to a certain limit. If this limit was exceeded, the rod would engage to prevent excessive deflections.

The loading was applied using an equivalent weight in concrete masonry units. The CMU was placed in layers on the floor area with the stage 1 loading applied in two layers of CMU and the stage 2 loading applied in up to 4 layers. The weight of the masonry was first determined using information from the material supplier, and we verified the weight by field sampling the units that were delivered. The verified weight was used to determine the number of layers of masonry to be placed. Each layer of masonry was a uniform loading of 48 lbs per square foot. The last layer of masonry was a partial layer to match the target weight as close as possible. With the unloaded condition set to zero, measurements were taken at the addition of each layer of CMU. The repeated measurement allowed us to monitor the floor as it was progressively loaded and to ensure that the floor was deflecting in a stable manner and within the specified limits.

LOAD TEST DATA SUMMARY

The floor system was able to support the stage one and stage two loadings without the tie rods in all five of the tested areas. The maximum measured deflections are shown in Tables 5 through 10.

Table 5: Maximum vertical deflections observed during Stage 1 Load Test

Floor	Deflection	Span to deflection ratio	Maximum Allowable deflection (L/360)	Results
7th	.082”	L/3600	.825”	Pass
9th	.066”	L/3700	.678”	Pass
25th	.054”	L/4500	.678”	Pass
28th	.110”	L/2700	.825”	Pass
37th	.067”	L/3600	.678”	Pass

Table 6: Maximum lateral deflections observed during Stage 1 Load Test

Floor	Deflection	Span to deflection ratio	Maximum Allowable deflection
7th	.025”	L/11900	.3”
9th	.014”	L/17400	.3”
25th	.023”	L/10600	.3”
28th	.050”	L/5900	.3”
37th	.014’	L/17400	.3”

Table 7: Maximum vertical deflections observed during Stage 2 Load Test

Floor	Deflection	Span to deflection ratio
7th	.174”	L/1700
9th	.116”	L/2100
25th	.114”	L/2100
28th	.190”	L/1600
37th	.116”	L/2100

Table 8: Maximum lateral deflections observed during Stage 2 Load Test

Floor	Deflection	Span to deflection ratio	Maximum Allowable deflection
7th	.041”	L/7200	.3”
9th	.034”	L/7200	.3”
25th	.039”	L/6300	.3”
28th	.067”	L/4400	.3”
37th	.024”	L/8400	.3”

Table 9: Maximum vertical deflections observed after removal of the load

Floor	Set	Calculated elastic deflection	Max. Allow Set.	
7th	.010”	.902”	.226”	Pass
9th	.019”	.512”	.128”	Pass
25th	.021”	.512”	.128”	Pass
28th	.009”	.902	.226”	Pass
37th	.011”	.512”	.128”	Pass

Table 10: Maximum lateral deflections observed after removal of the load

Floor	Set	Elastic deflection	Max. Set.	
7th	.014”	.495”	.124”	Pass
9th	.009”	.407”	.102”	Pass
25th	.021”	.407”	.102”	Pass
28th	.020”	.495”	.124”	Pass
37th	.011”	.402”	.102”	Pass

It should be noted that the maximum deflection of approximately 1/50” is negligible relative to the span.

No signs of severe distress were observed in the floor system after load removal. The load test showed that the requirements set by the New York City Building Code were met.

COMPARISON OF ANALYSIS PREDICATION AND LOAD TEST RESULTS

The floor deflections were less than those predicted by the analysis in the vertical direction. At stage 1, the load test vertical deflection is approximately 85% of the deflection predicted by the analysis model, and at stage 2, the deflection is roughly 50% of the deflection predicated by analysis. The load test results suggested that an additional effect is at work to increase the stiffness of the floor. In the finite element analysis, we noted that the floor system is showing stress both in the direction of the arch span and in the direction parallel to the arches, which suggested that the floor system is acting as a two way slab instead of a one-way arch system. It is also possible that the cement finish on the floor may have some bond with the underlying concrete, and thus contributed to the stiffness of the floor. However, the exact contribution cannot be quantified as the topping slab was most likely poured after the concrete is set, and the bond strength between the topping and the structural slab is not known. The topping slab was not considered in the of the computer analysis.

The lateral deflections observed in the load test were larger than those predicted by the analysis model. However, both the predicted and the observed deflections were insignificant compared to the allowable deflections and the span.

CONCLUSION

The finite element models showed that the behavior of the floor slab is similar regardless of the presence of the tie rods. The load tests showed that without the tie rods, the floor system was able to meet the strength and deflection requirements of the New York City Building Code. The lateral movement of the slab was monitored during the load test to ensure that the façade would not be damaged due to the lateral movement after the tie rods were removed, and the movement was found to be minimal. Based on these results, we concluded that the ties rods were required only during construction to support the wet concrete, and that it was acceptable to remove the rods after the concrete has set.

REFERENCES

NYC Building Code Committee (1968), *The City of New York Building Code 1968*, New York City, NY.