Structural Engineering Documents

2

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DYNAMIC RESPONSE OF REINFORCED CONCRETE BUILDINGS

International Association for Bridge and Structural Engineering IABSE
Association Internationale des Ponts et Charpentes AIPC
Internationale Vereinigung für Brückenbau und Hochbau IVBH

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CONTENTS

1. INTRODUCTION

As the necessity of a reliable evaluation of the structural safety against destructive earthquakes is increasingly felt, more attention has been given to the dynamic performance of building frames deformed into their ultimate states. The primary concern therein is a better understanding of the process of failure by relating the mechanics of structure to the characteristics of intense ground shaking. This type of question is being asked repeatedly in the formulation of earthquake-resistant building standards as well as in the design of specific important structures.

In terms of the dynamic structural analysis, an important step forward to meeting the current need will be taken by reflecting the nonlinear structural behavior in a manner which is in a sufficient agreement with known data of experiments and observations. Particularly strong-motion response studies along this line can be of a practical importance for lowrise reinforced concrete (R/C) buildings, because this class of structure is comparatively vulnerable to earthquake excitations. The vulnerability has been demonstrated by their critical damage sustained in recent earthquakes. The present report, which briefly outlines the important aspects in modeling the dynamic failure of R/C frames from the particular point of view, is intended to provide a summary of the progress having been made during these twenty years. In the 60's and 70's, the wide availability of high speed electronic computers and the associated development of numerical techniques have had a significant contribution to the ease of carrying such studies. Also, the 1968 Tokachi-oki earthquake in Japan and the 1971 San Fernando earthquake in California were epoch-making events in this field of structural engineering. References [1, 2]* are among the review papers related to the current subject.

In formulating the strong-motion response behavior of a structural system, the nonlinearities involved are well separated into two main categories. One is the so-called "material" nonlinearity that refers to the inelastic and hysteretic properties of restoring force arising from the hereditary stress-strain relations of constituent elements. The other is the geometrically nonlinear effects associated with the deformed configuration of system. For the R/C framed structures of practical interest, consideration to the former factors alone suffices for attaining most of the present objective, while the latter may become fully responsible for the system destabilization in the limited region near ultimate collapse.

Reasonably close identification of the inelastic and hysteretic performance of R/C buildings and its adequate mathematical modeling can be indeed a difficult practice. Throughout the current report, an empirical approach to a set of generalized nonlinear formulations is emphasized on the basis of the direct use of the member properties derived from experiments. With differing levels of complication, this leads to a straightforward, but often cumbersome, member-by-member modeling of total system, or to a considerably simplified modeling that incorporates the restoring force characteristics of members. To use stress-strain formulations for the entire structure is judged less desirable, since such approaches will be far more unwieldy without any compensating improvement in reliability. It is in fact probable that when accounting for the inelastic and hysteretic action of the stress-strain models with current concepts and methods the laborious analysis will not describe actual behavior as well as the

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* Numerals in brackets indicate the reference number in the bibliography.

member-by-member models can.

A synthetic view of the whole problems related to the member-by-member formulations is included in this report, which indicates necessary fundamentals for the in-depth study of dynamic behavior. An elaborate analysis of the current type can positively reflect all minor details of structure, and there is one-to-one correspondence between the components of an actual frame and its idealized system. By use of realistic analytical models for the constituent elements, this will greatly contribute to predicting or reproducing, with a desired accuracy, the behavior of a specific building under severe earthquakes.

On the other hand, the interest of analysis is frequently taken in the overall performance of structure disregarding the microscopic local effects. A necessity also arises to examine the crude but general nature in the strong-motion response of a certain group of buildings. In such instances, simpler and gross models of total system are preferred so long as the reduction does not impair the essential features in the nonlinear dynamic response. Some of the work in the current area have proceeded along the direction of developing a macroscopic model that results, with relatively small efforts, in a satisfactory estimate of the significant trends. In addition to the foregoing complex modelings, another emphasis of the discussions made in the present report is put upon this aspect of simple formulations.

2. PRINCIPLES OF MEMBER-BY-MEMBER ANALYSIS

The evident advantage of a member-by-member analysis lies in the quantitative assessment of overall structural safety with a direct reference to the process of the damage sustained by each constituent element. Provided that complications in the procedure of computation pose no problem, this is best fitted to interpret the dynamic failure of a particular building due to a destructive ground motion. Also the analysis can permit to predict the earthquake-resistant capacity of existing buildings by reflecting their specific minor details.

Since the advent of high speed electronic computers, rapid progress has been made in the general area of nonlinear stress analyses. Most of the researches are however found within the traditional context of structural or solid mechanics. Even though steel frames may be sufficiently analyzed by means of such approaches, the member-by-member analysis of R/C frames is under considerably different situations. For instance, the conventional analysis of material nonlinearity often fails to evaluate, with a practical sufficiency, the inelastic deflection of R/C structures.

Inelastic and hysteretic performance of R/C frames would be clarified on the basis of the fundamental principles of mechanics, if the material properties of concrete and reinforcement were realistically modeled under arbitrary triaxial stress history and then combined with such displacement discontinuities as flexural and/or shear cracks, crushing and spalling, bond slippage and dowel action. Not a few studies along this line can be found in the literature, and the analyses have been partly applied to examining the results of the loading test of structural components. However, the current state of the art is far from being satisfactory; ad hoc rationalizations have been frequently used for taking account of complicated and unclarified factors. It may be also affirmed that the overly detailed characteristics obtained from the nonlinear analyses of stress and strain fields in the interior of an individual component are under arbitrary loading history. The second step deals with the extension of the characteristics thus formulated into those under more general conditions of loading. As explained above, this is intended to take account of the actual states in a loaded frame. In the third step, the effect of the deformation rate of member is treated with a specific emphasis upon the damping forces in non-elastic region.

3. STIFFNESS, STRENGTH AND DEFORMABILITY OF ELEMENTS

With the aforementioned principles in mind, a basically empirical approach is taken to study the properties of R/C framing members. From this standpoint, it becomes desirable that the important parameters describing the inelastic and hysteretic behavior of constituent elements are given in the form of empirical equations. These should include relations between end-moment and end-rotation for flexural or flexural-shear columns and girders including the flexure of shear walls, those between shear force and shear deformation for shear-failure-type columns and shear walls, and those between panel distortional moment and shear distortion for beam-to-column connections. Some of the load-deformation relations are simply illustrated in Fig. 1.

Having been derived by reviewing the data of monotonic loading tests, a number of empirical equations for evaluating the skeleton curve parameters are available for various structural elements and for different modes of failure. Such studies are particularly found in Japanese literature; the reference [6] which examined the plastic state of R/C beams in flexure is among the pioneering works. Of those equations, a self-contained group developed by Umemura, Aoyama and their coworkers [7 to 9] has been widely used for practical purposes. Following the guiding principle referred to in the previous chapter, they made a series of investigations for correlating the empirical characteristics of typical elements with a certain set of significant factors. Although being of a nature of pilot study, this includes the skeleton curve parameters in flexural (flexural-shear) failure of girders and columns, and shear failure of beam-to-column connections and shear walls.

Even the reliable characterization of the failure process of girders and columns cannot be simple. Part (a) of Fig. 1 shows the members subjected to antisymmetric loading with the single varying factor of bending moment and shear, and under invariant axial force. Depending upon the specimen details, their ultimate states are differently characterized by the phases of flexural yielding, shear failure, bond failure, crushing and spalling of concrete, steel buckling, and so on. A total review on this subject is beyond the scope of the current report and should be best left for more specialized literature. The following discussions begin with the relatively simple evaluation of their flexural (flexural-shear) parameters.

In most cases of practical interest, the load-deformation diagram is featured by several breakpoints, the skeleton curve tending to become a polylinear type rather than a single rounded one. These points can be interpreted to reflect the flexural cracking of concrete in tension, the development of flexural or flexural-shear crack of concrete, the yielding of tensile longitudinal reinforcements and the compression failure of concrete, all of which occur at and near their critical sections. Associated with the cracking, moderately reinforced members exhibit a sharp break away from the elastic branch at a very low level of bending moment as compared to the yielding of reinforcements. Also, their moment-bearing capacity does not increase appreciably beginning in the 1930's. Recent studies [215 to 217, 182, 218 to 220, 193, 221 to 231, 111, 232] have included the effects of gravity or nonlinear geometry into both member-by-member and simplified analyses. The trend was toward more reliable examinations of the completely nonlinear behavior near collapse. One of their important aspects, which is the inclusion of the gravity term into the equation of overall structural performance, is left for the discussions made in a later chapter.

The geometrically nonlinear column has been mathematically modeled in different levels of sophistication. Considerable ease of formulation is provided under a simplifying condition that no axial deformation is developed along the member. It is a usual practice to linearize the P-Δ term with respect to lateral drift [e.g., 182], while its trigonometric expression can be found in the study of Jennings and Husid [216]. This term is specified by means of the so-called geometric stiffness matrix in the formulation of frame analysis. With different modes of deflection assumed along the longitudinal axis, two techniques of linear and Hermitian interpolations for determining the matrix have frequently appeared in the literature. Consistency with the deflection curve evaluated from the concurrent analyses of material nonlinearity is not necessarily reflected upon this type of modeling. In addition to the above matrix synonymously called initial stress matrix, contribution of another additional term represented by initial or large displacement matrix becomes relevant to the incremental formulation of completely nonlinear columns. A more fundamental treatment of the finite deformation follows the use of Green's strain and Kirchhoff's stress in the general area of solid mechanics.

6. ADDITIONAL ASPECTS IN MEMBER-BY-MEMBER MODELINGS

In succession to the previous chapter, discussions proceed to summarize some problems important for member-by-member analyses. These are panel element formulations of beam-to-column connection and shear wall, slipping interaction between structural components, and uplift of foundation. Treatment of viscous damping forces in the evaluation of inelastic and hysteretic response is also examined with relation to the properties of dynamically loaded elements.

During the last two decades, the local behavior of beam-to-column connection zones has attracted considerable attention in the area of steel and R/C structural engineering. It is a generally accepted understanding that their resistance to shear can have an appreciable influence upon the overall performance of structure. For R/C frames of practical interest, ultimate failure at the connection zones will be improbable and thus the contribution of their shear distortion to the overall deflection may not become so significant after the yielding of framing members. The joint distortion however acts as an important factor to increase the flexibility of elastic system. Moreover the zone tends to soften markedly due to shear cracking.

In practical terms of frame analysis, the joint area has been modeled by a panel of certain geometrical volume deformed by the uniform action of shear [233 to 239, 193, 240 to 242, 174]. The classical notion of the rigid zones that suppress flexural and shear deformations at member ends is being accounted into this type of idealization. Its four-degree-of-freedom displacement field consists of combined rigid-body displacements (horizontal and vertical translations, and rotation) and shear distortion. The contribution of this shear deformation to the deflection of R/C frames can be closely the softening condition of $K/K_e < 1$, their relative magnitudes of instantaneous damping lie obviously in the order of (Type I) < (Type III) < (Type II).

Nothing definite is known on the question which of the three types provides a relatively realistic formulation. However, it appears that the damping in the non-elastic stage becomes a little larger than modeled by Type I and is close to Type III. Most commonly used in the literature is the "tangent stiffness proportional damping" of Type I. This probably leads to the underestimation of the effect of damping after non-elastic action has taken place. Application of the damping of Types I and II to a multi-DOF formulation can be straightforward. Also the Type III damping for a multi-DOF system was examined in the reference [164] by use of a certain approximation of oscillation mode.

Apart from the reliability of the various idealizations of viscous damping, it will be informative to illustrate the resulting dynamic response. One example calculation for this purpose is shown in Fig. 6, where a 1-DOF degrading trilinear system is subjected to an earthquake motion. In the case of $\zeta_e = 1$ %, the discrepancies that result from the different assumptions are seen to be relatively small. This is particularly the case between Type I and Type III. On the other hand, serious differences feature the case of $\zeta_e = 5$ % according to the higher intensity of excitation.

The damping of $\zeta_e = 1$ % may be used to approximate the inherent damping capacity of structural components or their subassemblages as noted previously. Thus the above illustration indicates not so crucial a choice for the separate formulation of damping in a member-by-member analysis. On the contrary, the results highlight the use of considerable caution in the simpler modeling of the proportional damping. This is because the apparent damping capacity of total system may amount to $\zeta_e = 5$ % or more by the contribution of the radiation damping.

7. EXAMPLES OF MEMBER-BY-MEMBER ANALYSIS OF PLANAR FRAMES

Along the lines so far reviewed, the member-by-member formulation of a R/C frame proceeds toward an elaborate microscopic description of its performance with full attention to local effects. The choice of the levels of complication to represent structural elements is a crucial one in terms of the practice of lengthy computation as well as the ease of programming. For both research and practical purposes, various computer programs have been developed to implement the analytical procedures chosen. A few examples of such analyses are shown in this chapter to demonstrate the advantages of member-by-member modeling.

A three-story R/C building is taken up. The purely moment-resisting frame is one of the design examples that are included in the appendices of the AIJ (Architectural Institute of Japan) Standards for Structural Calculations of Reinforced Concrete Structures with Commentary - 1971. This may be regarded as a typical R/C structure in Japan, designed in accordance with the Japanese Building Codes and the AIJ Standards; a working stress design of sections was performed by specifying a seismic coefficient of 0.2 at each story and by applying the stress analysis on the basis of the elasticity of materials [normal concrete with design strength ${}_c\sigma_B = 180\,\mathrm{kg/cm^2}$, and deformed reinforcing bars SD 30 (yield point of ${}_s\sigma_y = 3.0\,\mathrm{t/cm^2}$)]. The design is close to the one based on ultimate strength analysis because of the high allowable stress response process, respectively. These are intended to display in what process the damage has developed over the structure and how serious damage the respective elements have finally suffered from. All the critical sections of girders and columns are seen to have succumbed to cracking, some of the beam-to-column connections remaining elastic. The damage of the most severely impaired element (column-top section of the left-inner column at the 3rd story) is extremely critical. Its maximum ductility factor is about 50 which is far beyond the ultimate deformation losing the own restoring capacity and corresponds to an absolute pseudo-antisymmetric rotation of 0.2 radian.

From a practical point of view, one of the major limitations in member-by-member analyses is the tremendous amount of computation required. However, the modelings on such low levels of complication as implemented in the above example study can permit to render the analysis practicable with relatively small effort. Also it is reasonably felt that much important information provided by this type of detailed analysis deserves well the costly calculation.

8. SIMPLE MODELINGS OF OVERALL BEHAVIOR

Despite the marked advantages, it must be admitted that the member-by-member approach can frequently become involved and costly. This imposes the invention of less complicated structural models without impairing the essential features of dynamic response. In practical terms, the latter means a necessity to study the possibilities of evaluating, with a similar reliability, the seismic deflections of structure, which serve as the fundamental criterion in the conventional context of earthquake-resistant design. Also, the present modeling of a gross nature may be used to cover a certain class of buildings without going into specific minor details and by including relevant common properties of a particular interest. This is designed for examining their general trends when subjected to intense earthquake motions.

From early days of nonlinear structural dynamics, shear beam representation of multi-story buildings has been expected to meet the above purposes [e.g., 249 to 251, 64, 252, 253]. Indeed the story-by-story modeling is almost exclusively used when inelastic response analysis is required for practical design of building frames. According to this concept, restoring force properties of framing elements can be condensed into the characteristics of stories in a certain way, shear force and drift corresponding one to one at each story. Many types of the shear-drift relations such as elastoplastic, bilinear and degrading trilinear expressions have appeared in the literature. The simple formulation of structural performance leads to remarkable ease of computational effort.

Given the cracking and yielding parameters of framing members, both practical and sophisticated methods are available for evaluating the skeleton curves in the shear-drift relations of R/C structures. A comparative examination of this problem can be found in the reference [203]. Their representation form reduced to a bare minimum is trilinear, two prominent features of crack and yield being accounted on the story levels. Besides the elastic story stiffness, two shear forces associated with the cracking and yielding of the story, and a reduction factor of secant stiffness defined at yield point, become relevant to the formulation. In evaluating these parameters, caution must be used to the fact that the story failure cannot be uniquely determined unless the story is ideally of a weak column - strong girder type. Coupling of failure among stories features the other instance of a completely or partially weak girder type, which is often the case of practical R/C frames. In the latter situation, characterization of story shear forces premises a particular choice of the distribution mode of applied lateral forces.

Then a certain hysteresis rule for reversal and cyclic loading is associated with the skeleton curves. Hopefully this reflects the hereditary behavior of framing members. For instance, the degrading trilinear formulation may be applied which is a simpler version of the hysteresis rule developed for ductile R/C components.

Several studies have been reported to examine the reliability of the story-by-story modeling when used in the evaluation of strong-motion response [202, 203, 228; 200, 254]. The basis of direct comparisons was the time-histories of interstory drifts obtained from the corresponding member-by-member analyses. The fact that some assumptions of the shear beam idealization can be arbitrary and questionable for weak girder frames has motivated these studies.

Results of an examination along this line [228] are summarized in Fig. 9. The six examples include various patterns of structural failure as schematically shown in its left part. All are lowrise (3-storied) R/C structures. The examples A and B are flexible moment-resisting frames, while one of the two frames of example C and the example D have stiffened components of wall girder. The example E is a frame with shear walls, and removal of its shear walls provides the example F. Discrepancies in their interstory drift response, according to the two different means of member-by-member and story-by-story modelings, are indicated in the right half of this figure. The equivalent modeling referred to there will be explained later.

Denoting the i-th story drift by δ_i, the shear beam modeling applied to the examples A and B is shown to yield a marked trend of $\delta_1 > \delta_2 > \delta_3$. This is entirely inconsistent with the findings from member-by-member analysis: $\delta_1 \simeq \delta_2 > \delta_3$ for the example A and $\delta_1 \simeq \delta_2 \simeq \delta_3$ for the example B. The latter patterns of story drift response can be interpreted to reflect their modes of failure; the 1st and 2nd stories in the example A and all the stories in the example B are coupled each other, thus forming 2-DOF and 1-DOF mechanisms of yield-hinge kinematics, respectively. On the contrary, the results of the two analyses appear to coincide well in the examples C and D. Even though their modes of failure are of a hybrid type, performance of the weak column portion dominates over the weak girder portion in both stiffness and strength. The findings in the example E showing an ideal 1-DOF failure mechanism and in the example F having a dominant weak girder portion apply correspondingly to the comments given to the example B.

As demonstrated in the limited illustrations, serious differences can generally arise between the response predicted by the member-by-member model and the story-by-story model. The two can be in reasonable agreement only when the structure is of a weak column type. It has been found that such discrepancies are not attributable to an improper determination of shear-beam parameters but to the common nature inherent in the series combination of yielding elements. In a shear beam system, yielded drift can separately develop at a relatively weak element accompanying a significant action to suppress the progress of yield at its upper elements. The trend of failure concentration at a single element becomes particularly notable when the base element is

9. MODELS OF BIAXIAL BENDING AND TORSION

It goes without saying that realistic formulation of the out-of-plane performance of structure is very important for understanding the actual process of dynamic failure. This is the case because the torsional oscillation may arise significantly even under the action of unidirectional excitation. Moreover the horizontal orbit of earthquake ground motions is, in general, of a complicated and random pattern, showing a very poor trend of shaking confined in a single direction. Under the circumstances, a difficult problem is posed to model the inelastic and hysteretic interaction of restoring forces that occurs among the three components of two-dimensional drift and twist on the horizontal plane. It appears that the state of the art in this respect requires a great deal of improvement, since current methods of space frame analysis are mostly limited to the evaluation of elastic response.

On the levels of member-by-member modeling, columns and shear walls in a loaded space frame are subjected to the concurrent action of two pairs of bending moments (and associated shear forces), axial force and twisting moment. Among these six components, the torsional resistance of columns can be of a lesser contribution for an ordinary configuration of floor geometry. Usually the most significant are the two components which correspond to the two-dimensional antisymmetric distribution of bending moment along the axis of member, under constant axial force. Their resultant at a one or several story level is responsible for most of the floor-to-floor resistance both to the two-dimensional drift and to the twist. The following discussions deal with the simplest and most important case. This will serve as the reference state for more general states of stress. Some time-dependent deviations from the state occur in the actual case, and other appropriate techniques such as the nonlinear beam model in the case of planar frames become necessary for space frame analysis. This extension of the current problem is not included here.

The primary objective in this chapter is to summarize the mathematical formulations which permit to account for the significant features in the two-dimensional interaction of restoring forces of columns acted upon by biaxial flexure and shear. Studies on this particular subject began with the experimental work to examine the strength of R/C columns of rectangular cross section [e.g., 260 to 263]. In the literature, special attentions were given to the interaction relation for their yield or ultimate strength in terms of the two components of lateral load along the principal axes of section. Particularly, vulnerability of corner concrete block to crushing and spalling when loaded in the diagonal direction attracted considerable concern. The interaction relation was recognized to be affected by the magnitude of axial compression and so on.

The first formulation of two-dimensional restoring force systems can be found in a work of Nigam in the late 60's [264 to 266]. He developed a two-dimensional version of the elastoplastic or bilinear hysteresis model by means of the plastic potential theory in the solid mechanics. Similar approaches were used in the two studies by Wen and Farhoomand [267], and by Toridis and Khozeimeh [268]. Their elastoplastic or bilinear frame analyses took account of the interaction among the two bending moments and twisting torque including, in the former study, the other factor of axial force. Okada et al [269 to 271] disregarded the flow rule for plastic deformations and considered only the interaction of yield capacities in their analysis of inelastic torsional response. A study by Selna et al [272, 273] also lies along the identical line of mathematical included additionally were the corresponding one-dimensional evaluations for the major and minor-axis motions. It was found that the current effects differ between the two different criteria of damage and collapse and can be more complex than simply embodied by the sprawling parameter of energy. The study highlighted an insufficient characterization of the contribution of the minor-axis component to structural collapse by using the two-dimensional total energy.

Then the discussion supplements the remaining aspect of torsional oscillation. Because of its crucial role in modeling the structural dynamics, this subject has a long history of investigation. A series of studies by Shiga and his coworkers [292 to 295] is one of the early contributions. The previous reference of Okada et al [269 to 271] has provided an example analysis of the earthquake damage caused by a significant torsion. However, it is felt that the torsional effects in identifying the damaging process of R/C buildings have not been fully examined. This appears to be the case in both the two directions of microscopic and macroscopic analyses. A study by Kan and Chopra [296 to 299] is among the recent investigations of elastic torsional response.

10. CONCLUDING REMARKS

With emphasis upon the formulations of a basically empirical nature, the present report has surveyed the literature on the analyses of the failure of R/C building structures under the action of intense earthquake motions. Important aspects were first reviewed for the microscopic or member-by-member modelings, while the interest was then taken in macroscopic or simple modelings of overall behavior. Their advantages and limitations as well as the directions of future improvement were noted by including simple illustrations. Particularly, the survey emphasized unsatisfactory state of the art for a reliable description of the out-of-plane failure process of structure. In this concluding discussion, some additional comments are given as necessary for completing the brief study.

Since the topics of examination were chosen from the earthquake-resistant analyses, important subjects in the dynamic response of R/C systems have not been included unless they fall within the particular area. Vibration of girders and slabs due to vertical load is an example of this deletion, which often becomes practically important as related to the mitigation of their intolerable motion. On the other hand, the dynamics of building-foundation-soil interaction system was not taken up despite its essential significance in modeling the earthquake response of structure. This is because the problem was judged to belong to the earthquake geotechnical engineering or to more general area of structural dynamics rather than told within the context of the dynamic performance of R/C buildings. Nor studied was the influence of flexible floor. The rigid-floor assumption may fail in some practical instances, the in-plane deflection of floor plate providing additional degrees of freedom along the horizontal axis and inducing associated modes of oscillation. Moreover ground shaking may differ at separate positions of foundation, which leads to a problem of multi-input excitations. In view of these points, caution should be used to limitations imposed on this report.

In addition to the need of enhancing the capability of analytical methods, experimental evidence must be sought with greater efforts in forthcoming researches. This is related to the recognition that assumptions which are often fictitious or arbitrary and cannot be straightforwardly verified by simple experiments tend to be introduced at the current

About the first volume in the new IABSE series Structural Engineering Documents 1:

CONCRETE BOX-GIRDER BRIDGES

by Jörg SCHLAICH, Stuttgart, FRG
Hartmut SCHEEF, Stuttgart, FRG

The box girder is today the most widely used superstructure in concrete bridge construction. That fact justifies the suggestion made by the Commission III of the IABSE that a comprehensive paper be written concerning this particular bridge type.
The authors' aim is less that of encouraging the one-sided propagation of box-girder bridges but rather much more that of contributing to the improvement of the quality of such bridges. They hope to contribute to this by extensively relieving the engineer of the study of today's hardly surveyable mass of literature on the subject so that he can better devote that time to the actual design of the bridge.
The paper directs itself especially to the design engineer and therefore follows the sequence of a practical bridge design process by dividing itself into three main parts, namely, "Design", "Structural Analysis", and "Dimensioning and Structural Detailing".

Part I presents the most important factors influencing the architectural and structural design of box-girder bridges.

Part II follows with the structural analysis in longitudinal and transverse direction, and mainly deals with the interaction between both namely the folded plate action.

Part III mainly treats the structural detailing which includes the analysis and dimensioning of those regions of the bridge whose stresses cannot adequately be described with the Technical Bending Theory. Appropriate prestressing tendon profiles and reinforcement arrangements are suggested, and some bridge finishes are proposed.

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Dynamic Response of Reinforced Concrete Buildings

During the last two decades, remarkable progress has been made toward the better understanding of the performance of reinforced concrete buildings when subjected to destructive earthquake motions. This volume examines the literature with regard to analytical methods for modeling their dynamic failure, and is intended to provide a summary report on the current state of the art.

The survey places major emphasis on formulations of a basically empirical nature. Its necessity arises mostly from the complexities inherent in the inelastic and hysteretic response of reinforced concrete. Important aspects are first reviewed for the microscopic or member-by-member modelings, while the interest is then taken in the macroscopic or simple modelings of overall behavior. Their advantages and limitations as well as the directions of future improvement are noted by including simple illustrations. In particular, the examination emphasizes unsatisfactory state of the art for a reliable description of the out-of-plane failure process of structure.

An extensive presentation of the bibliography features this publication, dealing with the literature up until the middle of 1980 and covering no less than 300 papers. The work will contribute to bringing structural engineers up-to-date with the methods of strong-motion response analysis for reinforced concrete frames.