Vertical Structural Deformation

Estimation and Control for a Deformation-Sensitive Building

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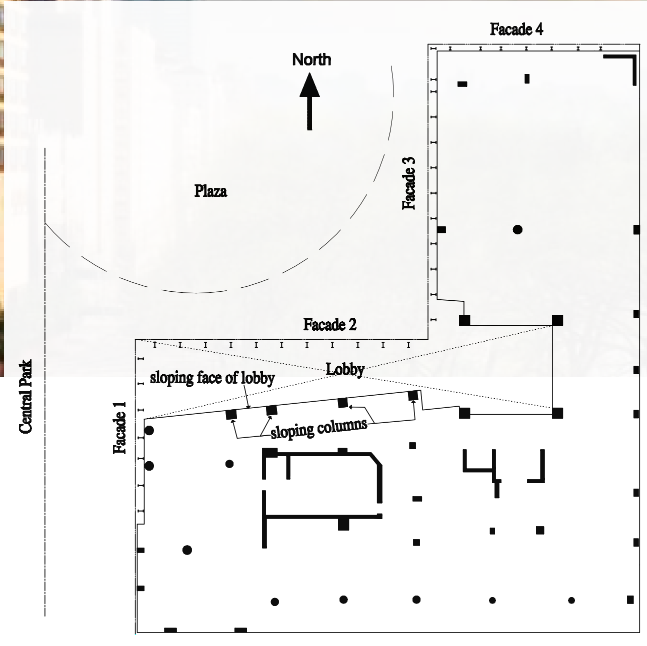

Figure 1: The building plan at museum level. Note that the steel façade 1-4 are only guided laterally by the concrete structure.

Located on Fifth Avenue at East 110th Street in New York City, the Museum for African Art is a new addition to the Fifth Avenue Museum Mile. Its L-shaped plan cradles a plaza facing west at the north-east corner of Central Park (*Figure 1*). The Museum occupies the four lower levels (one below the ground floor) of a 19-floor reinforced concrete flat-slab residential tower, and includes a lobby 75 feet tall, a 245-seat staged theater, over 15,000 square feet of exhibition galleries and a third floor year-round roof garden. To accommodate the architectural space requirements for the museum, a portion of the 16 residential floors is picked up by one-story-high cantilever concrete beams at the 4th floor. The cantilevered beams are supported by sloping concrete columns which define the museum lobby space. The west and north exterior walls of the museum are steel space frames with a maximum plan length of 100 feet and a maximum height of 75 feet (*Figure 1*). The building façade system is a combination of story-high nonstructural precast concrete panels and glass windows, which are partially supported by the reinforced concrete structure and partially by the steel frames enclosing the museum space. Due to the different time-dependent properties of the two supporting structures and deformation sensitivity of the building façade system, sophisticated control of the vertical time-dependent relative movement between the two structures is needed to ensure the building serviceability performance. The size of the soft joint at the 4th floor precast panels and the cantilever concrete beams camber are challenging tasks for the design firm of this project.

Factors Affecting Vertical Relative Movements between the Two Structures

While the cantilever beam camber depends on the total concrete beam downward deflections (short-term and long-term), the precast panel soft joint size at the 4th floor is controlled by the future relative movements between the concrete structure and the steel frames after the panels are installed. The main factors affecting the relative movements between the two different panel-supporting structures are: (1) temperature-induced expansion/contraction of the steel frames; (2) differential foundation settlements; (3) effect of lateral loading; (4) short- and long-term concrete column axial deformation; (5) tilting of cantilever beams due to differential concrete column short- and long-term axial shortening; and (6) short- and long-term concrete beam bending deflections. All the above effects were tackled in the design process in order to preserve the integrity of the steel supported façade system which required lateral bracings from the concrete structure.

Temperature effect

The steel frames at the exterior walls are subjected to temperature fluctuations and member elongation/contraction can result. The reinforced concrete structure is inside the building envelope and no significant temperature variations are expected during the building service life. Based on engineering experience in New York City area, a temperature fluctuation range of ±50F° for the steel mullion is assumed, and the corresponding temperature-induced horizontal and vertical relative movements between steel mullion and concrete structure can be obtained accordingly.

Differential Foundation Settlement

The sub soil condition for the building is of low bearing capacity to depths of over 100 feet and differential foundation settlement must be kept to an acceptable level. The steel frames are sitting on 15-foot tall foundation walls with spread footings, and tower columns are supported by steel pipe friction pile foundation (14-inch diameter), which are driven 90 feet into the lower glacial till bearing stratum. Proportioning of the foundations was carried out to limit the differential foundation settlements between columns to a minimum. Regular survey monitoring during the construction by a third party indicated that the differential foundation settlements between the two supporting structures described above have been appropriately accommodated for by the design team.

Lateral loading effect

Interaction between the steel frames and the concrete structure is expected when the building is under wind or seismic excitation. A finite element model was developed with the ETABS software to estimate the relative movements of the two structures under service wind or seismic loading. In the FEM structural model, the shear walls at the lower floors were assumed cracked based on tension stress level and an equivalent beam model for flat-slab structure, considering the magnitude of lateral loading, was used.

Column axial shortening and associated beam tilting

As indicated in *Figure 2*, a portion of the 16 residential floors above is picked up by concrete beams with varying cantilever spans at the 4th floor. As a result, the compressive stresses in the rear columns/walls supporting the cantilever beams are less than those in the front sloping columns. The difference of the compressive stresses tends to yield different column axial shortening, which will be gradually enlarged at a slow rate by creep and shrinkage over long periods of time. The effects of column axial shortening are twofold in this project: one is the downward translational movements of the beams that equal to the smaller accumulated axial shortening of two supporting columns; the other is the tilting of the cantilever beams resulted from column differential axial shortening.

The ACI 209R-92 model for creep and shrinkage was used for column long-term axial shortening estimation due to its widespread popularity among design engineers. The concrete mixture proportion was provided by the concrete supply contractor of this project for creep and shrinkage estimate inputs. Annual average ambient relative humidity for New York City area is taken as 70% based on PCI Design Handbook; loading schemes were initially assumed considering common construction cycle and modified later according to the actual construction sequence.

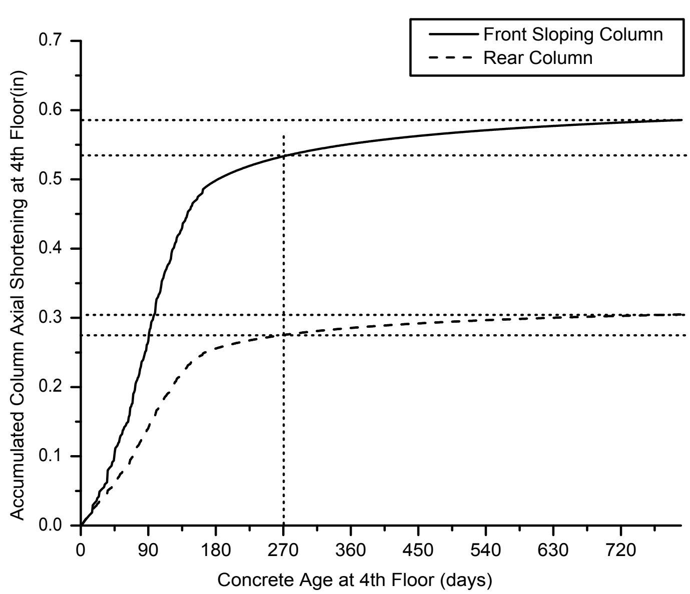

Figure 3: Variation of column axial shortening estimates for one concrete cantilevered girder at the 4th floor.

Figure 2: Sloping columns and cantilever beams under construction (prior to the installation of exterior steel frames and the façade system).

The dead loads on a particular column are applied floor-by-floor as construction progresses; the live load and most of the superimposed dead load will not be applied until the entire building is occupied. To obtain the sequential gravity loads for each column involved below the 4th floor, construction sequential analysis was carried out with the FEM structural model. The axial shortening of one column is the summation of the effects of each individual floor load on the column based on the linear superposition assumption in creep and shrinkage prediction. For serviceability limit states involving creep and shrinkage-induced long-term effect, ASCE-7 suggests a service load combination as D + 0.5L, in which D is the service dead load, and L the design live load. The load combination was used to predict the long-term deformation of the concrete columns and beams.

Beam bending deflection

The traditional procedure specified in ACI-318 was followed for the estimates of the cantilever beam bending deflections. The curve of long-term deflection multiplier varying with load duration shown in the figure R9.5.2.5 in ACI-318 (2005) was used to estimate the time-dependent beam bending deflections after the installation of precast panels at the steel mullion.

Deflection and Relative Movement Control

The cambers of the cantilever concrete beams at the 4th floor are the summation of the time-dependent total cantilevered beam bending deflections and the downward movement due to cantilevered beam tilting as a result of column different axial shortening. The picked-up floor super elevation is set the same as the beam camber specified at the 4th floor to maintain the designed story height, and the floors above are built to compensate for the stepped downward deflection to level off the floors in the long run. Although the maximum deflections vary with cantilever beam spans, the maximum camber was set as 0.5 inches at the tip of all the cantilever beams for construction practicalities. To ensure that concrete strength reaches the design value, temporary shoring of the cantilever beams at the 4th floor was not removed until 40 days after the concrete placement at the 4th floor.

The precast panel soft-joint size at the 4th floor is governed by summation of the relative movements between the two panel-supporting structures after the panel installation. In order to reduce the size of the panel soft joints, the installation of the precast concrete panels at the steel mullion was arranged to a later construction stage so that larger portions of the long-term downward deflection of the cantilevered concrete beams occurred before the panel installation at the steel supported facade. The actual concrete panel installation at the steel mullion was carried out six months after the concrete roof was placed, when the concrete age at the 4th floor reached 9 months. To show the effectiveness of the construction arrangement, the estimates of accumulated axial column shortening at the 4th floor for the supporting concrete columns of one cantilever concrete beam are shown in *Figure 3*. The variation of the accumulated axial shortening with concrete age indicates that most of the column axial shortening occurs within the first 6 months, and the additional long-term accumulated concrete column axial shortening estimates after the precast panel installation at the steel mullion (the concrete age at the 4th floor is about 270 days) are much smaller: 1/12 inch for the front sloping column and 1/25 inches for the rear column. It is well known that there are large inherent uncertainties in concrete long-term deformation predictions, and the coefficients of variation can be up to 30% and more. To account for uncertainties, two standard derivations (with assumed 30% coefficient of variation) were added to the estimated additional long-term deformation values in the determination of panel soft-joint size. Combining all the associated effects (temperature variance, foundation differential settlement, additional creep and shrinkage of the concrete members and construction inaccuracy), the soft-joint size at the 4th floor was therefore set as 2.0 inches. This dimension can accommodate a maximum future relative movement of 1.0 inch.

The steel mullion of the façade is laterally supported by the concrete structure. In order to allow relative movements between the façade steel frame mullion members and the concrete structure supports, special "sleeve" connections with neoprene bearing pads were designed to allow free relative vertical movement and provide supports for the steel mullion in the horizontal directions.

Survey monitoring was carried out for this project with emphasis on differential foundation settlements and the cantilevered survey beam deflections at the 4th floor. *Figure 4* shows the measured beam total deflection at the 4th floor and the associated regression curve for the same cantilever beam in *Figure 3*. Although additional deflections at the cantilever beam are expected, the survey data indicate that the long-term deflections will be relatively small. This gradual stabilization of the long-term deflection nine months after concrete placement indicates the appropriateness of the soft joint at the 4th floor to accommodate the future relative movements between two panel-supporting structures. The survey data also indicate that the actual elevation at the 4th floor cantilever beams nine months after the concrete placement is generally close to the original floor design elevation. The elevation difference is within ±0.32 inches, which are in acceptable engineering range considering the uncertainties in short- and long-term deformation estimation, possible construction error and survey error.

Summary

In this deformation-sensitive project, there are several main factors affecting the relative movements between the steel frames and the adjacent concrete structure: temperature-induced elongation/contraction, movements due to wind/seismic loading, differential foundation settlements, differential column axial shortening and associated cantilever beam tilting, beam bending deflection, construction inaccuracy, etc. All the effects are considered in the design process in order to specify reasonable beam cambers and precast panel soft-joint size at the 4th floor. The precast panels supported by the steel faced mullions

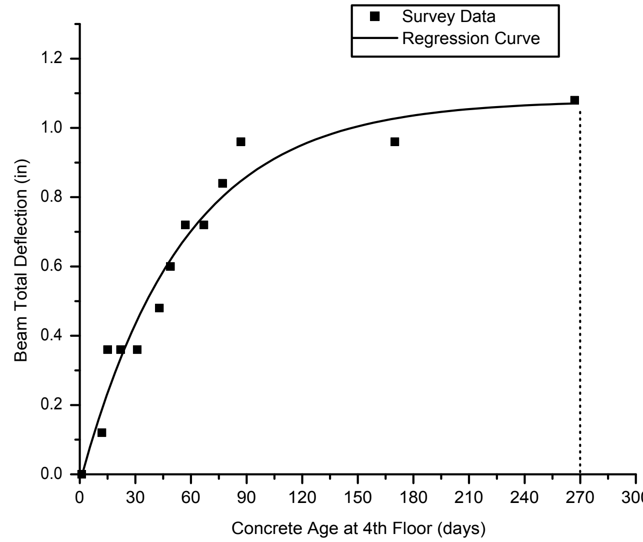

Figure 4: Measured total deflection at the tip of one concrete cantilevered girder and the regression curve.

below the 4th floor were intentionally installed at a later construction stage to reduce the long-term concrete beam downward relative movements after the panel installation. The special "sleeve" connections with neoprene pads were designed to allow the relative vertical movements between the steel frames and concrete structures.

Due to the inherent uncertainties of deformation estimation and possible construction error, survey monitoring during the construction period was carried out for this deformation-sensitive building to ensure serviceability performance. As shown, with prediction based on ACI 209R-92 model and actual survey data, most of the long-term deformation occurred within the first 6 months. The survey data indicate that the proposed cantilever camber successfully leveled off the beam and the picked-up floors above, and that the proposed soft joint of the precast panels at the 4th floor can appropriately accommodate the relative future movements of the two structures.▪

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