Construction Loads and Serviceability Requirements: Deflection Control, Span/Thickness Limitations

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Synopsis: Throughout the history of concrete construction, numerous construction failures have occurred involving excessive deflections and cracking of the completed structure. This paper presents two building construction cases where concrete slabs developed extensive cracking and excessive deflections soon after the slab construction and formwork removal. The effects of the shoring-reshoring operations, the rate of concrete strength development, as well as the effects of design details on the slab cracking and deflections, are investigated. The ACI 318 requirements of minimum thickness and deflection control are applied to both construction cases, and the adequacy of these code requirements is discussed. Based on the findings of this work it was concluded that the ACI 318 long term creep and shrinkage deflection calculation method does not adequately account for the early-age high construction loads.

Keywords: code provisions; concrete slabs; cracking; deflection control; long-term deflection; reinforced concrete; serviceability

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INTRODUCTION

It has been well documented that failure of concrete structures during construction manifested by extensive cracking and large deflections most often occurs as a result of formwork failure, overloading, and/or lack of concrete strength at the time of the construction load application. Premature removal of formwork during the construction of multi-story reinforced concrete buildings has been the cause of some dramatic failures, as well as the source of many serviceability problems such as excessive deflections and cracking. The high construction loads that are imposed on immature slabs, in combination with long term creep and concrete shrinkage, seriously affect the serviceability of the concrete structure.

Deflection control, however, cannot be achieved with the use of proper construction procedures alone. The selection of adequate slab thickness, which is a measure of slab stiffness and resistance to deflection and cracking, is equally important. Most codes, including ACI 318-99 (1), offer the Engineer the choice of either calculating the expected deflections and limiting these deflections to some specified allowable limits, or using a minimum slab thickness to satisfy certain maximum span/thickness ratios.

According to the commentary in ACI 318-99 Code, the minimum thickness requirements are applicable only in situations consistent with "previous experience in loads, environment, materials, boundary conditions, and spans." It has been a matter of a long debate whether a structure during construction falls within the "previous experience" based on which code provisions were developed. A structure under construction is often subjected to heavy construction loads which sometimes exceed by far the design loads. Ambient environmental conditions directly affect an open wall structure during construction, while completed structures are usually protected from the elements. The properties and strength of early age concrete differ from those of the cured and matured concrete. As a matter of fact, the early-age concrete strength is as difficult to predict as the prediction of the weather conditions which directly affect the concrete strength development. Furthermore, the boundary conditions and redundancy which affect the magnitude of deflections within a building under construction differ from those of a completed structure. Usually such conditions and peculiarities of buildings under construction are not accounted for during the design of the structure. Failure to comply with the ACI Code's minimum slab thickness requirements combined with problems during construction have resulted in excessive deflections.

Though serviceability failures in the form of excessive deflections are fairly common, they do not, however, generate as much interest and publicity as dramatic collapses. Deflection problems usually remain within the circles of the construction team "family" and rarely see the light of publicity. Engineering reports are generated usually to assist the opposing legal teams, but seldom attract media attention or become lesson for the public in avoiding similar failures in the future. The objective of this paper is to present the deflection and cracking problems of two separate construction cases, investigate the cause of these failures, and discuss the implications of the deflection requirements and the deflection calculation procedures of the ACI 318 code.

Construction Cases

The case studies presented in this paper involve two low-rise reinforced concrete buildings where the slabs developed extensive cracking and excessive deflections soon after the slabs construction and formwork removal. A common characteristic of both cases is that both were constructed during the winter months with prevailing low and sometimes freezing temperatures. The low temperatures which retarded the concrete strength development, combined with improper shoring procedures, resulted in excessive deflections and cracking. Inadequate slab thickness further exacerbated these conditions.

CASE STUDY I

Type of Construction

This building is a three-story flat slab reinforced concrete structure, approximately 180 ft. long by 150 ft. wide. It is an educational building consisting of offices, classrooms, and laboratories. The first floor and part of the second floor are 7-inch thick concrete slabs on grade. The second and third floors are elevated framed 9-inch thick, two-way flat slabs, with 8 ft. x 8 ft. by 5-1/2 inch deep drop panels. The roof slab is similar to the floor slabs except that an 8-inch thick flat slab was used. Floor areas adjacent to the elevator core are framed with one-way reinforced concrete slabs and beams. The flat plate slabs span 25 ft. in each direction and are supported by 18-inch square reinforced concrete columns and by reinforced concrete spandrel beams along the periphery. All columns are continuous down to the foundation level and are supported on spread footings. The partition walls consist of 8-inch concrete masonry block walls bearing directly on the floor slabs. The partition walls are generally located along the column lines. The third floor plan is shown in Figure 1.

Construction of the building started in the spring of 1992. About ninety percent of the structural slabs were placed between October 1992 and February 1993. The remaining slab areas were completed by July 1993.

Problem Description

Soon after completion of the floor slabs, and before the construction of the interior partition walls, it was discovered that the slabs displayed extensive cracking and excessive deflection.

The masonry partition walls were constructed on the top of the deflected slabs by thickening the wall bedding joints at the deflected areas. After a few months time the wall bedding joints displayed further racking and separation from the slabs. This condition suggested that the slabs continued to experience further deflection after the erection of the masonry walls. The slab deflections have caused several problems with the partition walls, door jambs, laboratory equipment, and long counters resting on the floors.

Concern was raised for the structural integrity of the building and the long term serviceability. Our firm was retained by the owners to investigate the cause of the problem and assist them with the subsequent legal battle. The building was not occupied during our investigation.

Our investigation consisted of field, laboratory, and office studies. The field studies included floor deflection surveys, crack surveys, reinforcement location surveys (pachometer surveys), impact-echo testing to verify the slab thickness, and coring of the concrete slabs. The laboratory studies included compressive strength testing and petrographic examination of the concrete cores. The office studies included the structural analysis, review of the design of the slabs to verify their load carrying capacity at various stages of construction, construction load analysis, and deflection calculations. This paper presents only the parts of our investigation that are relevant to the construction loads and deflections.

Field Studies

Slab deflections were measured by taking survey readings along column lines and slab centerlines. The relative slab deflections at the middle of the slabs vary from 1-1/2" to 2" for the second floor, 1-1/2" to 2-7/8" for the third floor, and 1/4" to 2-3/8" for the roof. Figure 1 shows typical slab deflections of the third floor. Typical crack patterns consisted of radial cracks emanating from the columns and parallel cracks along the column lines. The crack widths vary from hairline to 1/8" and most extended through the slab full depth. Figure 2 shows typical slab cracking.

Impact-echo testing indicated that the actual slab thickness varied from 9" to 11" with an average thickness of 10". The design thickness was 9". Thus, the slabs had to support the additional dead load of the excess concrete. The flexural capacity of the slabs was not increased by the thicker slabs since the effective depth of the top reinforcement was not increased. Pachometer readings indicated that the top rebars were placed deeper into the thicker slab.

Laboratory Studies

Testing of concrete cores indicated that the in-place concrete strength at the time of coring (approximately 3 years after construction) exceeded the specified design strength f"c of 4,000 psi. Examination of a core taken at a crack location indicated that the crack extended through several hard coarse aggregates. Such a crack is usually the result of structural overstress after hardening due to overloading.

Deflection Control

The flat slab design thickness was 9" with 5-1/2" drop panels. This slab thickness is within the ACI 318 (Section 9.5.3) requirements for minimum slab thickness. The immediate computed deflections based on the design loads vary between 0.25" and 0.4". The total long term deflections, including creep and shrinkage, are estimated in accordance with ACI 318 (Section 9.5.2.5) to be approximately one inch. According to Table 9.5d of ACI 318, the immediate live load deflections are limited to L/360 or 0.83" so that non-structural elements will not be expected to be damaged by large deflections. The long term deflections due to sustained loads and the immediate live load deflections are limited to L/480 or 0.63" when non-structural elements are expected to be damaged by large deflections. The calculated design deflections are within the limitations set by ACI 318. Therefore, this is a case where the slab thickness meets both the minimum slab thickness requirements, as well as the calculated deflection requirements as required by the ACI Building Code. However, the measured slab deflections were almost three times as much as the theoretical calculated deflections. The question is, what has gone wrong . . . . the code requirements or the construction methods?

Slab Age and Construction Loads

In order to answer the above question, the construction loads and the slab concrete strength/capacity at the time of the construction load application must be investigated. Table 1 shows the construction operations, and the maximum construction load on each floor as well as the age of the slab when the load is applied. The construction loads are evaluated based on the elastic construction load analysis by Grundy and Kabaila (2). In addition to the slab self weight, the slabs are assumed to carry a 50 psf construction live load, as recommended by ACI 347-92 (3). The 50 psf live load is assumed to be applied only at the top floor during concrete placement. A 20 psf miscellaneous construction load is assumed during the shoring/reshoring removal and installation. Also, a 5 psf formwork load is used as suggested by the formwork drawings.

Slab Strength Analysis - Safety During Construction

The structural strength of a young slab depends primarily on the available concrete strength at the time of the early age loading. To ensure safe construction, the slab construction loads must be smaller than the available flexural, shear, and bond strength of the concrete members.

The early-age concrete strength can be estimated by testing of field-cured cylinders in combination with other available non-destructive methods such as penetration probes, pulse velocity measurements, and maturity methods. The specifications of this project called for field-cured cylinders to determine the in-place concrete strength.

For predicting the estimated in-place concrete strength, ACI 306 (4) "Cold Weather Concreting" recommends maturity based methods which must be confirmed by field testing. The strength-maturity relationship of concrete mixes was developed on the basis that the strength of concrete depends on the curing time-temperature history of concrete. Maturity represents quantitatively the cumulative effects of temperature and time up to any given age. It is assumed that every concrete mix possesses a unique strength-maturity relationship and is valid for any given temperature history. The maturity relationship for a proposed concrete mix can be developed experimentally by a laboratory controlled testing before starting the building construction. Also, field-cured or laboratory-cured cylinders of the actual concrete provided at the site can be used to develop the strength maturity relationship of the supplied concrete mix. The most common definition of maturity is:

$$M = \sum (T - T_o)\,\Delta t \qquad (1)$$

where, M = maturity in °F-days
T = concrete temperature in °F during the particular time interval
T_o = datum temperature in °F, lowest temperature at which concrete ceases to gain strength with time.
Δt = time interval in days.

The appropriate value for the datum temperature "To" depends on the type of cement, the type and quality of admixture, and the range of the curing temperature. For cold weather conditions, ACI 306 recommends a datum temperature of 23° F.

The relationship between the concrete strength and the maturity function is:

$$fci = [AMi/(1 + AMi)]fcu \qquad (2)$$

where, "fci" is the concrete strength at a given time, "fcu" is the limiting value of concrete strength as maturity approaches to infinity, "A" is the normalized slope of the strength-maturity curve divided by "fcu", and "Mi" is the concrete maturity at a given time (F-Days).

Based on concrete cylinder strengths obtained from testing laboratory reports, the maturity relationships were developed for each placement of each floor level. The strength-maturity relationships can be used to determine the approximate concrete strength during construction, as long as the curing temperatures are known. Lacking records of construction daily temperatures, temperatures obtained from a nearby airport were used. The estimated maturities and predicted compressive strengths for each placement of each floor indicated that none of the predicted in-place concrete strengths reached the 4000 psi specified strength at the age of 28 days.

ACI 306R-88 recommends to keep the building heated to maintain a temperature of 50°F or more for several days after the concrete is placed. A review of the construction field reports showed that no adequate heat and protection of the freshly cast concrete was provided. The low and often freezing temperatures delayed the development of concrete strength significantly.

ACI 318 gives the minimum required strength limit state load capacity, U, as: U = 1.4D + 1.7 L, where D and L are the service dead and live loads, respectively. It is reasonable to assume that the ultimate load capacity of an early-age slab is proportional to the ratio of the specified design concrete strength to the partially developed early strength, i.e. load capacity during construction is given by:

Uc = [Load Capacity During Service]fc/f'c

where, Uc is the available slab strength during construction, fc is the partially developed concrete strength, f'c is the specified design concrete strength at 28 days.

At the time when this investigation was performed, there was no existing design code specifying load factors to be used during construction. It was then reasonable to assume the same load factors specified for service loads. The upcoming ASCE-37 (5) publication provides load factors for construction loads.

Under normal design and construction circumstances, it is logical to presume that the building structure has been designed to satisfy the relevant code provisions for flexure and shear. The design loads are 121 psf slab dead load (for the 9" slab with 5-1/2" drop panels) and 60 psf live load, and about 50 psf superimposed partition, floor finishes, and miscellaneous dead load. The total factored design load is 341 psf for the second and third floor slabs.

Table 2 shows the estimated concrete compressive strength and the available slab load capacities along with the applied construction loads. The table shows that the slab construction loads exceed the available load capacity of the slabs during the various phases of construction. The overloading is more than two-hundred percent of the available slab capacity at the second floor and almost two-hundred percent overloading at the third floor.

Calculated Deflections

The total long term deflections, including creep and shrinkage, were estimated in accordance with ACI 318, Section 9.5.2.5. The long term deflections were calculated for a period of 2-1/2 to 3 years, which is the approximate time of measured deflections, thus comparisons could be made. The ξ value of equation 9-10 of the ACI 318 for this period is taken as 1.75. The sustained service loads for the long term deflection calculations include the slab self weight, masonry partition walls, and 50 percent of the live load. Table 3 shows the estimated and measured deflections for the above period for the various slab placement areas.

The immediate and long term deflections have been based on the cracked concrete section. The cracking of concrete is primarily related to the applied loads and the available concrete modulus of rupture. At this building, the maximum slab loads were applied during construction when the concrete had not fully developed its full strength and resistance to cracking. It is therefore appropriate to calculate the service load long term deflections based on the effective moment of inertia of the cracked section that occurred during construction. Once a slab is cracked during construction it does not revert to an uncracked condition during service condition unless crack repairs are performed.

The calculated total long term deflections for the various concrete placements of each slab range from 1" to 1.9" for the second floor slab, and 1" to1.4" for the third floor slab. The maximum measured deflections for the same period of time range from 1.7" to 1.9" for the second floor and 2.2" to 2.9" for the third floor. The measured deflections are approximately double the theoretical estimated deflections.

ACI-318 Estimated Deflections vs. Measured Deflections

It is important to note that most of the estimated deflections for the various slab areas do not compare well with the measured deflections. Evidently, the

ACI 318 calculations procedure for the long term creep and shrinkage deflections generally underestimates the actual slab deflections when the slabs are heavily loaded at an early-age. It was concluded that the ACI 318 does not adequately account for the early-age high construction loads and the time-dependent cracking.

The ACI 435 Committee, "State-of-the-Art Report on Control of Two-Way Slab Deflections" (6) recognizes that the actual deflection may be larger than the ones calculated due to the restraint cracking caused by shrinkage and thermal contraction. ACI 435 cautions that for multistory slab construction, the extent of cracking is usually determined by the construction loads resulting from shoring and reshoring procedures. It further suggests to use high multipliers for the long term creep and shrinkage deflection estimation of slab systems.

Previous experimental and analytical studies (7-12) reported the inadequacy of the ACI 318 deflection procedure. These studies underlined the sensitivity of the slab deflections to early-age construction loads and suggested several methods to improve the ACI 318 procedure, including reduction of the concrete modulus of rupture, reduction of the effective moment of inertia, increased creep and shrinkage multipliers, introduction of the age of load and the construction-to-service load ratio into the deflection calculation, and other techniques. Most of these suggested methods yield approximately double the calculated deflections as compared with the present ACI 318 method. Indeed, had higher long term multipliers been applied in this case, the calculated deflections would have compared very well with the measured deflections. The calculated deflections of this case study based on the code provisions yielded deflections approximately half as much as the measured ones.

The question still remains - Is there a deficiency with the code provision, or with the construction method used for shoring and reshoring? However, before we attempt to answer this question, we should answer first another question - Why through the history of concrete construction have many floor slabs displayed deflections which were within acceptable code limits and why only a few slabs exhibited excessive deflections even though all slabs presumably have been designed using the same code requirements? The answer to the second question can be found within the different construction methods, distribution of construction loads, and environmental conditions that exist in the first few weeks of the life of the structure.

This case study is a typical case where the slabs have been overloaded during construction. Experience has shown that concrete loaded at an early age will have greater initial and long term deflections. Both the concrete strength and modulus of rupture were low when the construction loads were applied. Consequently, the concrete cracked and resulted in larger deflections. Furthermore, creep deflections are expected to be large since creep effects are dependent on the magnitude of the applied stress relative to the available concrete strength. Had the concrete been properly cured and protected from early overstress, it would have developed adequate strength to resist cracking and deflections and the deflections would have been within acceptable limits as required by the code.

CASE STUDY II

Type of Construction

This building is a two-story reinforced concrete structure supported on spread footings and grade beams. This educational building consists of a library and laboratories. The building plan measurements are approximately 75 feet in the east-west direction by approximately 188 feet in the north-south direction. The building was designed and constructed in 1996/97.

Both the first floor level and the roof level are ten-inch normal weight concrete slabs reinforced as one-way slabs. The typical bay size is approximately 32 feet by 32 feet. A 20 foot wide, 20-inch deep slab band at the center column lines spans in the north-south direction. Typical floor plan is shown in Figure 3.

Problem Description

The first floor and the roof slabs were constructed in December 1996 and January 1997. All formwork, shores, and reshores were removed by March 1997. Soon afterwards, by the middle of March 1997, the contractor installed metal stud walls at the ground floor. After about a three month period, by June 1997, the contractor observed that the metal wall studs under the first floor slab started to buckle. The stud buckling was due to the excessive deflection of the first floor slab exerting compressive forces on the metal studs.

The first floor slab and the roof displayed excessive deflections and extensive cracking. Questions were raised concerning the structural integrity and long term serviceability of the structure. At the recommendation of the owners, our firm was retained by the contractor to investigate the cause of the problem. Our investigation consisted of field, laboratory, and office studies, similar to Case Study I.

Field Studies

Top of slab elevation surveys of the first floor showed that the relative slab deflection at the middle of the slabs varied from about ½" to 2". The survey was conducted approximately nine months after the slab construction. Typical crack patterns consisted of cracks along the column lines in the east-west direction. Some of these cracks appeared to be through the slab. Also, several diagonal cracks, i.e. along the diagonal long dimension of the slab, were observed at the underside of the first floor slab. The width of the cracks varied from hairline to approximately 1/16". The cracking pattern of the top of the first floor is shown in Figure 4.