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Correcting Design Deficiencies in a New Five-Story Building

Correction des faiblesses du projet d'un bâtiment de cinq étages

Korrektur von Konstruktionsmängeln eines neuen Bürogebäudes

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SUMMARY

A five-story office building was investigated to assess its structural integrity immediately following construction, prior to occupancy. Each floor level consisted typically of two-way post-tensioned concrete slabs, except the fourth level which was built in reinforced concrete. Significant structural deficiencies were identified. Some elements had little or no reserve against failure at normal service loads. A proposed load test was judged to be inappropriate, as it would have likely resulted in failure of structural elements. Repair and strengthening measures involving various methods and materials were engineered.

RÉSUMÉ

Un bâtiment de cinq étages a été contrôlé pour vérifier l'intégrité de la structure avant son occupation. Chaque étage est constitué d'une dalle précontrainte, excepté le quatrième étage construit en béton armé. Une étude compréhensive a identifié les points faibles de la structure. Certains éléments de cette structure ont peu ou pas de réserve de résistance ultime aux charges de service. Un essai de charge proposé a été refusé car il aurait pro-bablement conduit à la ruine des éléments de la dite structure. La réparation et le renfor-cement, réalisés à l'aide de plusieurs méthodes et différents matériaux, sont présentés.

ZUSAMMENFASSUNG

Ein fünfgeschossiges Bürogebäude wurde direkt nach Errichtung und vor der Inbetriebnahme auf seine statische Zuverlässigkeit untersucht. Die Geschossdecken bestanden aus in zwei Richtungen vorgespannten Platten. Nur die vierte Geschossdecke war in gewöhnlichem Stahlbeton ausgeführt. Ausführliche Berechnungen deuteten auf erhebliche Unzulänglichkeiten des Tragwerksystems hin. Einige Bauteile wiesen wenige oder gar keine Reservekapazitäten gegen Versagen selbst unter Gebrauchslasten auf. Der Vorschlag einer Probebelastung wurde verworfen, da diese leicht zum Versagen einiger Strukturelemente führen konnte. Reparatur- und Verstärkungsmassnahmen mit verschiedenen Baustoffen und Verfahren wurden entwickelt.

1. INTRODUCTION

The structure of a new five story office building was almost finished when the Architect became concerned about its adequacy and the trustworthiness of the Structural Engineer of record. In quest of a second opinion, the Architect contacted a well known consultant, recognized as an expert in prestressed concrete, and requested a limited inspection and evaluation of the structure under construction. The Consultant issued a report listing a number of design-related structural deficiencies, including some on the fourth and fifth floors, which would render the structure unsafe.

Realizing that the need of protracted, in depth engineering assistance, to resolve the problems encountered and to design the strengthening measures required, could most practically and efficiently be provided by a consultant located nearby, the Architect was referred to, and engaged, Schupack Suarez Engineers, Inc. (SSE).

Initially, the objective of SSE's investigation was to confirm the first Consultant's findings regarding the inadequacy of the structural design of the fourth and fifth floors. Later, this was expanded to also assess the structural adequacy of the entire building and to engineer retrofit strengthening measures where the structure was found to be substantially deficient.

2. STRUCTURE DESCRIPTION

Fig. 1 represents the general structural framing of a quadrant of the original as-designed building. The roof, fifth and fourth floor slabs stepped back for architectural purposes. The third, second and first structural levels (not shown) have supporting columns around the perimeter of the building.

The roof and all structural levels, except the fourth, are two-way post-tensioned

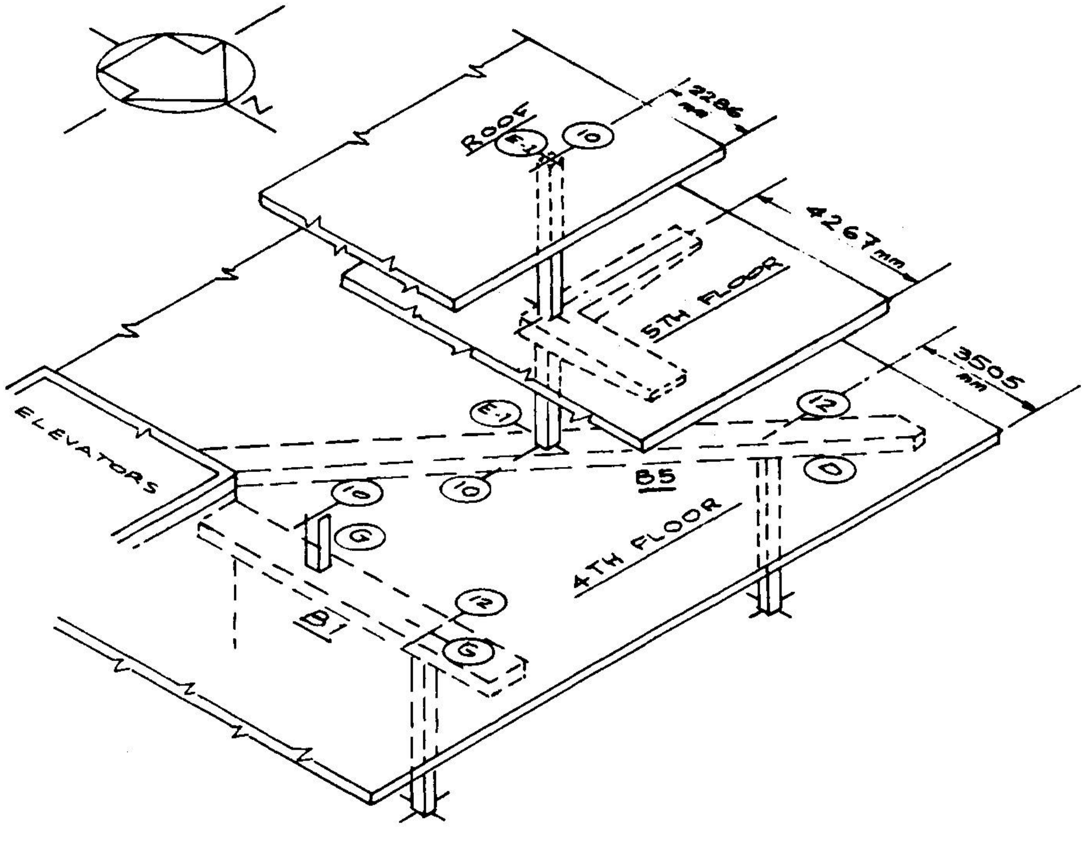

Fig. 1 Schematic view at corner of structure showing upper floors.

flat plates. Typically, based on US current practice, the post-tensioning strands are uniformly spaced in one direction (north-south) and banded along column lines in the orthogonal direction (east-west). Not typical of current practice was the use of relatively long cantilever overhangs of the roof and fifth level slabs, and the short beams cantilevered from the exterior columns, composite with the fifth floor slab.

The fourth level was constructed as a conventionally reinforced concrete slab and beam frame. As is seen in Fig. 1, the exterior perimeter columns supporting the roof and fifth levels did not extend below the fourth level but, instead, were supported on and transferred their loads to shallow, reinforced concrete transfer girders.

There is a three level parking garage structure which is north of and contiguous to the office building, and separated from it by expansion joints. It has been excluded from this report because of space constraints.

3. ANALYSIS

The behavior of two-way flat-plate reinforced or prestressed concrete slab systems, with rather uniform column spacings, is fairly well understood. They can be analyzed by using any one of a number of available "approximate" methods. This is not the case of the fifth and fourth level framings of the structure in question, considering the unusual cantilever beams and shallow transfer girders, as well as the use of relatively large slab overhangs, which make the load path distribution much more complex. Therefore, "approximate" methods of analysis were not considered appropriate.

By using linear elastic finite element computer analysis techniques, and explicitly modeling the beams coupled to the slabs, a requisite assumption regarding load distribution was not necessary. In this way, the actual load distribution to slabs, beams, columns and walls would be more rigorously determined, implicitly accounting for the most effective load carrying capability of the as-designed structure. The computer results were verified with approximate hand computations.

Based on the analysis results, concrete components were checked against rated capacity as per code. Where deficiencies were pinpointed and could not be rationalized as structurally acceptable, strengthening was judged to be required. After the strengthening details were formulated, the component structure was re-analyzed to ensure general compliance to the building code.

4. MOST SIGNIFICANT FINDINGS

4.1 Fifth Floor

4.1.1 Deficiencies

The principal deficiencies determined by analysis were: the cantilevered slab/beam assemblies were under-designed and had excessive deflections; most of the supporting columns, especially the perimeter ones supporting the overhangs, were also found to be significantly under-designed.

Fig. 2A shows the deflected shape, as determined by analysis, of the fifth floor slab system, as originally designed, under full service load. It is quite apparent that the slab deflections, especially those of the cantilever overhangs, are excessive, possibly implying that the slab is overstressed. There was some visual evidence of this overstress in the form of slab top and column cracking. Fortunately, the fact that the cantilevered slabs had remained shored all along must have limited the amount and size of cracking observed.

It was estimated that some portions of the cantilevered slab and some peripheral

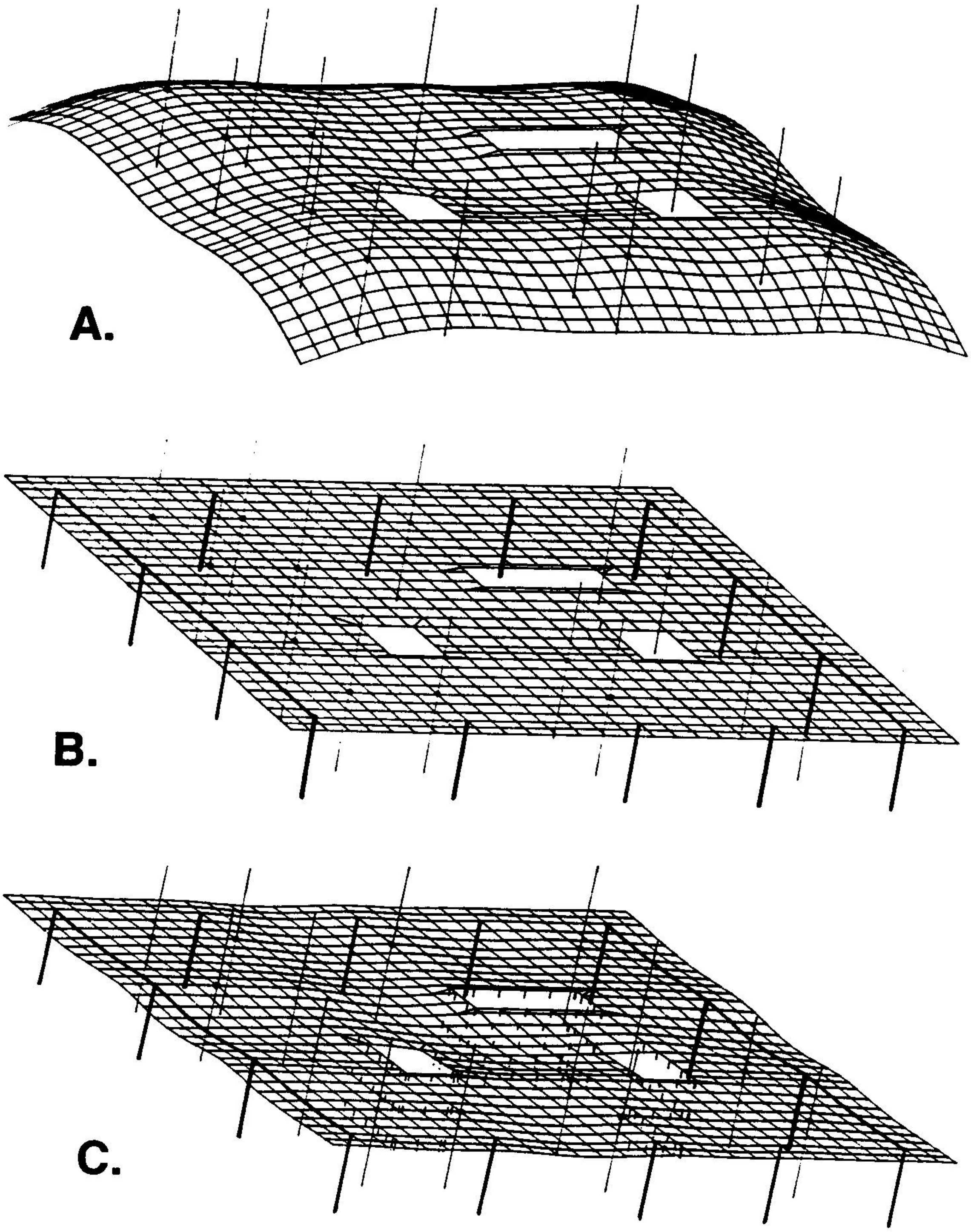

Fig. 2 Finite element models of fifth floor two way posttensioned slab system:

A. Deflected shape under full service loads as originally designed (scale factor = 50).
B. Location of added steel beams and perimeter columns (shown bold on undeflected model).
C. Deflected shape under full service load with added steel beams and perimeter columns (scale factor = 50).

columns would possibly have failed had full service loads ever been achieved. Therefore, a code load test, as requested by the Engineer, was judged to be inappropriate as it would have likely resulted in permanent damage to or partial failure of the structure.

4.1.2 Strengthening Scheme

The major strengthening was accomplished by adding a new line of steel columns (in effect, "permanent" shores) around the periphery of the floor to reduce the slab cantilever (see Fig. 2B). A steel beam was placed on top of the columns parallel to the north and south slab edges in order to control the cantilever deflections in the uniform spaced tendons direction. The new steel beams were preloaded by shimming to predetermined beam deflections, in order to pick up, at least, part of the dead load. The steel columns on the east and west sides were shimmed directly under the short cantilever beams. All the new steel columns sit directly on top of the periphery concrete columns supporting the fourth floor. Consequently, the loads picked up by them is directly transferred down the structure to the footings. This strengthening scheme was chosen because of its relative ease and speed of implementation, and its relatively minor disruption of lease space.

Fig. 2C represents the deflected shape of the strengthened fifth floor under full service loads. Comparing this figure with Fig. 2A, it is readily evident that the cantilever slab deflection and, thus, tensile stresses, have been substantially reduced. The latter, as shown by analysis, appear to be within allowable code levels.

Significant reduction in the cantilever slab stresses and moments resulted in a correspondingly significant reduction of bending moments transferred to the concrete columns.

Where analysis indicated several local regions of slab bottom overstress, these areas were locally strengthened by adding supplemental reinforcing steel bonded to the slab soffit with latex modified shotcrete. A slab top area near the elevator core which analysis indicated was overstressed, since no top steel was called for in the original design, was strengthened by placing reinforcing bars in slots cut into the slab and bonding the bar to the slab with latex modified mortar.

4.2 Fourth Floor

4.2.1 Deficiencies

The principal design-related deficiencies found were: the shallow, reinforced concrete transfer girders, as well as a number of their supporting columns were significantly under-designed; some local slab areas were found to be overstressed (in tension). It is not known why the fourth floor slab was designed in conventional reinforced concrete, when post-tensioning could have been particularly beneficial.

Probing hand calculations made initially by SSE had confirmed the findings of the original Consultant that the transfer girders and slab were highly deficient; so, in order to expedite the design of the necessary strengthening, a full analysis of the as-designed fourth floor was never carried out. The computer analysis actually done contemplated the changes already designed for the fifth floor, including the effect of the loads from the new supporting steel columns. In essence, some of the load which would have been placed on the fourth floor beam/slab system, as originally designed, by-pass the floor and go directly down the building through the steel columns, to the columns and footings below. Even with this improved load distribution, the transfer beams and slab and many of the peripheral concrete columns supporting the fourth floor cantilever were still found structurally deficient.

Evidence of the probable distress which could be anticipated in the transfer girders, under full service loads, was the observed extensive flexural cracking under dead load only, principally under the columns supported by the girders. The magnitude of the cracking of the transfer girders, under partial dead load and relatively light construction loads, clearly indicated a lack of structural strength.

The columns below, supporting the transfer girders, also had evidence of distress under reduced dead load conditions in the form of flexural cracking. Because of the heavy concentrated column load coming down at about midspan of the transfer girders, the top of the supporting columns would bend in towards the center of the building, with cracking thus expected on the outside faces of the columns. This cracking was clearly evident under partial dead load conditions only. Excessive deflections and failure of some structural elements might have occurred had full service loads ever been achieved.

Fig. 3 External post-tensioning of transfer girders and steel collars confining column tops

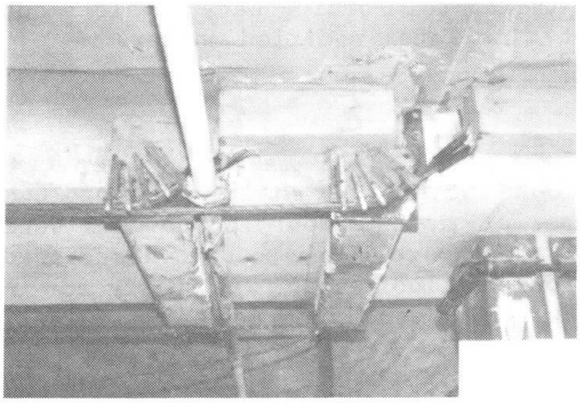

Fig. 4 Close-up of external post-tensioning tendon deviation saddles at transfer girder

4.2.2 Strengthening Scheme

As previously discussed, the analysis of the fourth floor revealed significant structural deficiencies in the transfer girders, slab, and in 7 out of the 16 concrete columns supporting the fourth floor.

The most practical solution to retrofit the transfer girders was determined to be the use of external post-tensioning (see Fig. 3). To fully resolve all structural deficiencies resulting from the analysis would have been impractical. Therefore, a re-analysis of the fourth floor allowing for plastic deformation of the concrete (the creation of hinges) at factored load conditions was carried out. In this way, moments were redistributed to girder and slab regions which would be more convenient to reinforce. Yet, up to factored load conditions, no structural integrity would be compromised.

Based on this re-analysis, using plasticity principles, the external post-tensioning was designed to ensure that the transfer girders achieve ultimate strength at factored load conditions. The deficient top and bottom slab regions were strengthened with additional reinforcement bonded to the slab (as was done on the fifth floor level). The re-analysis revealed that the strengthened slab system would perform adequately if the tops of the 7 deficient columns were allowed to develop a plastic hinge as a connection to the concrete beams above. To ensure the safe plastic deformation of the concrete column tops, confinement was provided using steel angle collars tightened together by high-strength bolts to achieve about 3.4 MPa of lateral pressure (see Fig. 4).

5. EPILOGUE

This case study is, in synthesis, a sad exposé of a structural engineer who totally disregarded the standard of reasonable care that is expected of any professional.