

Prediction of Long-Term Deflection of Reinforced Concrete Beams Suitable for Iraqi Conditions

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Abstract

This work includes theoretical and field measurements for long-term deflection of reinforced concrete beams. The calculated deflection of field beams are made according to a proposed model made in this work. This model takes into consideration the effects of construction loads on deflections, effects of cracking, and with a statistical support using Mean Error (M.E.) criteria.

A field-measured investigation is made in this work for reinforced concrete beams in a structure located in Al-Diwania city using precise leveling technique. The results are evaluated and compared with the values resulting from various world-wide methods.

The comparisons show incompatibility of these methods with local field-measurements. The reasons for which has been discussed. A proposed empirical formula to predict long-term deflection of reinforced concrete beams suitable for Iraqi conditions depending upon field-measurements of this work is presented and discussed.

الخلاصة:

يتضمن هذا البحث حسابات نظرية و حقلية للهطول طويل الأمد للعتبات الخرسانية المسلحة. الهطول المحسوب للعتبات الحقلية قد جرى بالاستناد الى نموذج اقترحه هذا البحث. يأخذ هذا النموذج بنظر الاعتبار تأثيرات أحمال الإنشاء على الهطول، و تأثيرات التشققات مع إسناد إحصائي باستخدام معايير متوسط الخطأ الإحصائية.

تم إجراء بحث حقلية للعتبات الخرسانية المسلحة في منشأ يقع في مدينة الديوانية باستخدام تقنية التسوية الدقيقة. تم تقييم النتائج و مقارنتها مع قيم ناتجة من مختلف الطرق العالمية.

أظهرت المقارنات عدم توافق هذه الطرق مع القياسات الحقلية المحلية. أسباب ذلك قد تم مناقشتها. تم تقييم ومناقشة صيغة وضعية مقترحة للتنبؤ بالهطول طويل الأمد للعتبات الخرسانية المسلحة ملائمة للظروف العراقية و معتمدة على القياسات الحقلية في هذا العمل.

1:- Introduction

Prediction of immediate and long-term deflection is important in design of concrete member for satisfactory performance during its use.

Calculating the deflection of reinforced concrete members is complicated by several factors, including tensile cracks, creep and shrinkage of concrete. The accuracy in prediction of deformation of reinforced concrete structures depends upon the rigor of the method of analysis and the closeness to actual conditions of the parameters used input data. It is impossible to eliminate the error caused by lack of accuracy in the input parameters, but use of a rational method of analysis can reduce the error considerably. An empirical approach can be accurate only when the conditions of the members considered are similar to those of experiments used to derive the empirical equations or the multipliers (Heiman, J.L.).

In 1974 a comparison of measured and calculated deflection of flexural members in four reinforced concrete buildings was presented by J. L. Heiman. Field measurements were made using a precise level and a finely-graduated staff. Reading were taken adjacent to the supporting columns and at the centers of slab panels or at the midspans of beams. The measured deflections were found to exceed those obtained by calculation using ACI Code method.

In 1984 Sbarounis J.A. presented a study for computing long-term deflection in a multistory flat plate building. Field measurement of deflections were obtained using standard level surveying techniques. The computing long-term deflections were in satisfactory agreement with measured one-year deflections.

In 1987 results of a survey of two-way slab deflections of 28 story office tower are presented by Jokinen and Scanlon. A comparison is made between measured deflections and deflections computing using a finite element program. Effects of construction loading and time dependent deformations are included in calculation. Deflection measurements were made at approximately 1 year after construction. These measurement were made by stretching a string line a long the diagonal between columns and measuring the deflection of the slab relative to the string line at mid-panel.

In view of the poor correlation between calculated long time deflections, and field-measured deflections, specific values of long-term multiplier (λ) is recommended in various countries. It will be shown in this work that a certain multiplier recommended by a certain code can not be recommended in every place. The reasons stem from the different circumstances, environments, practice and technological levels of every region.

This work is intended to estimate a suitable expression for a long-term multiplier under Iraqi circumstances and practice.

2:- Requirements of ACI318-05

Unless more comprehensive analysis is done, ACI318-05 requires that the immediate deflection be calculated by elastic analysis using an effective moment of inertia (I_e) not greater than (I_g)

$$I_e = [Mcr/Ma]^3 I_g + [1 - (Mcr/Ma)^3] Icr \leq I_g \quad \dots\dots\dots (1)$$

Where

$$Mcr = fr I_g / y_t \quad \dots\dots\dots (2)$$

Mcr is the cracking moment; Ma is the maximum moment member at a stage deflection is calculated; Icr is the moment of inertia of cracked section; fr is modulus of rupture (the tensile stress at which cracking occurs by flexure); I_g is the moment of inertia of concrete gross section neglecting reinforcement; and y_t is distance from the centroidal axes of cross section to the extreme fiber in tension.

For the additional long-term deflection resulting from creep and shrinkage of flexural members, ACI 318-05 requires multiplying the immediate deflection due to sustained load by factor

$$\lambda = \xi / (1 + 50 \rho') \quad \dots\dots\dots (3)$$

Where $\rho' = (As'/bd)$ is the ratio of compression reinforcement at midspan for simple and continuous beams and at support for cantilevers; As' is the area of the compression reinforcement; b is the width of compression face of member; d is the distance from extreme compression fiber to the centroid of tension reinforcement; and ξ is a time dependent factor equal to 2.0, 1.4, 1.2, or 1.0, respectively for 5 years or more, 12, 6, and 3 months.

3. Requirements of ACI Committee 209R-92

The equations recommended by committee are simplified expressions representing average laboratory data obtained under steady environmental and loading conditions.

The procedures used the material response parameters; i.e., strength, elastic modulus, creep, shrinkage, and coefficient of thermal expansion.

The difference between field measurements and the predicated deformations or stresses are mostly due to the lack of correlation between the assumed and the actual histories for water, temperature, and loading (ACI Committee 209-92).

Since live load dose not act in the absence of dead load, the following procedure must be used to determine the various deflection components:

$$(a_i)_D = \xi M_D L^2 / E_{ci} I_e \quad \dots\dots\dots (4)$$

Frequently (I_e) for M_D equals I_g ,

$$(a_t)_D = \xi_r \nu_t (a_i)_D \quad \dots\dots\dots (5)$$

Where ξ deflection coefficient, ξ_r reduction factor, and ν_t is the creep coefficient, (ratio of creep strain to initial strain),

A fictitious value:

$$(a_i)_{D+L} = \xi M_{D+L} L^2 / E_c I_e \quad \dots\dots\dots (6)$$

and then for live load,

$$(a_i)_L = (a_i)_{D+L} - (a_i)_D (E_{ci} / E_c) \quad \dots\dots\dots (7)$$

Note: dead load in the last four equations refers to the sustained load. In general, the deflection of a non-composite reinforced concrete member at any time and ultimate value in time is given by Eqs.(8) and (9) respectively.

$$a_t = \overset{(1)}{(a_i)_D} + \overset{(2)}{(a_t)_D} + \overset{(3)}{a_{sh}} + \overset{(4)}{(a_i)_L} \quad \dots\dots\dots (8)$$

a_u = Eq. (8) except that ν_u and $(\epsilon_{sh})_u$ shall be used in lieu of ν_t and ϵ_{sh} when computing terms (2) and (3) respectively $\dots\dots\dots (9)$

Where:

Term (1): is the initial dead load (sustained load) deflection as given by Eq. (4).

Term (2): is the dead load (sustained load) creep deflection as given by Eq. (5).

Term (3): is the deflection due to shrinkage warping as given by Eq. (14) in the next section.

Term (4): is the live load (that portion remained from sustained lode) deflection as given in Eq. (7).

4:-Other Methods for Computing Time-Dependent Deflection

4.1:- Yu & Winter, 1960

In 1960 Yu and Winter reporting on an extensive study of beam deflection, suggested two methods for calculating short-time deflections:

Method A: the cracked transformed section I at midspan is used as constant value throughout the length of the span for simple span, and the average moment of inertia for the positive and negative moment regions is used for continuous beams.

Method B: to account for the participation of the tensile concrete between the cracks, the short-time deflections computed by Method A were multiplied by the following correction factor:

$$\left(1 - b' \frac{M_1}{M_{max}} \right) \quad \dots\dots\dots (10)$$

In which M_{max} is the moment under working load, b' is the width of beam at the tension side and M_1 is defined as:

$$M_1 = 0.1(f'c)^{2/3} D(D - kd) \quad \dots\dots\dots (11)$$

The derivation of Eq. (10) followed an elastic theory approach with the factor 0.1 having been determined empirically.

Comparison with test data indicated that method B provides somewhat better results than method A. In the same year, Yu and Winter presented two methods for calculating long-term deflection:

Method C: (initial plus creep plus shrinkage deflection using increased " n " approach)

Method D: (creep plus shrinking deflection using the modified " Δ_i " approach)

Based on the long-time test results of 68 beams, the simpler method D was found to be somewhat closer to the experimental values than method C.

4.2:- Branson, Method

In 1963, a study by Branson (Branson, D.E., 1963, Branson, D.E., 1977, Branson, D.E., 1986, and ACI Committee 435, 1989) aimed to estimate the deflections of reinforced concrete beams under both instantaneous and long-term loading:

1:- Instantaneous deflections: An empirical expression cross section of a beam as a function of the bending moment, section properties, and concrete strength (in a form that includes the effect of extent of cracking):

$$I_{eff} = \left(\frac{Mcr}{Ma}\right)^4 I_g + \left[1 - \left(\frac{Mcr}{Ma}\right)^4\right] I_{cr} \quad \dots\dots\dots (12)$$

Where $Mcr = \frac{fr I_g}{y_t}$ with fr as the modulus of rupture I_g and y_t refer to the gross concrete section. The appropriate power of 4 was determined numerically from a sizable number of test results that included both rectangular beams (simple and continuous) and T- beams (simple).

An expression for the average effective moment of inertia over the entire length of a simply supported, uniformly loaded, rectangular or T-beam was determined as:

$$I_{eff} = \left(\frac{Mcr}{Ma}\right)^3 I_g + \left[1 - \left(\frac{Mcr}{Ma}\right)^3\right] I_{cr} \quad \dots\dots\dots (13)$$

Equations. (12) and (13) apply only when M or M_{max} is greater than or equal to Mcr ; otherwise $I_{eff}=I_g$.

2:- Long-term deflections: the method of Branson provided for separated considerations of shrinkage deflection are computed using Eq. (14).

$$a_{sh} = \epsilon_w \phi_{sh} L^2 \quad \dots\dots\dots (14)$$

Where: ϵ_w : shrinkage deflection coefficient, ϕ_{sh} : the curvature due to shrinkage warping. Creep deflections are computed using:

$$\Delta_{cp} = K_r C_t \Delta_i \quad \dots\dots\dots (15)$$

Where the K_r is reduction factor for the effect of compressive reinforcement C_t is the creep coefficient, and Δ_i is the deflection under initial loading computed with an effective moment of inertia given by Eq.(13). Values suggested for the reduction factor are:

$K_r = 0.85$ when $As'=0$; $K_r = 0.6$ when $As'= 0.5 As$; and $K_r = 0.4$ when $As' = As$.

Continuous beams are provided for the use of average positive and negative moment section values $(\rho - \rho')/\rho$ for shrinkage, and K_r for creep .

4.3:- Clarke, et al, 1987

Clarke, C.V., Neville, A.M., and Houghton, W., 1987 presented an approach to calculate the creep deflection of cracked flexural reinforced concrete members. A rational mathematical model had been developed relating the creep factor established from prism tests to an equivalent creep factor pertaining to the flexural member.

4.4:- Ghali Method

In 1993 Amin Ghali proposed code changes for predication of immediate and long-term change in length and deflections of reinforced members without prestressing. He reviewed the equations of ACI 318-89 and the Canadian Standard CAN 3 - A23.3 for prediction of immediate and long-term deflection in one-way nonprestressed construction.

4.5:- Samra, R.M., 1997

Renewed assessment of creep and shrinkage effects in reinforced concrete beams was presented by Samra, R.M. (1997). A creep model was proposed which involved the use of an enhanced iterative procedure that converges quickly, within 2-4 cycles.

4.6:- AL-Nu'man & AL-Baiaty, 2001

In 2001 AL-Nu'man and AL-Baiaty, presented a mathematical derivation of formulas, which based on ACI Code limits and requirements for many cases of beams condition. These formulas are presented in term of simple design charts using dimensionless parameters to control deflection.

5:- Field Measurement of Total Deflection

5.1:- General

Al-Qadissyia University building was chosen as the typical building to be investigated for many reasons. In the first place, its age is at least (7) years i.e. the long-term deflection had occurred almost completely. Other reasons are the Iraqi materials were used in construction, the fixed function of the building throughout its life, and a building in Al- Qadissyia city so it is suitable to be an example of strictly an Iraqi-construction practice.

5.2:- Description of the Building

The administration building of Al-Qadissyia University was built at least (7) years ago. Building construction is a skeleton type. The system of floors is slabs with beams. Clear beams without partitions below were chosen in the investigation in order to discard the restraints of the beams deflections.

The building consists of three stories; each story consists of one part and interior corridor. Figure (1) shows a plan of building. There are four sets of typical frames which included the 34 chosen beams, as indicated in Fig.(2)

5.3:- Field Measurement of Beams Deflections

Field Measurement of Beams total deflections was taken by means of leveling instrument (precise level and engineering staff). Each span took five readings, two of them are the opposite columns where the deflection value is zero, the third is approximately at the midspan of the beam where the deflection has maximum value, and the other values are approximately at the quarter-span. Measured deflections are obtained by subtracting the midspan readings from average column readings. The field works were conducted in the 2011.

Statistical situation of the (34) beams considered was made, these beams were classified into (10) sets according to section properties, span length, and loading. The final beams deflections statistics are summarized in Table (1):

5.4:- Field Measured Compressive Strength

Concrete compressive strength field-tests were made by means of direct method of Schmidt hammer rebound instrument. Over (100) readings were taken and the average of them was used to estimate the specified compressive strength. The statistical situation of field-readings is summarized in Table (2):

Cube compressive strength (w_m) may be approximately estimated using the curve of Schmidt hammer:

Average hammer rebound (R)= 40.7, hammer direction (a)= +90

Therefore, cube compressive strength (w_m) = 34.0 N/mm²

From Schmidt curve the range of tolerance (Δ) is ± 6.7 N/mm²

Therefore, $w_{max} = w_m + 6.7 = 40.7$ N/mm² and $w_{min} = w_m - 6.7 = 27.3$ N/mm²

Building code editions adopted the cylinder compressive strength, that is obtained by testing a cylindrical specimens, therefore, cube compressive strength must be adjusted:

Cube compressive strength / cylinder compressive strength = 1.2

Maximum cylinder compressive strength; $f'c_{(max)} = 40.7 / 1.2 = 33.75$ N/mm²

Minimum cylinder compressive strength; $f'c_{(min)} = 27.30 / 1.2 = 22.750$ N/mm²

The value of (25) N/mm² was finally selected between maximum and minimum value $f'c$ to be used in the calculations.

5.5:- Results of Deflection Measurements

Appendix (A) presents the details of the field – measurement of total deflections.

6:- Proposed Computed Long-Term Deflections of Reinforced Concrete Beams

6.1:- The Significance of Calculations

The current code editions give empirical equations for predicting the long-term deflections for reinforced concrete flexural member. An empirical approach can be accurate only when the conditional of the members considered are similar to those of the experiments used to derive the empirical equations or the multiplier.

Therefore, many authors have suggested different empirical equations or multiplier, which are suitable for their country's conditions.

The aim of these calculations is to find an empirical equation for computing the additional long-term deflection of reinforced concrete beams under Iraqi conditions.

6.2:- Method and Procedure

The following simplified procedure, is proposed for estimating deflections in multistory construction.

1. Calculate the maximum deflection, a_{max} , due to construction load.
2. Cracking is accounted for in the analysis using Branson's effective moment of inertia. The modulus of rupture must be specified. In this study, the next value (ACI318-2005) was adopted.

$$f_r = 0.62 \sqrt{f'c} \quad \dots\dots\dots (16)$$

3. The maximum deflection, a_{max} is scaled to the sustained load level to obtain the immediate deflection.
4. Estimate the long-term deflection using appropriate additional multiplier (λ) which give satisfactory agreement with field measured deflections.

The details of the procedure are explained in the following flow chart, Fig. (3), which shows, in addition, the use of the mean-error criteria to reach an appropriate multiplier.

6.3:- Calculations of Maximum Construction load:

A simple procedure to determine slab loads during construction was proposed by Grundy and Kabaila (1963). More refined analysis procedures reports subsequently, e.g., Liu, Chen, and Bowman (1985) and Aguinaga- Zapata and Bazant (1986) give results quite similar to the original Grundy and Kabaila procedure.

The maximum load during construction including loads due to shoring and reshoring plus an allowance for construction live load can be estimated using relationship:

$$W_{max} = K_1 K_2 R W_{slab} + W_{cl} / N \quad \dots\dots\dots (17)$$

Where:

K_1 : allowance for error in theoretical load ratio, K_2 : allowance for weight of formwork, R: load ratio calculated by Grundy and Kabaila = applied load / slab dead load, W_{slab} : slab dead load, W_{cl} : construction live load, and N: number of shored and reshored level.

Gardener (1985) recommended $K_1=K_2= 1.1$. The construction live load may be taken as 50 lb/ ft² (2.4 kPa), as recommended by ACI 347R-1990. The factor K_1 accounted for error in computing R due to variations in stiffnesses between the stores in the supporting system.

The factor R has been shown to from 1.8 to 2.2, depending primarily on the number of stories of shores and reshores the system. If the shoring system to be used is unknown, a value of R=2.0 can be used in the calculations (ACI Committee 209-92).

Depending on the data obtained from the building under consideration, the following values are adopted:

$K_1=K_2=1.1$, R= 2.0, $W_{cl}= 2.4$ kPa, and N=3

Calculations of (bf, Y_t , and I_g) for section of beams are made. (bf) the effective flange width is calculated according to ACI318-2005.

Program STAAD Pro.2005 is used to calculate maximum moments and instantaneous beam deflections due to maximum construction load.

Calculations of non-linear beams deflections due to maximum construction load (a_{max}), sustained load, (W_{sus}), and Non-linear beam deflections due to sustained load, (a_{sus}) are explained in the following steps:

- 1:- Non-linear $a_{max} = \text{Linear } a_{max} \times I_g / I_e$
- 2:- $W_{sus} = \text{dead load} + 40 \% \text{ live load}$

Coefficient method (Nilson, A.H., 2004) was used for the distribution of the slab sustained load to the supporting beams.

Using well-known method that were mentioned previously (ACI 318-2005, ACI 209R-92, Branson, and Yu & Winter method D), long-term deflection are computed for the (34) beams considered.

7:- Prediction of Long-Term Deflection of Reinforced Concrete Beams Suitable for Iraqi Conditions

7.1:- General

In this section, an empirical equation for computing the additional long-term deflection of reinforced concrete beams under Iraqi conditions is attempted. The proposal is discussed and compared with well-known methods from literature.

7.2:- Proposal of New Value of (ξ) for Iraqi Practice

The general form of ACI318-2005 equation for computing the additional long-term deflection is:

$$\lambda = \xi / (1 + 50 \rho')$$

Where: λ = multiplier for additional long – term deflection

ξ = time-dependent factor for sustained load

ρ' = compression reinforcement ratio, $A's/ bd$

The experimental results of (34) beams showed that the time-dependent factor is not pertained in the beams under Iraqi conditions, therefore; a new ultimate values of (ξ) are proposed to make this approach suitable for Iraqi practice, these proposed values are obtained using (M. E) criteria:

$$\text{Mean Error (M.E.)} = \text{MRi} / n \quad (18)$$

Where : $R_i = Y_i - Y_j$, Y_i = field-measurement value , Y_j = predicated value, MRi = average of R_i and n = standard deviation of predicted values. (M.E.) values generally ranged from -3.00 to + 3.00. to get a good agreement between measured and predicted values, (M.E.) must be convergent to zero.

Satisfactory convergence was made, and according to ($A's / A_s$) ratio, two values of (ξ) are accepted as suitable value for Iraqi practice instead of the value of Ref. No.9-19 in the ACI 318-2005 Equation. Table (3) summarize the last operations:

7.3:- Proposal Model for Calculation of Long-Term Deflection of Reinforced Concrete Beams Suitable for Iraqi Conditions:

- 1:- Calculate maximum construction load, (w_{\max}).
- 2:- Estimate maximum moment due to maximum construction load, (Ma_{\max}).
- 3:- Calculate liner maximum construction load deflection, (a_{\max}), according to gross moment of inertia, and by means of any suitable linear method.
- 4:- Select the case of stage of cracking by choosing the suitable modulus of rupture, f_r , compatible to this stage.
- 5:- Compute the effective moment of inertia, I_e , according to the selected modulus of rupture.
- 6:- Convert the linear maximum construction load deflection to non-linear maximum construction load deflection by multiplying a_{\max} by (I_g / I_e)
- 7:- Calculate the sustained load, (W_{sus}).
- 8:- Scale the non-linear maximum construction load deflection to the sustained load level to obtained the immediate deflection, a_{sus} .
- 9:- To obtain the time - dependent or the long - term deflection, multiply the immediate deflection, a_{sus} , by the ultimate additional time - dependent deflection factor, λ , and according to ($A's / A_s$) ratio, as follow:

$$a_{\text{total}} = a_{\text{sus}} + \text{additional ultimate deflection} = a_{\text{sus}} + a_{\text{sus}} (\lambda) = a_{\text{sus}} (1+\lambda)$$

$$a_{\text{total}} = a_{\text{sus}} (1+\lambda) \quad (19)$$

Where

$$\lambda = 3.586 / (1 + 50\rho') \quad \text{when } (A's / A_s) = 0.5 \quad \dots\dots\dots (20)$$

$$\lambda = 2.326 / (1 + 50\rho') \quad \text{when } (A's / A_s) = 1.0 \quad \dots\dots\dots (21)$$

ρ' = steel ratio in the compression zone.

7.4:- Discussion of Result and Comparative Study

A comparison is made between measured and calculated long-term deflections of the investigated beams in the considered building. Figures (4) through (7) represent that the measured deflections were found to exceed those obtained by calculation using the methods of ACI318-2005, ACI 209R-92, Branson, D.E., 1968, and Yu & Winter Method D (Yu, W. W. and Winter, G.), some causes of the discrepancies between the two sets of results were:

- (1) The actual loads that were applied to the beams often differed widely from those used in the calculation method.
- (2) The values of some of the preparation of the concrete used in the calculations, often did not represent closely the properties of the in-situ concrete.

(3) The assumptions on which the methods of calculations were based did not correspond accurately to the behavior of the beams under field conditions.

(4) The long-term deflections of the beams were influenced by their early loading history, by the construction practices used and by environmental conditions that accrued during this period, factors that are not accurately taken into consideration in commonly-used methods of calculating deflection.

On the other hand, several researchers like Heiman and Branson, D.E., 1968 indicated that the deflections calculated with the ACI code method are less than the measured deflections. The last one suggested that using the value of (2.5) rather than (2) for ξ in the ACI equation for calculating the additional long-term deflections gives improved results.

The developer for the ACI code method for determining long-term deflections was based partly on laboratory investigations carried out by Yu & Winter (Yu, W. W. and Winter, G.) and by Washa and Fluck on reinforced concrete beams subjected to continuously-applied live loads that produced sustained stresses considerably higher than those likely to be procedure by the actual service loading.

The deflection behavior of the laboratory beams differed markedly from that of common building construction. The statistical situation of the problem is given in the next comparative table (Table 4):

Figures (8) through (12) shows a comparison of ACI 318-2005, ACI 209R-92, Branson, Yu & Winter method D, and the proposed computation methods respectively along with the field-measured values.

Moreover, Figures (13) through (17) show clearly how each respective method of computing total deflection being in harmony with the measured values or not, in other words, which method will be acceptable and considered in view of the local field measurements. Except of the proposal of this work, neither method is shown acceptable to represent the long-term deflections of local reinforced concrete beams.

The best one which approaches the measured values is [ACI 318-2005] and the farther one is [Yu & Winter method D].

It can be concluded that the main reasons for the discrepancy between the well-known method and the difference of the circumstances under which the methods are established, the difference in the construction materials and in the construction methods also in the respective countries.

8:- Conclusions

1:- The results of investigations in this work indicate that the long-term deflections of beams obtained by the commonly-used methods are less than the corresponding measured values of local reinforced concrete beams.

2:- Ultimate values of the time-dependent factor for sustained loads (ξ) (ACI318-2005) approach-suitable for Iraqi conditions were searched for in this work and found to be equal to (3.586) when $A's = 0.5 A_s$, and (2.326) when $A's = A_s$.

Accordingly, it can be stated that the values of (λ), the additional multiplier in this work, is equal to (1.793 λ_{ACI}) and (1.163 λ_{ACI}) for $A's=0.5A_s$ and $A's = A_s$, respectively.

3:- The field measured long-term deflection of the beams of Al-Qadissyia University building were found statically consistent and therefore reliable for the forthcoming analysis.

4:- A proposal model for the prediction of long-term deflections of reinforced concrete beams conformable for Iraqi practice was developed in this work. This model takes into consideration the degree of cracking, construction load, and

concrete tensile strength. The flow chart in Fig. (3) shows the characteristic of this model.

5:- Of the various models in literature, the closest one which approaches the proposal model is [ACI 318-2005] model, and the most developed one is [Yu & Winter method D] model.

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Table (1) : Statistical situation of the measured deflection

No. of beams	X (mm)	S. D. (mm)	C.V. %
4	5.8500	0.8610	14.7179
4	8.1250	1.3409	16.5034
4	9.3625	0.0960	1.0254
4	11.5375	1.8464	16.0035
4	11.4375	0.9127	7.9799
4	10.2000	0.8624	8.4549
2	14.0000	0.2500	1.7857
2	11.8750	0.1250	1.0526
4	4.8875	1.1834	24.2128
2	13.0250	0.0250	0.1919

Where: S.D: standard deviation, X: means of deflection values, C.V.: coefficient of variation.

Table (2): Statistical situation of Schmidt hammer rebound value.

No. of readings	S.D	C.V.%	X	Range
116	2.850	7.000	40.700	18.000

Table (3) New Values of (ξ) Versus (A's / As) Ratio

A's / As	M.E.	(ξ)
0.5	6.2806×10^{-3}	3.586
1	6.4469×10^{-3}	2.326

Table (4): Statistical situation of the five methods.

Computed/measured	X (mm)	S.D. (mm)	C.V.%
ACI318-2002 ⁽¹⁾	0.820	0.198	25.560
ACI209R-92 ⁽⁷⁾	0.573	0.122	23.260
Branson ⁽²⁾⁽³⁾	0.756	0.168	22.990
Yu & Winter method D ⁽³⁾	0.651	0.129	22.080
Proposed	1.029	0.240	23.000

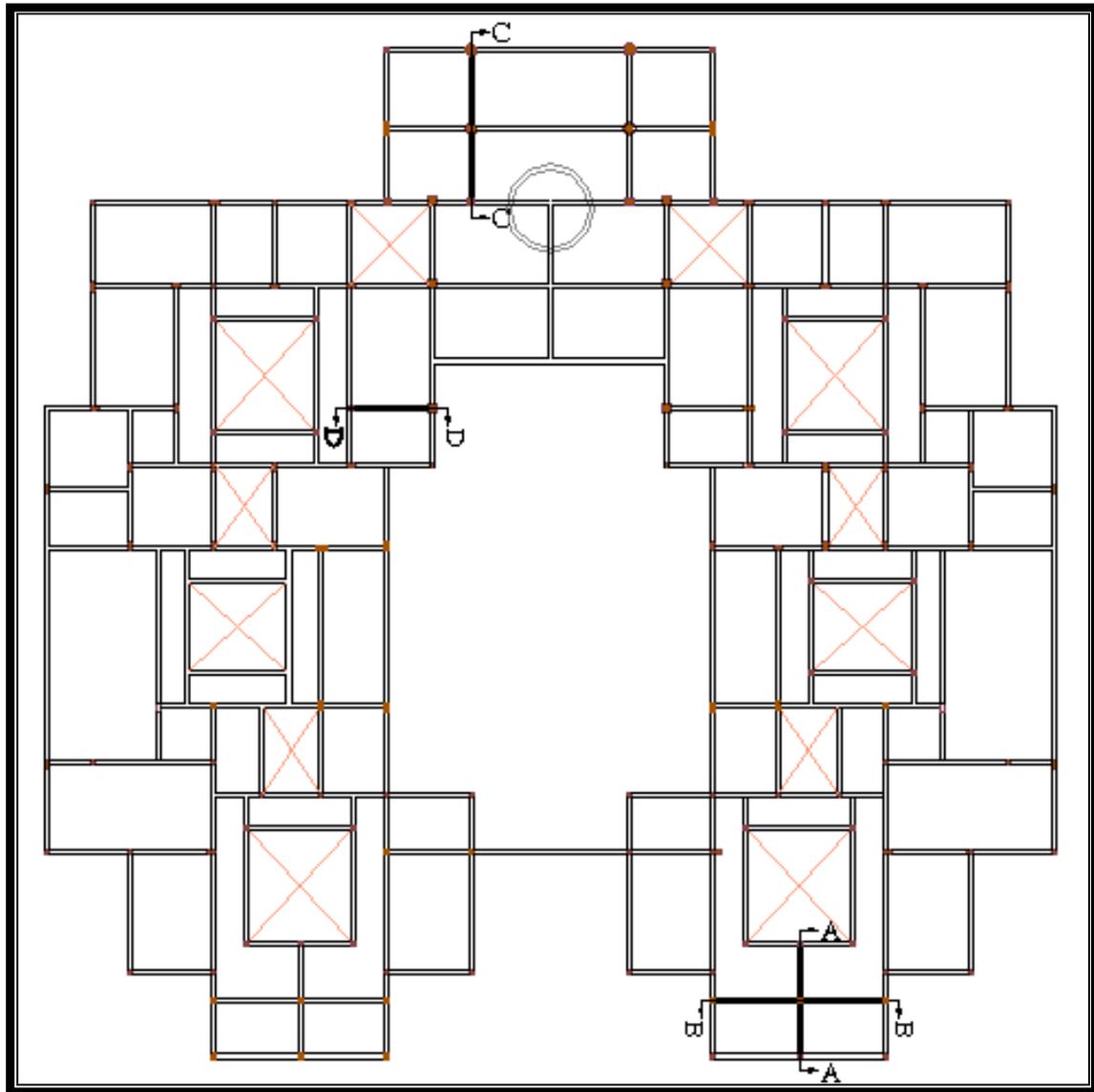


Fig. (1) Plan of the Building.

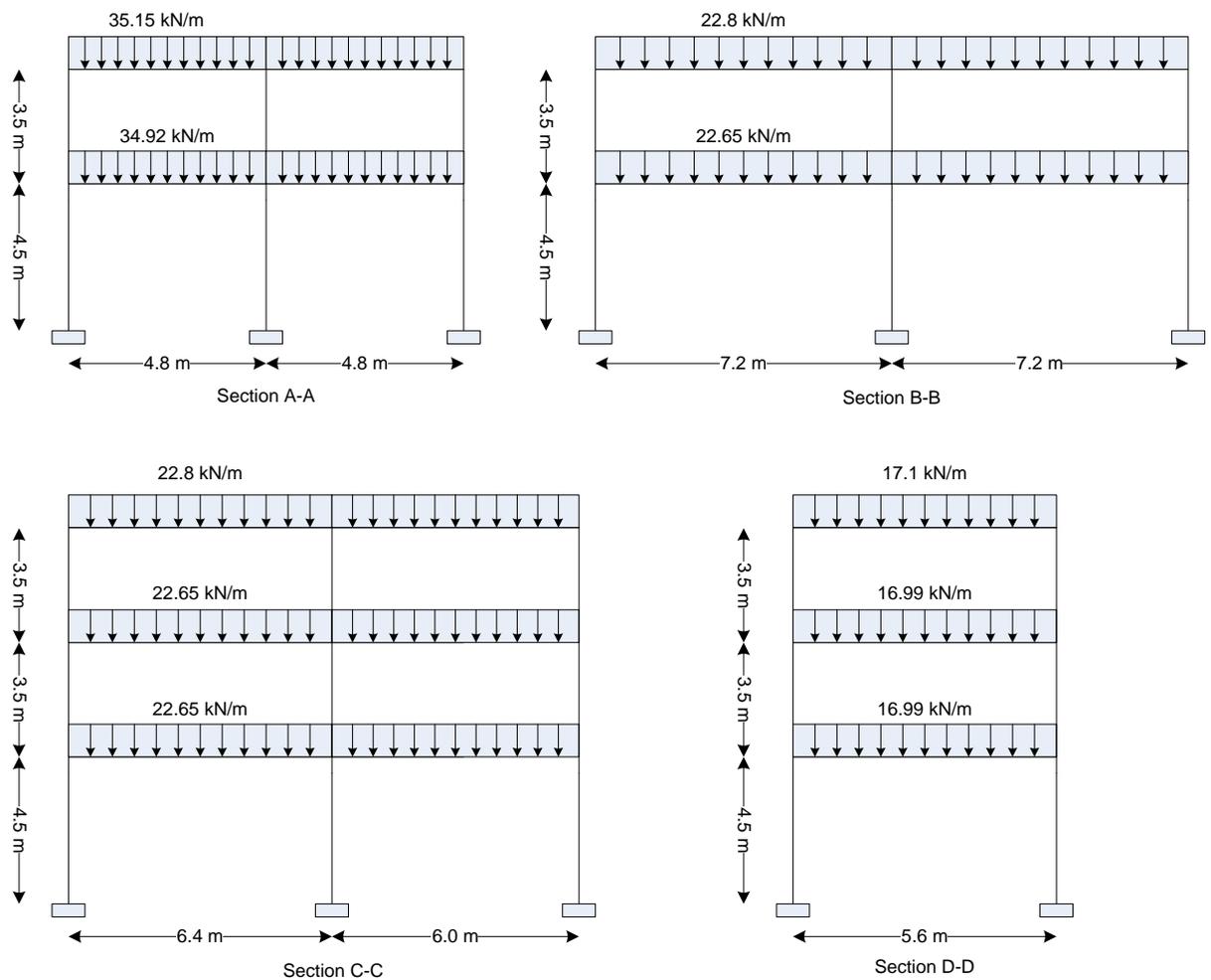


Fig. (2) Typical vertical sections with maximum construction loads

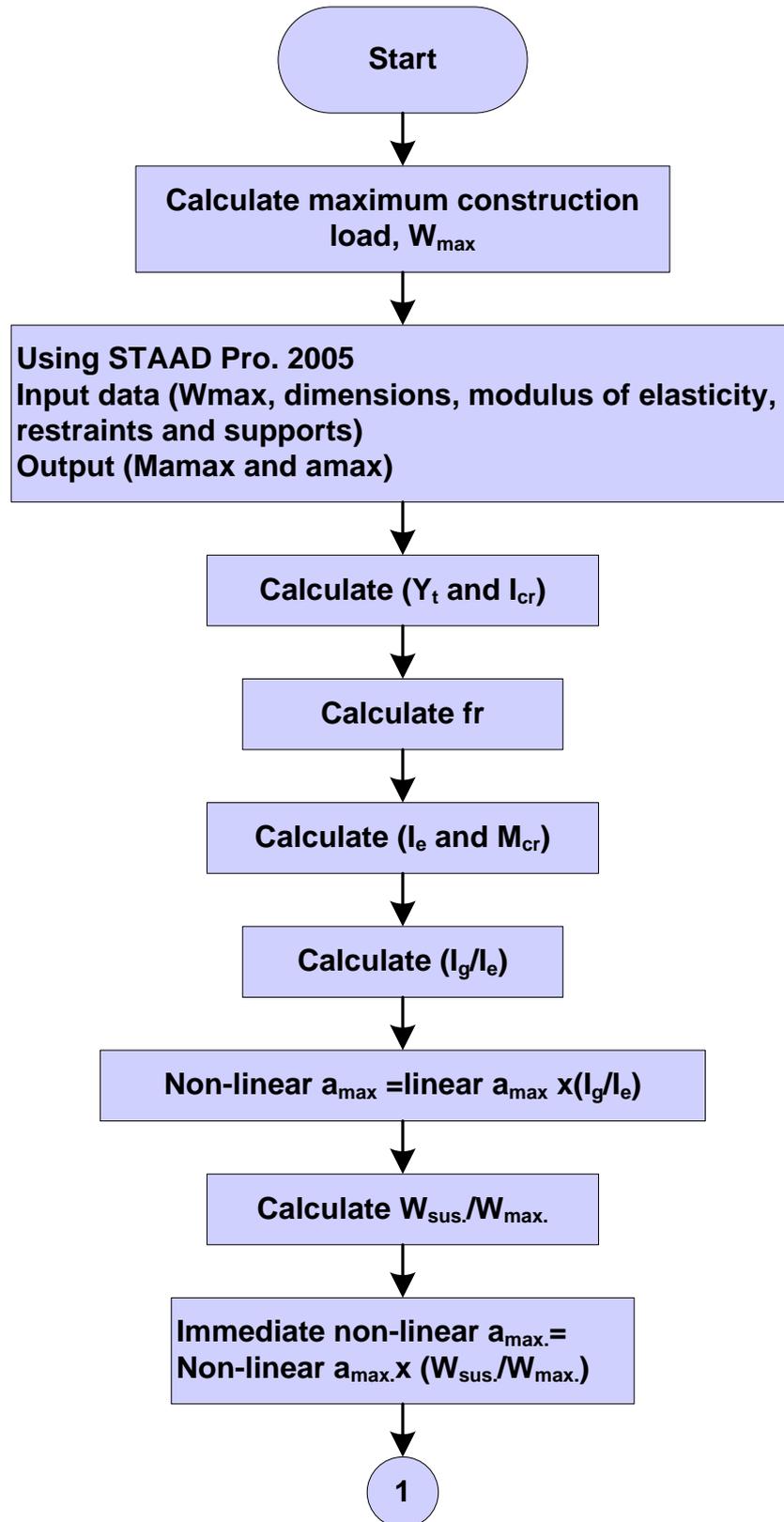


Fig. (3) Flow Chart for Deflection Computation.

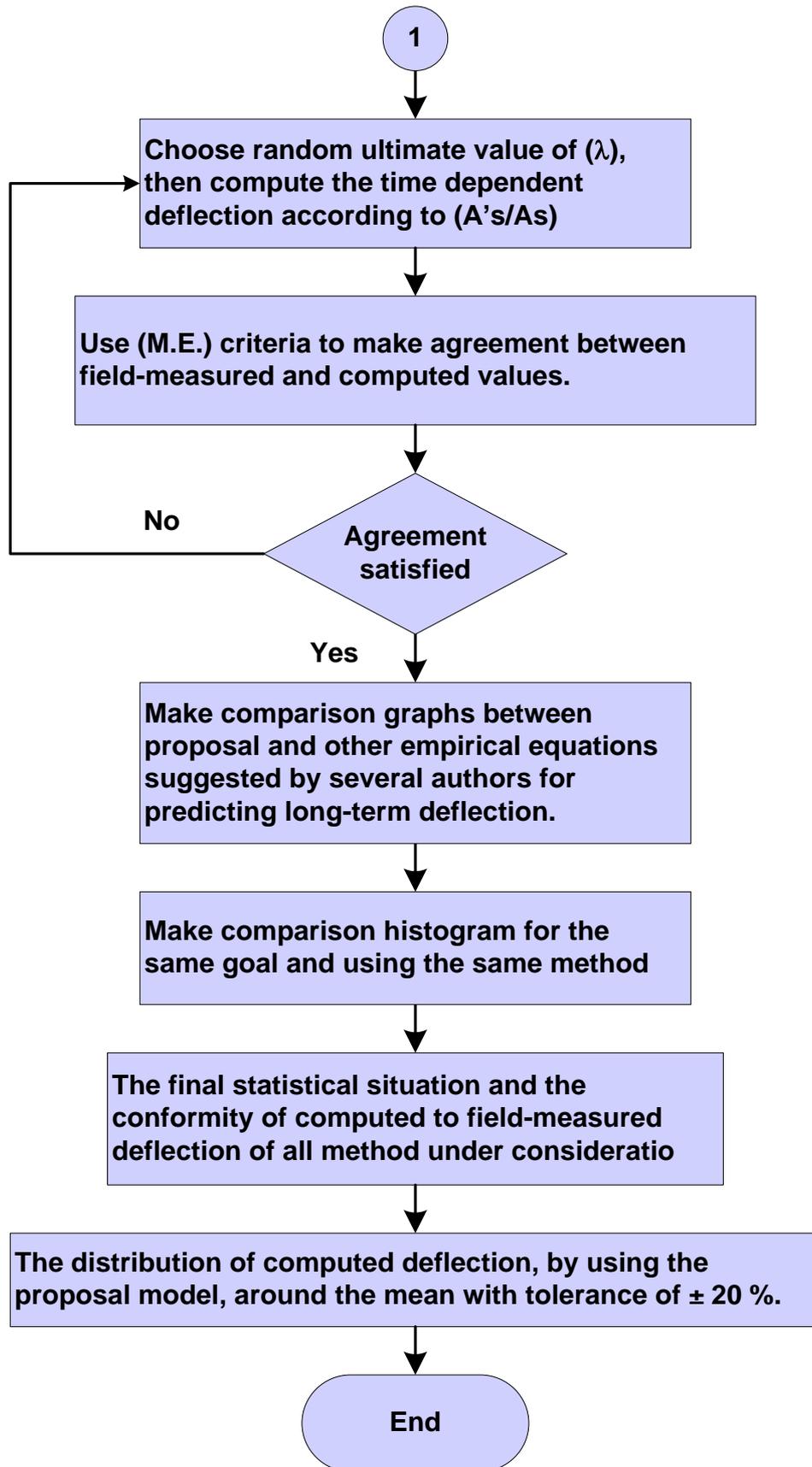


Fig. (3) Continued.

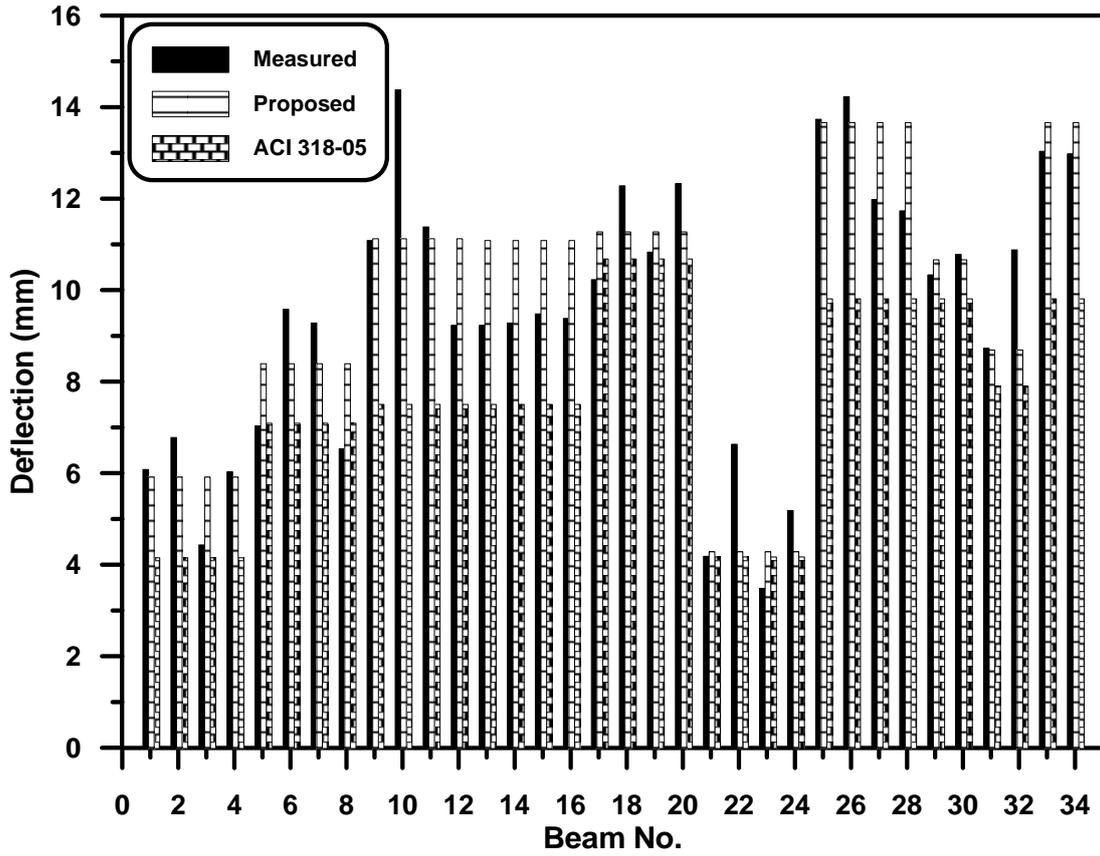


Fig. (4) Measured, Proposed model and ACI 318-05 predicted long-term deflections.

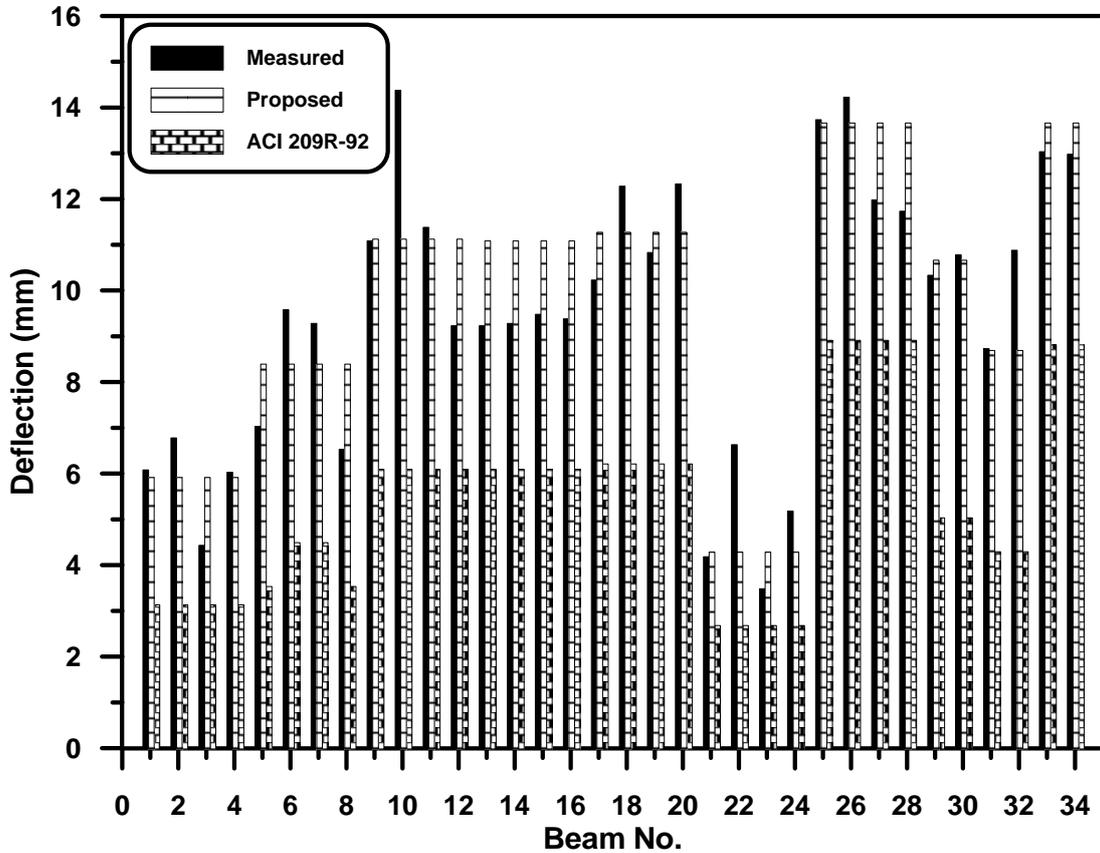


Fig. (5) Measured, Proposed model and ACI 209-92 predicted long-term deflections.

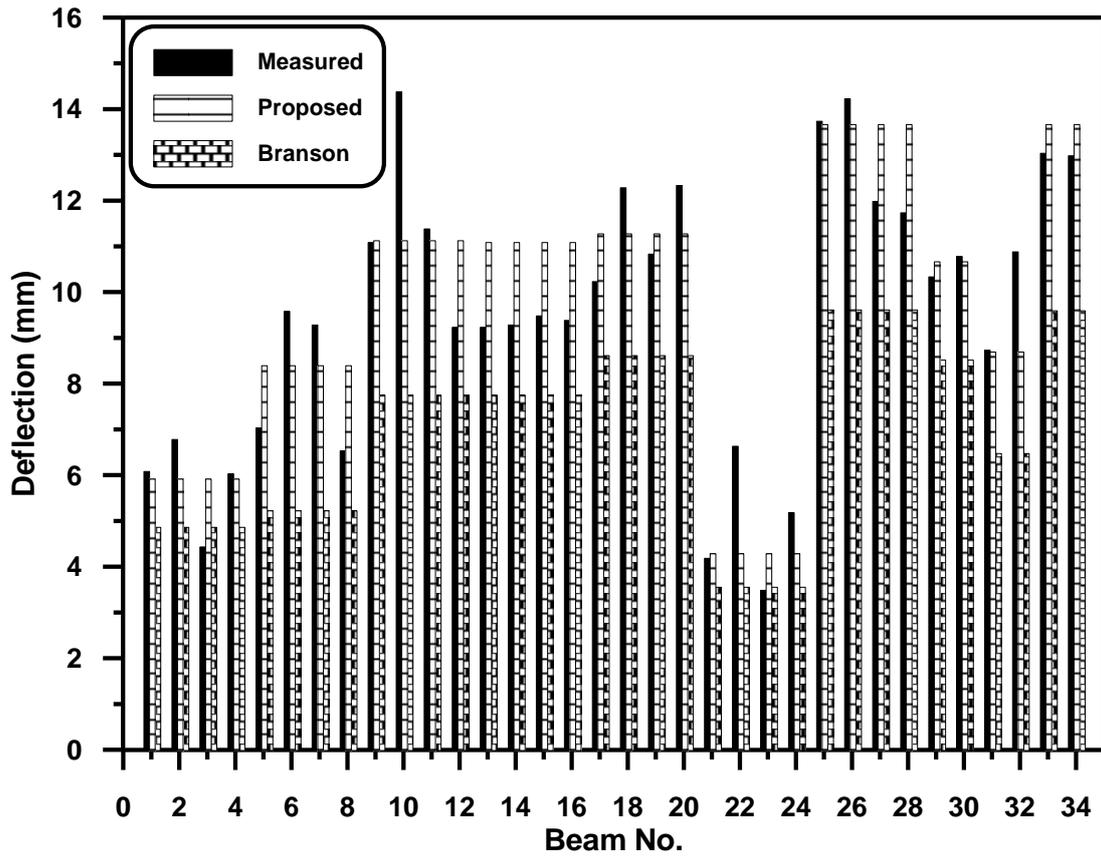


Fig. (6) Measured, Proposed model and Branson predicted long-term deflections.

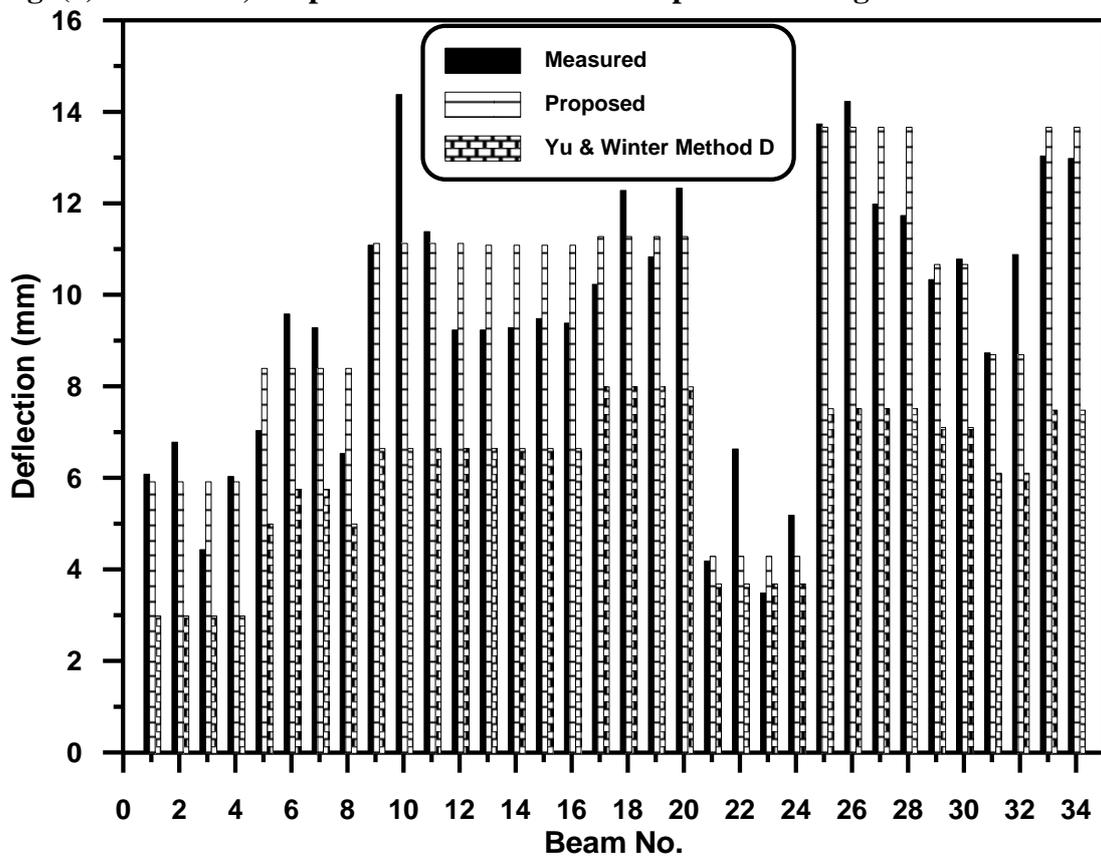


Fig. (7) Measured, Proposed model and Yu & Winter Method D predicted long-term deflections.

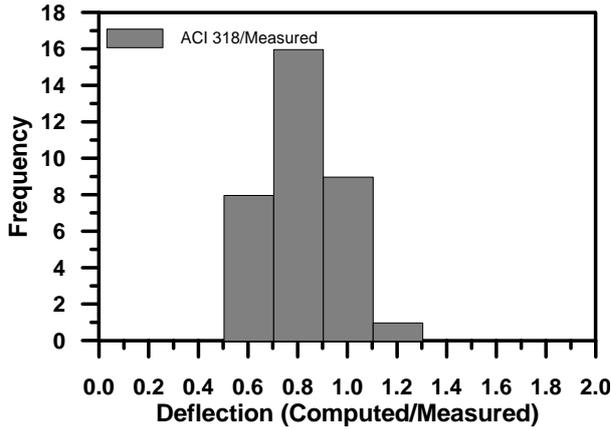


Fig. (8) Histogram of Ratios of Computed Total Deflection to Actual Measured.

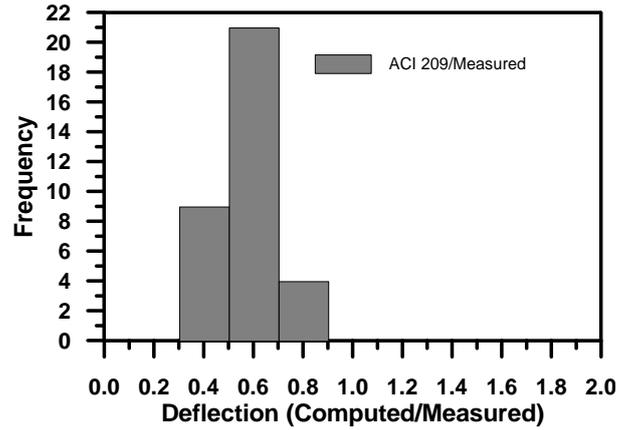


Fig. (9) Histogram of Ratios of Computed Total Deflection to Actual Measured.

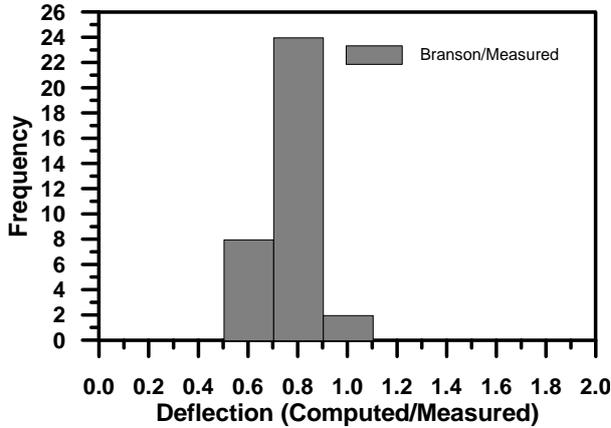


Fig. (10) Histogram of Ratios of Computed Total Deflection to Actual Measured.

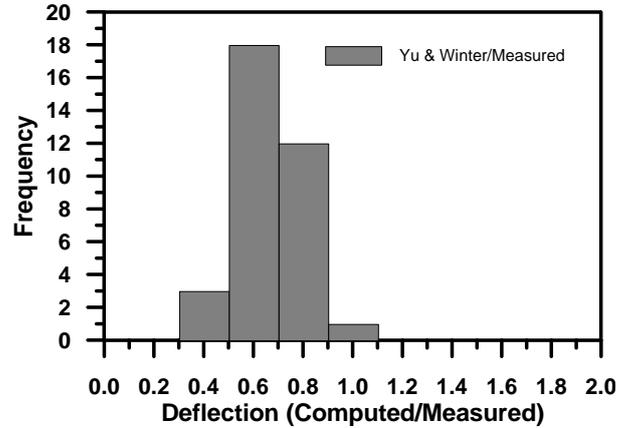


Fig. (11) Histogram of Ratios of Computed Total Deflection to Actual Measured.

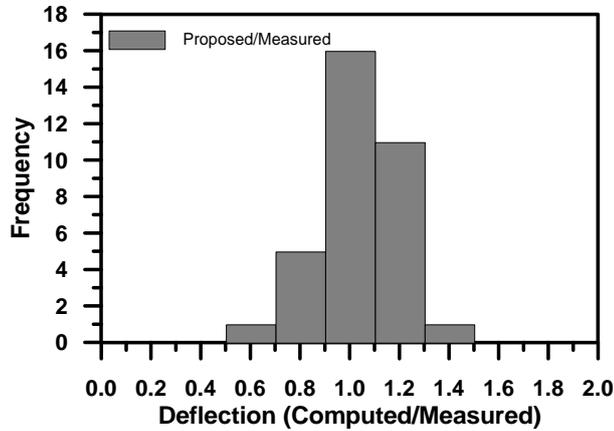


Fig. (12) Histogram of Ratios of Computed Total Deflection to Actual Measured.

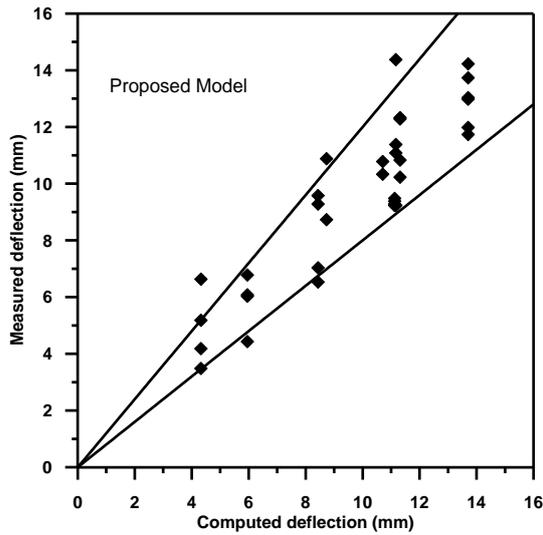


Fig. (13) Comparison of Measured and (Proposed) Predicted Long-Term Deflection.

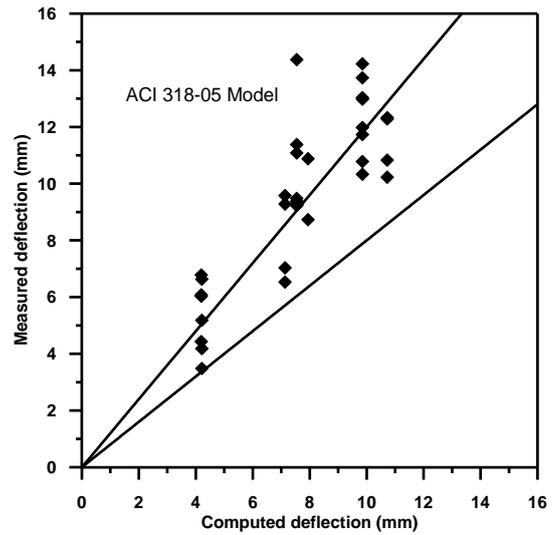


Fig. (14) Comparison of Measured and (ACI 318-05) Predicted Long-Term Deflection.

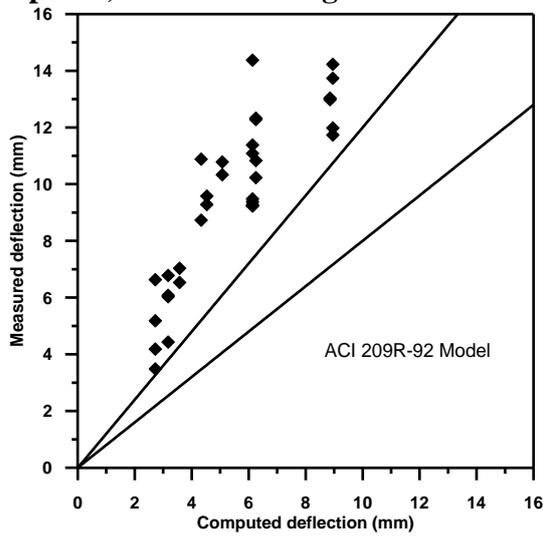


Fig. (15) Comparison of Measured and (ACI 209R-92) Predicted Long-Term Deflection.

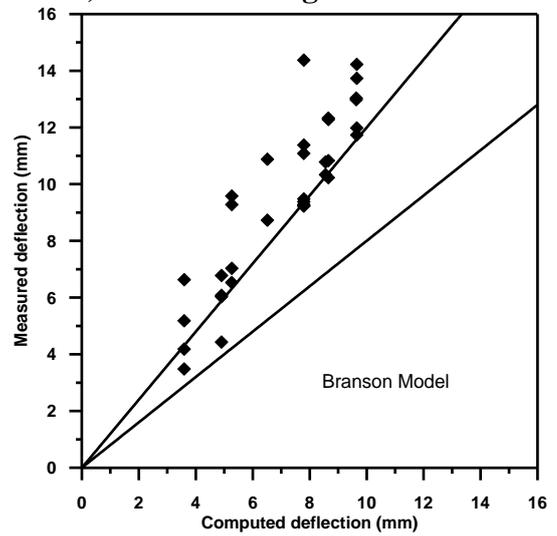


Fig. (16) Comparison of Measured and (Branson) Predicted Long-Term Deflection.

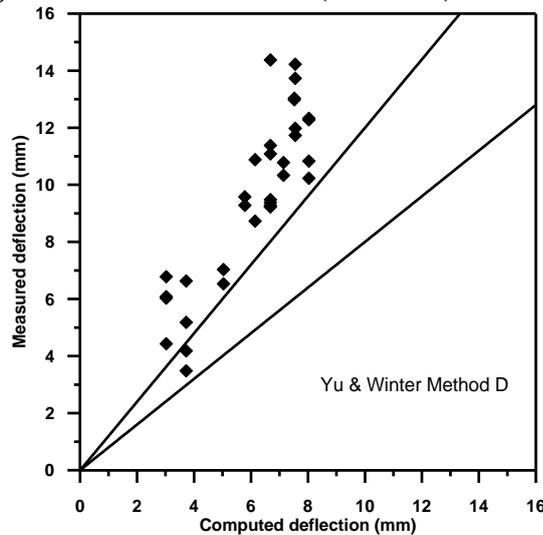


Fig. (17) Comparison of Measured and (Yu & Winter Method D) Predicted Long-Term Deflection.

Appendix: Field Measurement Table
Table (A) : measured deflections & section properties

No.	Beam Length (mm)	Measured deflection (mm)	Web depth (mm)	Beams width (mm)
1	4800	6.10	450	300
2	4800	6.80	450	300
3	4800	4.45	450	300
4	4800	6.05	450	300
5	4800	7.05	450	300
6	4800	9.60	450	300
7	4800	9.30	450	300
8	4800	6.55	450	300
9	7200	11.10	450	300
10	7200	14.40	450	300
11	7200	11.40	450	300
12	7200	9.25	450	300
13	7200	9.25	450	300
14	7200	9.30	450	300
15	7200	9.50	450	300
16	7200	9.40	450	300
17	6400	10.25	450	300
18	6400	12.30	450	300
19	6400	10.85	450	300
20	6400	12.35	450	300
21	5600	4.20	450	300
22	5600	6.65	450	300
23	5600	3.50	450	300
24	5600	5.20	450	300
25	6400	13.75	450	300
26	6400	14.25	450	300
27	6000	12.00	450	300
28	6000	11.75	450	300
29	6000	10.35	450	300
30	6000	10.80	450	300
31	6000	8.75	450	300
32	6000	10.90	450	300
33	5600	13.05	450	300
34	5600	13.00	450	300