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INELASTIC STRESS-STRAIN BASED SEISMIC RESPONSE PREDICTION OF RC STRUCTURES CONSIDERING DYNAMICALLY VARYING AXIAL FORCES

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SUMMARY

This paper presents a powerful stress-strain based analytical model and analytical procedure which can be used to predict inelastic earthquake response of reinforced concrete structures considering in the analysis dynamically induced interactive effects of bending and time-varying axial forces. Refined material discretization and sophisticated analytical representation of the observed hysteretic behaviour of concrete and reinforcing steel have been introduced to accurately model the induced material nonlinearity of an isolated RC member. Capability and practical applicability of the proposed model is demonstrated based on the results from the extensive on-line HYLSER laboratory tests and respective theoretical studies.

INTRODUCTION

Following the needs of increasing modernization and advances in the construction technology, numerous large-size and new types of engineering structures of the highest social and economic importance, such as PC cable-stayed bridges, suspension bridges, long-span and multi-purpose bridges, industrial and off-shore facilities, high-rise buildings, etc. have been designed and constructed in known seismically active regions. Since these structures are primarily supported by reinforced concrete columns or piers, their seismic resistance may be strongly influenced by the dynamically induced highly varying axial forces (Ref. 5 and 9). Drastic variation of axial forces, in RC critical elements, interacting with bending effects during earthquake induced intensive structural vibration produces a significant change in the structural overall inelastic response, and the assumption of constant axial forces in such cases may be quite inadequate. The assumption of constant axial forces during seismic structural response appears as essential limitation of all the proposed nonlinear mathematical models formulated based on analytical simulation of hysteretic force-displacement relations for the isolated RC elements. The developed new inelastic stress-strain based mathematical model (Ref. 7) and analytical procedure which can be used to predict nonlinear seismic response of RC structures considering the effects of dynamically varying axial forces in the analysis is discussed herein.

STRESS-STRAIN BASED NONLINEAR FINITE ELEMENT

In many of the previous studies, analytical stress-strain (SS) models for confined concrete (CC), unconfined concrete (UC) and reinforcing steel (RS) were to a large extent simplified and limited to the application of simple elastic-perfectly plastic models, bilinear models, piece-wise models, etc. in which case, disregarding many of the existing phenomena characterizing the real SS relations, the analysis accuracy has significantly been limited.

In this study, the authors adopted significantly improved SS hysteretic material models based on the available experimental results from the successful laboratory tests. Following the investigations by Muguruma, et al. (Ref. 1), adopted is a quite realistic CC and UC stress-strain model (Fig. 1) simulating all the basic features of the hysteretic concrete behaviour under cyclic loading recorded from the laboratory tests (concrete strength in tension, compressive stiffness degradation, hysteretic energy absorption for the repeated cycles, concrete failure in tension and in compression, etc.) through incorporating in total nine rules or time-dependent stress-strain paths. Similarly, to realistically simulate the steel behaviour under cyclic loading, the originally proposed stress-strain hysteretic model by Giuffre and Pinto, simulating the Bauschinger effects and later improved to include isotropic strain hardening (Ref. 2), have been presently implemented (Fig. 2). Details of the rules for the adopted material stress-strain models are presented in reference (7).

Introducing separate analytical models for the confined concrete, unconfined concrete and steel material, the so called stress-strain based nonlinear multi-interface finite element (SS-NONMIFE) of RC member, Fig. 7(b), have been formulated and the corresponding stiffness matrix computed by integration considering the material discretization, the current element state and the incremental element end deformations, Fig. 3. The three local element degrees of freedom consist of the two end rotations along with the axial deformation. The computation of the nonlinear beam element stiffness matrix presently involves the following steps: (1) dividing the beam element along its axes into smaller parts (sub-elements), (2) computing the increments of axial strain $\Delta\varepsilon_a$ and curvature $\Delta\phi$ at the centroidal point of each dividing cross-section (interface element), corresponding to the incremental element end forces, Fig. 3 with the assumption that the respective axial force is constant and bending moment varies linearly, (3) dividing the interface element into discrete areas (fibers) corresponding to the CC, UC and RS material, Fig. 7(c), (4) computing the total strain at each discrete area centroid ε_i assuming that the section before and after deformation remains plane, (5) getting the current tangent stiffness E_i of each discrete area considering the adopted hysteretic material rules, (6) computing the 2 x 2 interface element stiffness matrix $[K]_{i.e.}=[K]_s$ relating its current incremental forces $(\Delta M, \Delta N)_s$ and deformations $(\Delta\phi, \Delta\varepsilon_a)$, (7) computing the interface element flexibility matrix $[f(x)]_s$, by its inversion, (8) computation of the element local flexibility matrix $[F]_e$ by integration along the elment length, (9) inversion of $[F]_e$ to obtain the local element stiffness matrix $[K]_e$ relating the current incremental element forces and deformations, and then further steps directly follow the standard finite element and matrix analysis procedure (Ref. 3 and Ref. 10).

ON-LINE HYLSER TESTS AND MODEL VERIFICATION

To provide the experimental evidence of actual inelastic response characteristics of the reinforced concrete structures subjected to real earthquake ground motion and simulated high time varying axial loads for verification of the proposed model, extensive laboratory tests have been performed by using the newly developed On-line computer controlled hybrid loading system for earthquake response (HYLSER), schematically shown in Fig. 4, where earthquake response is calculated by digital computer (Eq. 1), adopting the real specimen's hysteretic restoring force F(u), directly measured from a loading actuator, Ref. (4) and (8).

$$M\ddot{u} + C\dot{u} + F(u) = - m\ddot{z} \tag{1}$$

Reinforced concrete specimens were designed with cross section of 150x150mm, total length of 2090mm, standard longitudinal reinforcement of 4Ø12.7mm, spiral type lateral reinforcement (Ø=5mm) with pitches of 60 and 90mm, and varied concrete compressive strength corresponding to normal and high-strength concrete. Component N-S of El-Centro record (18-05-1940) is used as input earthquake while varying compressive axial force is controlled to be proportional to restoring force F(u). Details of the developed HYSLER system (Fig.4) and conducted tests may be found in references 5 and 7. Generally, quite unique features of hysteretic loops due to time varying axial forces were found. At the moment positive side (increased axial force), high but deteriorating restoring force is found, while at the moment negative side (decreased axial forces) lower but undeteriorating restoring force was recorded.

Since moment-curvature relations were measured for critical cross-section, presently proposed stress-strain analytical model has been evaluated based on direct comparison of recorded (a) and predicted (b) moment-curvature relations (Ref.6), as shown in Fig.5 and Fig.6 for specimens D-1 and D-9 considering 30 sec earthquake time-duration. As in this two, comparative analysis in all other cases showed quite good agreement of complete pattern of the hysteretic loops as well as the corresponding moment and curvature (recorded and computed) levels, which actually encouraged further extension of the applied stress-strain based modeling concept to inelastic earthquake response prediction of the integral RC structures.

MODELING AND INELASTIC EARTHQUAKE RESPONSE ANALYSIS OF STRUCTURAL SYSTEMS WITH EXAMPLES

Since stiffness matrix of RC member (SS-NONMIFE) is time dependent, for multi-degree-of-freedom system earthquake response is computed by step-by-step solution of the dynamic equilibrium equation written in incremental form (Eq.2) considering constant structure stiffness for the small time interval Δt(Ref.3 and 10).

$$[M]\,{}^{t+\Delta t}\{\ddot{u}\}+[c]\,{}^{t+\Delta t}\{\dot{u}\}+{}^{t}[K]\{u\} = {}^{t+\Delta t}\{R\} - {}^{t}\{F\} \qquad (2)$$

where $[M]$ and $[C]$ are mass and Rayleigh damping matrix, ${}^{t}[K]$ and ${}^{t}\{F\}$ stiffness matrix and vector of nodal point forces corresponding to element stresses at time Δt, ${}^{t+\Delta t}\{\ddot{u}\}$, ${}^{t+\Delta t}\{\dot{u}\}$ and ${}^{t+\Delta t}\{R\}$, vectors of nodal point accelerations, velocities and external forces for time $t+\Delta t$, and $\{u\}$ displacement increments between time t and $t+\Delta t$. In the computer program NORA developed in this study, both Newmark's and Wilson's step-by-step numerical integration schemes have been employed to compute inelastic structural response discretized by the presently developed (SS-NONMIFE) finite element. Generally, some iterations are required for developed unbalanced nodal point forces during small time increment to obtain improved solution. However, in the folloeing demonstrative numerical example of RC bridge tower structure, assembling of new structure stiffness matrix have been considered at every time step (t=0.005 sec) and no additional iterations have been carried out.

As shown in Fig.7, discrete stress-strain based model of example RC bridge tower structure, 120 m in height, consists of 12 nodal points (a), 12 SS-NONMIFE finite elements (b) which are further discretized by 6 sub-elements and 7 interface elements (c) considering 10 steel and 20 concrete areas, i.e. SS material monitoring points. Assuming fixed boundary conditions at nodes 1 and 2, the present structure model include in total 30 degrees of freedom and 2520 stress strain monitoring points, Fig.7.

To demonstrate capability of the present stress-strain based model for inelastic earthquake response analysis of RC structures considering dynamically varying axial forces in total 3 separate nonlinear analyses have been performed using 3 different earthquake intensity levels EQI=1, 2 and 3 and the same earthquake record, while in the second and the third one, earthquake record is amplified by factors 2 and 3. To demonstrate the difference between computed structural response by linear model (LM) and nonlinear model (NM), linear earthquake response of the structure have been also computed using the same model and EQI=3. In all present dynamic analyses, only earthquake ground motion effects are analysed (10s) and no initial dead load stress-strain state is considered. Assuming element and additional nodal point masses, calculated initial dynamic characteristics, i.e. periods for the first four modes are T_1=2.260s, T_2=1.528s, T_3=1.036s and T_4=0.447s. In Fig.8 through Fig.15 are presented selected results of the computed earthquake responses demonstrating the most essential features of the model applicability and respective earthquake intensity effects as well as corresponding tendency of computed responses applying LM and NM, respectively. Fig.8 presents: (a) displacement and velocity response, and (b) acceleration response of NP=12 in x-direction for EQI=3, while Fig.9 presents moment (a) and axial force (b) with curvature response at EL=1 NP=1 for EQI=3 where highly varying axial force have been induced interacting with bending moment as shown in Fig.12 for EQI=1. Fig.10 shows moment response obtained by LM and NM at EL=12 NP=11 for EQI=3, clearly expressing increasing period of vibrations due to the structure stiffness degradation. The same tendency of the material and structure stiffness degradation in case of increased earthquake intensity is very well expressed in Fig.11 including plotted displacement response of NP=12 x-x for EQI= 1, 2, 3, which result in case of stronger earthquakes in inelastic and more open hysteretic moment-curvature responses as evident from Fig.13, Fig.15 and Fig.14 for NP=11 and earthquake intensities EQI=1, 2 and 3, respectively.

CONCLUSIONS

In this paper a refined stress-strain based modelling concept is presented for inelastic earthquake response analysis of RC structures considering in the analysis dynamically induced varying axial forces, and the following conclusions are obtained: (1) analytically estimated moment-curvature relations from stress-strain relations addopted in this study agree very well with experimental HYLSER results of earthquake response, (2) dynamically induced highly varying axial forces has a nonnegligable effects on M - ϕ relations of structural sections and the seismic response of the integral RC structures, (3) for some specific types of engineering structures high variation of axial forces may be induced, as evident in present example, and (4) the developed stress-strain based model and analytical procedure may be applied as sophisticated tool to carry out detalied inelastic earthquake response analysis of RC structures where consideration of the interactive effects of bending and varying axial forces is highly important.

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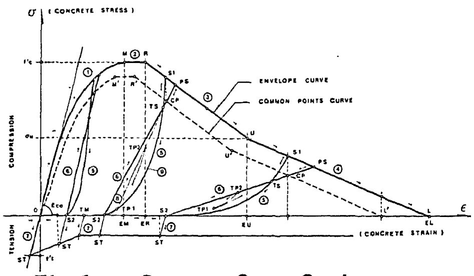

Fig. 1. Concrete Stress-Strain Hysteretic Model

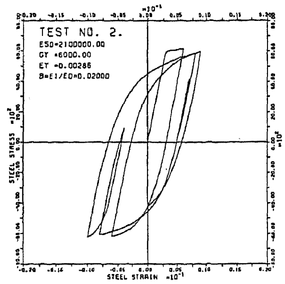

Fig. 2. Steel Stress-Strain Hysteretic Model

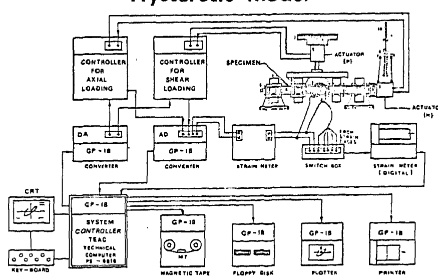

Fig. 4. On-Line HYLSER Laboratory Testing Procedure

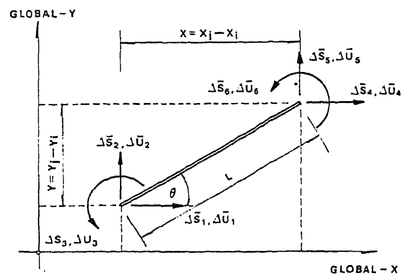

Fig. 3. Element Global Incremental Forces and Deformations

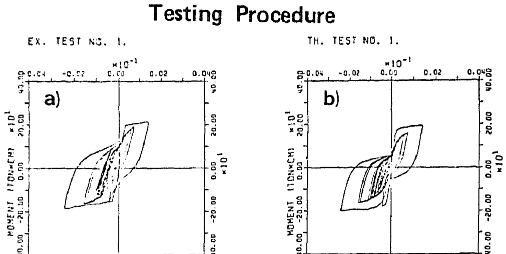

Fig. 5. Specimen D–1 : Experimental and Predicted M–Φ Response

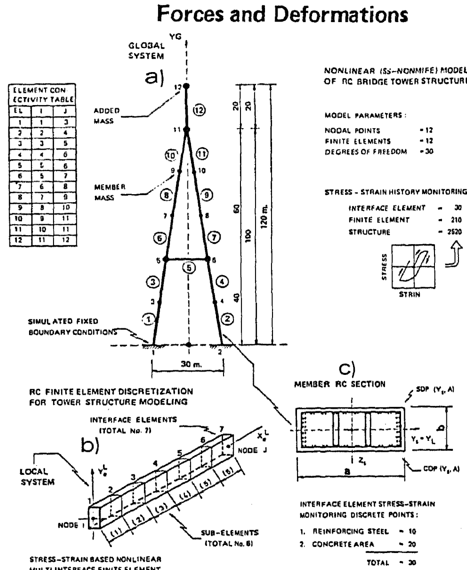

Fig. 7. Dicrete Model of RC Bridge Tower Example Structure

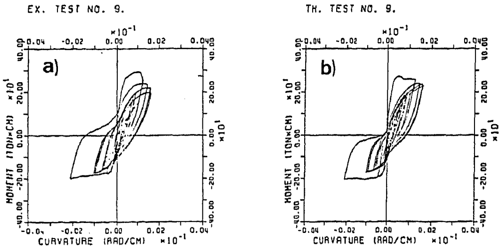

Fig. 6. Specimen D–9 : Experimental and Predicted M–Φ Response

RC BRIDGE TOWER H=120.M EL-CENTRO F=3.00

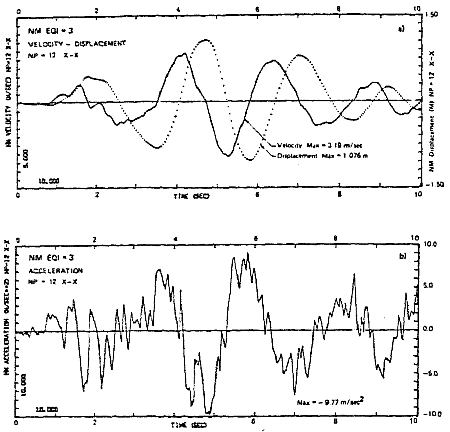

Fig. 8. Complete Response History of NP = 12 x-x for EQI = 3

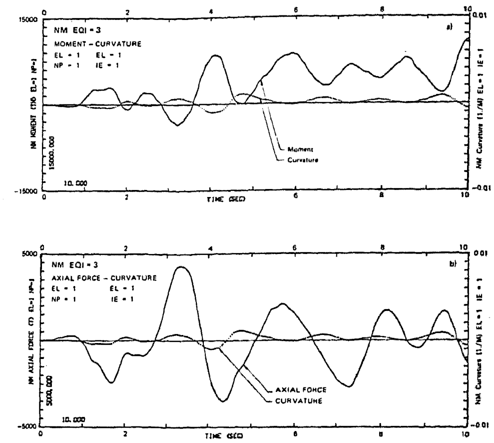

Fig. 9. M—Φ adn N—Φ Response at EL = 1 NP = 1 for EQI = 3

RC BRIDGE TOWER H=120.M EL-CENTRO L-N-3

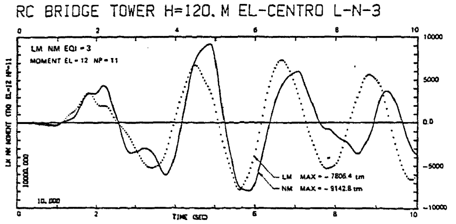

Fig. 10. LM and NM Moment Response at EL = 12 NP = 11 for EQI = 3

RC BRIDGE TOWER H=120.M EL-CENTRO 1-2-3

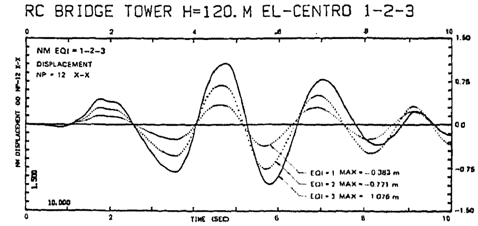

Fig. 11. Displacement Response at NP = 12 x-x for EQI = 1-2-3

RC BRIDGE TOWER H=120.M EL-CENTRO F=1.00

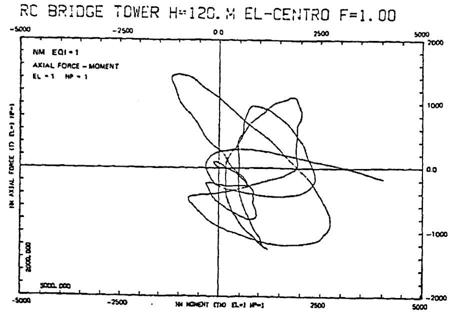

Fig. 12. Relation of N—M Response at EL = 1 NP = 1 for EQI = 1

RC BRIDGE TOWER H=120.M EL-CENTRO F=1.00

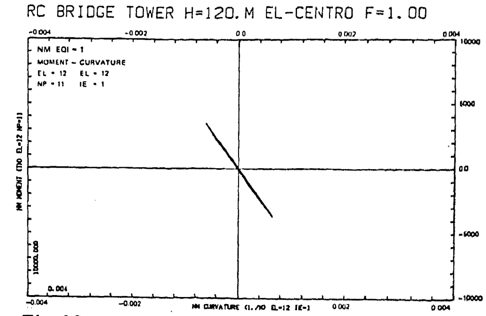

Fig 13. M—Φ Response at EL = 12 NP = 11 for EQI = 1

RC BRIDGE TOWER H=120.M EL-CENTRO F=3.00

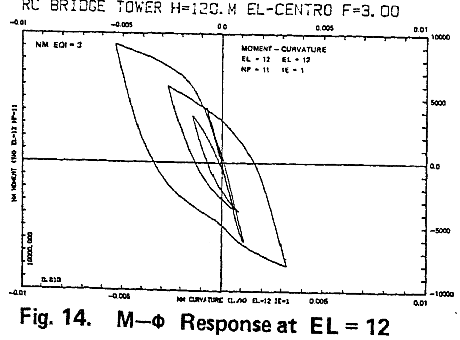

Fig. 14. M—Φ Response at EL = 12 NP = 11 for EQI = 3

RC BRIDGE TOWER H=120.M EL-CENTRO F=2.00

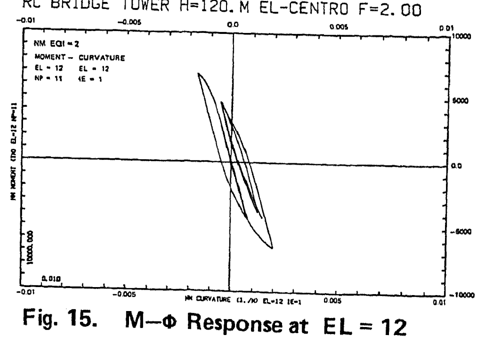

Fig. 15. M—Φ Response at EL = 12 NP = 11 for EQI = 2