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Authors: An Lin, Nanjing Institute of Technology
Ding Dajun, Nanjing Institute of Technology

Subject: Structural Engineering

Keywords: Concrete
Structure

Publication Date: 2001

Original Publication: CTBUH Journal, Spring 2001

Paper Type:

1. Book chapter/Part chapter
2. **Journal paper**
3. Conference proceeding
4. Unpublished conference paper
5. Magazine article
6. Unpublished

Studies on Serviceability of Concrete Structures under Static and Dynamic Loads

Ding Dajun and An Lin
Nanjing Institute of Technology, Nanjing, China

ABSTRACT

This paper presents a series of experimental and theoretical studies on the serviceability of concrete structures, conducted at Nanjing Institute of Technology (NIT) for more than 35 years. They include studies of cracking and deformations of concrete members under static loading and of deformations under dynamic loading. For crack widths and deformations under static loading, the first author has suggested a unified calculation procedure based on a larger number of experimental studies, which includes the members subjected to flexure under short- and long-term loading and under eccentric tensile or compressive forces. More than 700 specimens were tested. Most of the calculation procedures suggested have been adopted in relevant Chinese design codes and specifications. For the deformations under dynamic seismic and wind actions, the 318m high Nanjing TV Tower was chosen as the case study.

INTRODUCTION

The structural design codes current in most countries prescribe that two limit states should be considered for concrete structures, i.e., ultimate limit state and serviceability limit state. The CEB-FIP Model Code (published in 1993) classifies the serviceability limit state into three categories: 1) limit state of cracking and excessive compression, 2) limit state of deformations, and 3) limitation of vibrations.

The Chinese Design Code of Concrete Structures, GBJ10-89, classifies beams of different types (e.g., floor beam, roof beam, crane beam, etc.) according to their circumstantial conditions (such as normal room temperatures, outdoor or indoor conditions, or very humid conditions) in which these members are placed for service. Thus there are three levels (I, II, and III) of controlling cracking. Reinforced concrete (RC) members belong to level III because they cannot be guaranteed not to crack and maximum crack widths could not be limited to a maximum of 0.2, 0.3, and 0.4mm according to different cases of exposure. Prestressed concrete (PC) members may belong to level II or III for pre-stressing cold-tensioned steel bars, and to level I or II for high strength wires. The members of level I should be strictly precluded from crack formation and are not permitted to produce tensile stress in concrete on their tensile face under the combination of the effects of short-term actions. The members of level II should be generally precluded from crack formation and are not permitted to produce tensile stress in concrete on their tensile face under the combination of the effects of long-term actions. Members of level II are permitted to produce tensile stress σ_{ct} under short-term actions but σ_{ct} should not be larger than $\sigma_{ct}\ \gamma f_{tk}$, where α_{ct} is a limitation factor of tensile stress in concrete and is equal to 0.3, 0.5 or 1.0. In this case, γ is a factor representing the influence of plastic deformation in tensile concrete depending on the section shape, determined from the assumption of plane section hypothesis of beam theory and the tensile edge strain equal to $2f_{tk}/E_c$ as well as the tensile stress diagram on a rectangular section, as given in the Design Code GBJ10-89 (e.g., $\gamma = 1.75$ for rectangular and T-sections, whereas the values for γ are generally smaller than 1.75 for, inverted T- and I-sections, and f_{tk} is the characteristic tensile strength of concrete).

For calculating the deflections of simply supported members, the Chinese Design Code prescribes using the principle of minimal stiffness (i.e., that of a cracked section with M_{max}), for the lengths near the support in a beam under uniform loading or for the shear-span in a beam under concentrated loads though the stiffness is larger than that at mid-span. Minimal stiffness is determined by evaluating bending along normal sections. However, the increased influence of tensile steel strains due to the bending along diagonal sections and the influence of shear deformations should also be considered, so the above principle can be adopted along the entire beam length (Ding, et. al., 1986). For calculating the deflections of a continuous beam, the respective minimal stiffness along the lengths with maximum positive and negative moments should be used (Ding, 1989 and 1990).

For considering the redistribution of internal forces in continuous beams under service loading, with two fixed ends (for middle span) and one fixed end and one simply supported end (for end span), the first author suggested adjustment factors μ for the support moments (Ding, 1989 and 1990). In determining μ, the shift of inflection points after the adjustments of the support moment was considered, so the proposed values are more precise as compared with the values of tests in China and elsewhere, including the tests conducted at NIT in which the diagrams for deflection factors of continuous beams were given (Ding, 1989).

The allowable deflections [f] of flexural members listed in the Chinese Code, depending on span (ℓ), are given as ℓ <7m, [f] = ℓ/200 and ℓ >9m, [f] = ℓ/300.

EXPERIMENTAL STUDIES ON STIFFNESS AND CRACKS

In the early 1960s, the first author began conducting experimental studies on stiffness and cracking of RC flexural members under short-term loading. Since the late 1970s, he began testing members subjected to eccentric forces under short-term loading (Ding, et. al.,1985 and 1986; Ding, 1992a). In 1965, the first series of beam tests under long-term loading was carried out. He per-

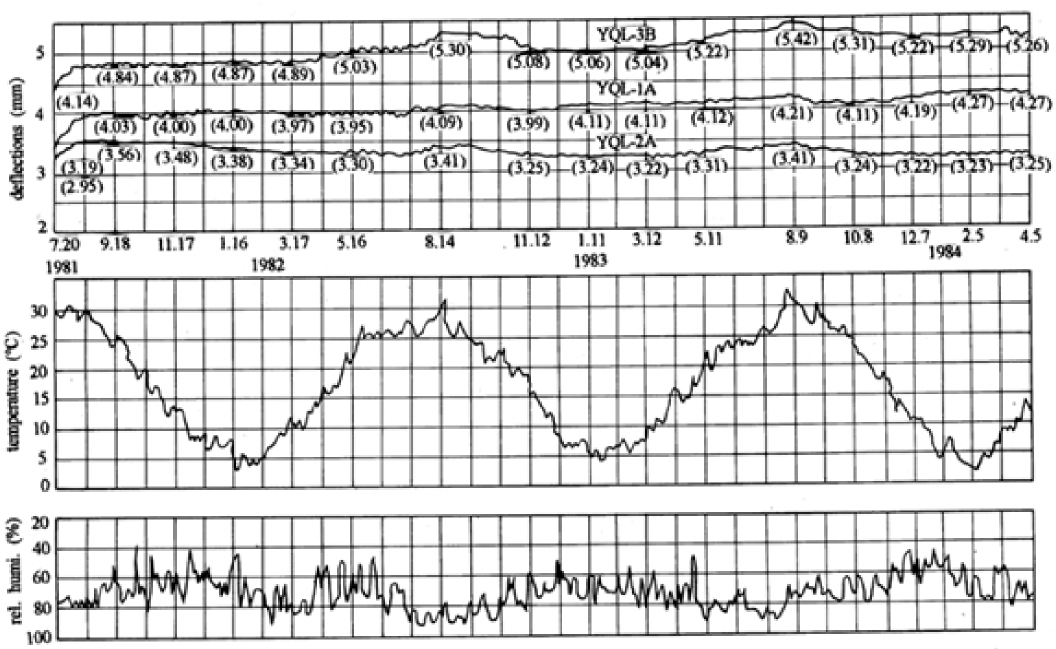


Fig.1 Results of 8th Series of Tests on 3pc Beams

formed10 successful test series of long-term experimental studies on RC beams, and another series of tests over a 23-year period of cracked and intact (without cracks) PC beams (Ding, et. al., 1985; Ding, 1992a). For the tested beams, the average strains along beam depth and the crack widths of every crack on the level of tensile steel in pure length of beams and the deflection f_0 at span center were measured, as well as their temperatures and relative humidity levels. For the first six series of tests, deflections and the temperature and humidity were recorded almost every day. The first series of long-term testing was sustained over a 6-year period (from December, 1965 to February, 1972). More complete data have been accumulated since then. Fig.1 shows the detailed records of 3 PC beams in the 8th series of tests. The curves are drawn from the test points of average values of each week.

From the large number of measured data, four conclusions can be drawn (Ding, et. al., 1985 and 1986; Ding,1992a):

(1) The average strains along section depth in pure bending length are consistent with the plane section hypothesis, not only under short-term loading, but also under long-term loading.

(2) The curves of ε_s - $\bar{\varepsilon}_s$ (ε_s = calculated steel strain at cracked section, $\bar{\varepsilon}_s$ = average steel strain measured) appear to be linear, and all the lineat curves for all beams in the same series are approximately parallel to one another, regardless of section shapes and steel ratios, i.e., the ctgø of ε_s $\bar{\varepsilon}_s$ lines (ø = slope of lines) is approximately equal to a constant, although in other series of tests the outcome may be different due to the existence of a different bond. Generally, ctgø can be taken as 1.1 (for plain bars, it should be slightly larger). Because the shrinkage of concrete in beams is relatively larger than that of other materials and the material bond is not strong, ctgø may reach a value as high as 1.3.

(3) The factor defining the section modulus for calculating the average concrete strain $\bar{\varepsilon}_s$ at the edge of the compressive zone,

$$\zeta = \frac{M}{\bar{\varepsilon}_c b h_0^2 E_c}$$

of a given section can be taken as a constant in the service stage and is the function of

$$\rho \frac{E_s}{E_c} = \rho\alpha_E$$

and section shape, where b and h_0 are the web width and the effective depth of member section, respectively; E_s, E_c are the moduli of elasticity of steel and concrete, respectively, α_E is the modular ratio and the steel ratio $\rho = A_s/bh_0$. This applies to all cases of rectangular, T, inverted T, and I –sections. A_s is the cross-sectional area of tensile steel. In annular section, $\rho = (A_s)/A$, in which (A_s) is the cross- sectional area of total reinforcement, and A is the entire sectional area of the member.

(4) After crack formation, the M-f curves for a PC beam and an RC beam with the same conditions except with the presence of pre-stressing are approximately parallel, f being the deflection at the center of the beam.

In this paper, only the calculations of flexural members with rectangular, T, inverted T and I (all with concentrated reinforcement) and annular sections (with uniformly distributed reinforcement), are considered. Members subjected to eccentric forces are not included.

From the conclusions presented in 1-3, it follows that the stiffness B_s under short-term loading can be derived as (Ding, et. al., 1985 and 1986; Ding, 1992a):

$$B_s = \frac{E_s A_s h_0^2 (or\ d_s^2)}{\psi/\eta + \alpha_E \rho/\zeta} \qquad (1)$$

where: d_s is the diameter around which the total steel reinforcement is uniformly placed in an annular section; ψ is a non-uniformity factor defining tensile steel strain, and is given by:

$$\psi = 1.1(1 - M_{cr}/M) \qquad (2)$$

M_{cr} is the cracking moment of concrete section without steel, which should be calculated using elastic theory techniques, then multiplied by the factor γ for considering the plastic deformation tensile concrete: $\gamma = 1.75$ for rectangular and T sections; $\gamma < 1.75$ for inverted T and I sections (definite values of γ are listed in the China Code); and, $\gamma = 2\text{-}0.4d_1/d$ for annular sections, where d and d_1, respectively, are the outer and inner diameters. The calculated value of M_{cr} multiplied by 0.8 is taken to consider the influence of shrinkage; η is the factor defining internal moment arm z, i.e., the distance between internal forces related to tension (T) and compression (C) in sections with cracking, where $\eta = z/h_0$; for members with rectangular., T , inverted T , and I sections reinforced concentrically, $\eta = 0.87$; for annular sections reinforced uniformly, $\eta = 1/3.2$ due to using the total value of A_s in the calculation and the higher location of the resultant resisting force in the tensile steel.

For members with rectangular., T, inverted T, and I sections,

$$\frac{\alpha_E \rho}{\zeta} = \frac{0.2 + 6\alpha_E \rho}{1 + 2\gamma'} \qquad (3)$$

where, γ' is the strengthening factor of compressive cantilever flange in a T- or I-section to the effective section of web, $\gamma' = (b_f' - b)h_f'/bh_0$; and b_f', h_f' are the width and thickness of the compression flange, respectively. For members with annular sections,

$$\frac{\alpha_E \rho}{\zeta} = 0.10 + 2\alpha_E \rho \qquad (4)$$

For cracked PC flexural members, deflections can be calculated by using conclusion 4 or another formula, suggested also by the first author (Ding, 1992b). For PC flexural members without cracks, B_s can be taken as $0.85E_cI_0$, where I_0 is the moment of inertia of the transformed section.

For long-term stiffness, Chinese design code adopts an amplification factor θ for deflections, the values of which were also given by the first author through his tests; $\theta = 2.0$ in general cases. Another formula for long-term stiffness of flexural members with the consideration of creep and shrinkage of concrete is represented by Eq.(1) (Ding, 1992b).

The formulas (Ding, et. al., 1985 and 1986; Ding, 1992a) applied to cracking in flexural members, proposed by the first author based on significant testing and analysis, are summarized here.

The general formula for average crack spacing:

$$\ell_{cr} = (1 + \gamma_{cr})(a + bd/\rho)\nu \qquad (5)$$

where: a and b are parameters, different for various shapes of members: for members with rectangular, T, inverted T, and I sections, a = 6 cm and $b = 0.06\,(1 + 2\gamma_1 + 0.4\gamma_1')$, $\gamma_1 = (b_f - b)\,h_f/bh$; b_f and h_f are, respectively, the width and thickness of a tensile flange and h is the overall depth of the section, and $\gamma_1' = (b_f - b)\,h_f'/bh$. For annular sections, a = 0.5s and b = 0.1, where s represents the stirrup spacing, which should not be less than 10cm and more than 20cm. γ_{cr} represents a special factor for tension members; $\gamma_{cr} = 0$ for flexural members and members in eccentric compression. In this paper only the calculations for flexural members are introduced, and therefore, γ_{cr} and the other relevant parameters will be neglected.

Calculated values of ℓ_{cr} were checked with those obtained from a large number of tests by the first author and were found to be in good agreement. The average crack width can then be calculated as follows:

$$w_{cr} = \psi(\sigma_s/E_s)\ell_{cr} \qquad (6)$$

$$\sigma_s = \frac{M}{A_s \eta L_0 (ord_s)} \qquad (7)$$

Finally, ν is a factor for considering surface configuration of steel bars: for plain bars, $\nu = 1.0$, and for deformed bars, $\nu = 0.7$ (in fly-ash ceramsite concrete $\nu = 0.8$).

When considering the non-uniform distribution of cracks and the influence of long-term loading,

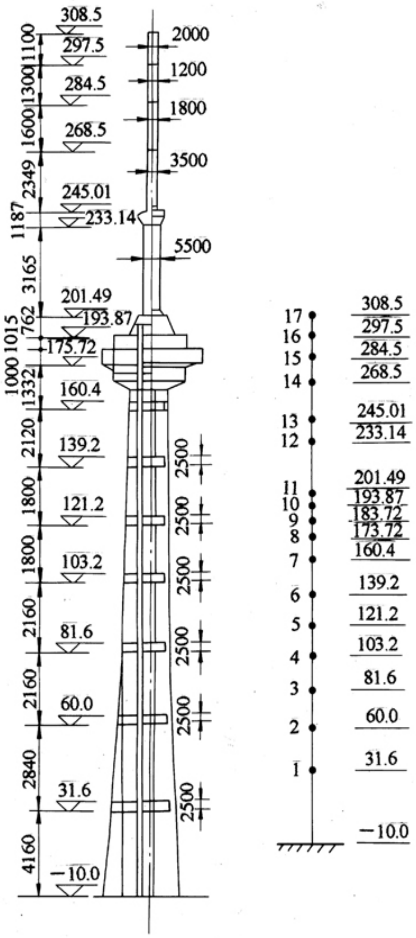
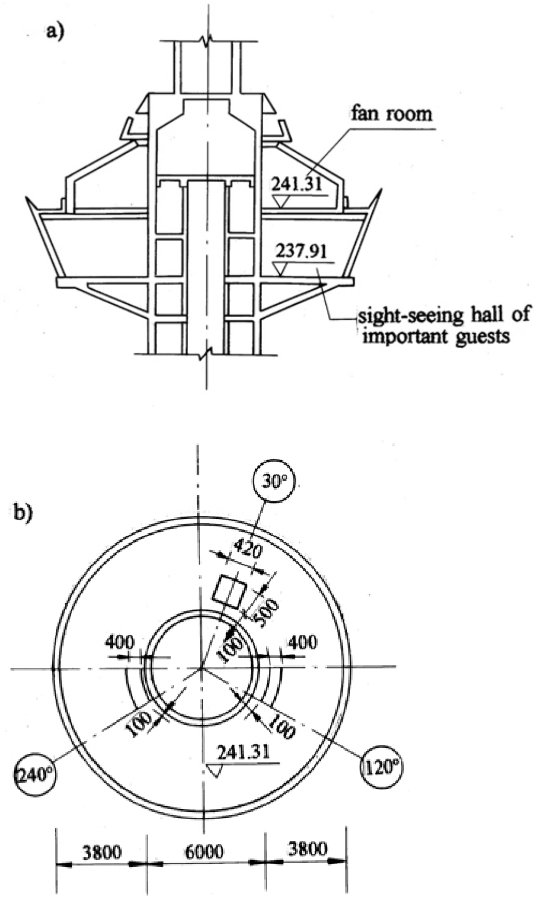


Fig. 2 Nanjing TV Tower



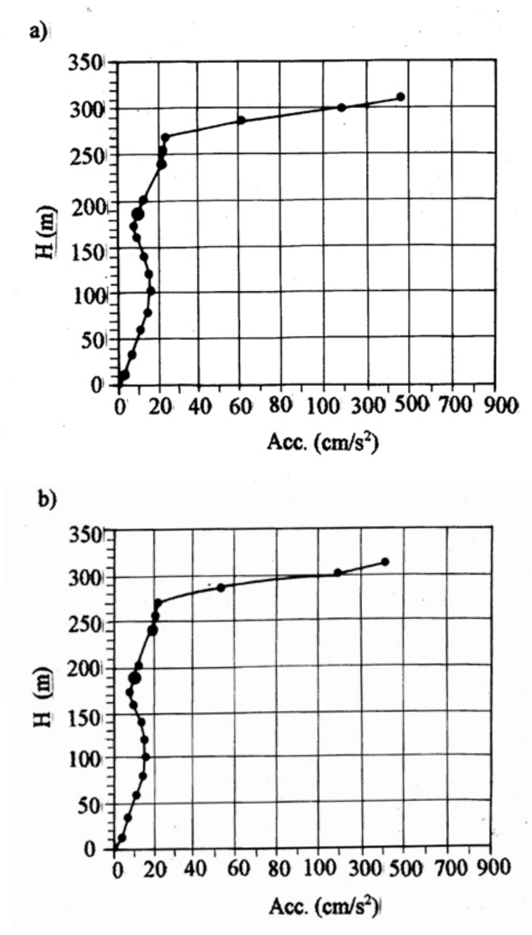
Figs. 3a and 3b Section (a) taken through Sky-Cabin and Floor Plan (b) of Fan Room.

test results revealed that w_{cr} should be multiplied by 2.0.

These proposals for calculating stiffness and cracking of flexural members with rectangular, T, inverted T, and I sections were accepted by the previous Chinese Design Code TH10-74 and are used continuously in the current code GBJ10-89 after some adjustments.

SERVICEABILITY REQUIREMENTS UNDER DYNAMIC EXCITATIONS

It is necessary to limit the accelerations of the sky cabins of tall TV towers subjected to seismic and wind actions so as not to cause discomfort to visitors and operators. In the PC spatial frame structures of the 318m high Nanjing TV Tower (Fig.2), for example, two sky-cabins, one large and the other small are supported by 3 PC branches (Ding and Maoquan, 1995; Ding and Zhenhua, 1994). The branch of Nanjing Tower is a box section with multiple cells (in the bottom part of the tower, there are 4 cells which are gradually transformed into 3, and then 2 cells at the junction with the large sky-cabin due to the battering of the branch). The Nanjing TV Tower is generally recognized as the first tall space-frame PC tower in the world (Ding and Maoquan, 1995). For verifying the maximum acceleration of sky-cabins under wind-



Figs. 4a and 4b Acceleration Diagrams of Sky-Cabin subjected to Dynamic Wind Loads before (a) and after (b) installation of Tuned Liquid Dampers

pulses, one doctoral student and two master degree students, supervised by the first author at NIT, studied the dynamic behavior of this tower (Zhenhua, 1993; Meixiao, 1993). It was also investigated for seismic actions (Lin, 1993).

Following the stipulation of the Chinese Code, the acceleration of tower sky cabins under wind-pulses should not be over 0.15m/s^2. Ding Dajun and Ren Zhenhua adopted artificial pulse samples to conduct direct dynamic analysis and determined the accelerations of the small and large sky-cabins to be equal to 0.209m/s^2 and 0.1m/s^2, respectively (Ding and Zhenhua, 1994; Zhenhua, 1993). From the calculations it can be shown that the acceleration of the small sky-cabin under wind-pulse loading cannot meet the serviceability requirement. The upper level of the small sky-cabin is reserved as a mechanical floor (Fig.3a) and the lower level is used as a sight-seeing floor for distinguished guests (the general sight-seeing hall is located in the large sky-cabin). For controlling the acceleration of the smaller sky-cabin, Reng (1993) suggested to set Tuned Liquid Dampers (TLDs) with diameters of 2.4m and 3.5m on the floor of the fan room (Fig.3b). The dynamic analysis in Fig. 4a shows the enveloping curve of the maximum accelerations of each mass point before the addition of control dampers. The envelope of maximum acceleration of each mass point after the installation of control dampers is shown in Fig. 4b (Zhenhua, 1993) where the acceleration of the small sky-cabin is reduced to 0.183m/s^2 (Ding and Zhenhua, 1994). Even with dampers, the controlling effect is only 12.4% and is still below the service requirement. This problem should be studied further.

Cheng Mexiao's master's thesis (Meixio, 1993) studied the use of Tuned Mass Dampers (TMDs) for controlling the pulsating wind response of the Nanjing TV Tower. She pointed out that the controlling effect for the small sky-cabin is larger with the increase of the mass of the TMD system. However, she also found that if the economic mass ratio (i.e., the ratio of the mass of the TMD system to that of the small sky-cabin) of 0.05 is used, then the optimum damping ratio is 0.1 and the optimum frequency ratio (i.e., the ratio of frequency of TMD to the 1st frequency of the structure) is 1.037.

An Lin's thesis (Lin, 1993) applied the TMD system to control the vibration due to wind-pulse and obtained the optimum controlling parameters by using the optimum controlling program. Fig. 5 shows the time-history-acceleration response curve before and after controlling (Lin, et. al., 1995). The maximum acceleration of the small sky-cabin is 0.24m/s^2 and is reduced to 0.21m/s^2 after controlling. The controlling effect is 12.5% which is the same as given in by Ding and Maoquan

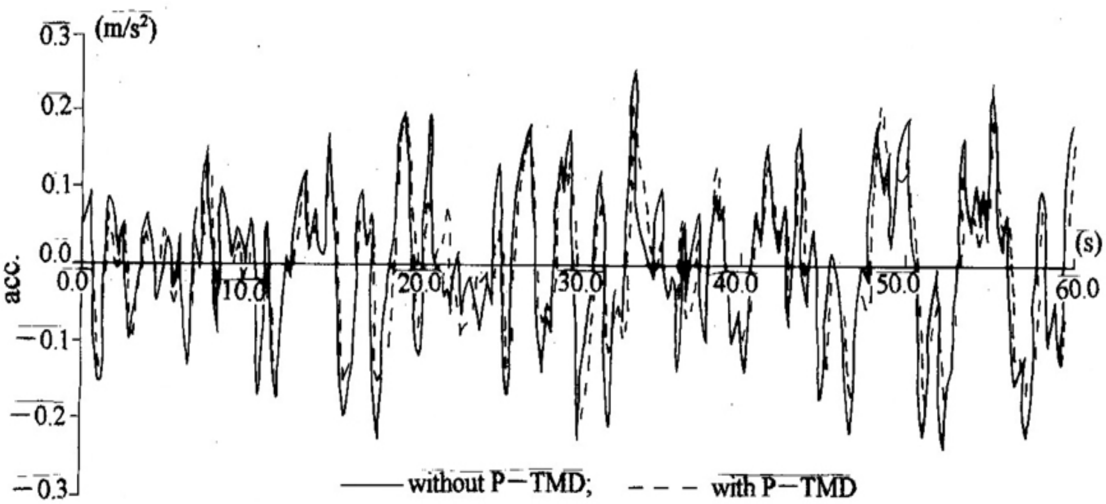


Fig. 5 Analysis with and without Tuned Mass Dampers installed in the Nanjing TV Tower

(1995) and Ding and Zhenhua (1994). The maximum acceleration is slightly different due to the fact that the wind-pulse sample used and the masses calculated were slightly different. The seismic action was also investigated and the controlling effect reached 50% (Lin, et. al., 1995; Lin and Ding, 2000).

CONCLUSIONS

For the calculation of stiffness and cracking of concrete members under static loading, the first author suggested a series of proposals, which are applicable to rectangular, T, inverted T, I, and annular sections currently used in engineering practice. They also can be used for RC, PC, and light-weight concrete members subjected to flexure and eccentric forces under short- and long-term loading. This method of calculation appears to be more convenient, seems to provide a high degree of accuracy (Ding, 1989, 1990, 1992and 1992b; Ding, et. al., 1985 and 1986) and can satisfactorily meet design requirements. It is important to note that the study of the Nanjing TV Tower – a tall structure subjected to dynamic actions – is preliminary and, therefore, it is necessary to conduct more research before definitive conclusions can be drawn.

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