DEVELOPMENT OF SEISMIC DESIGN CRITERIA FOR CONNECTIONS IN JOINTED PRECAST CONCRETE STRUCTURES

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SUMMARY

This paper discusses issues in the development of improved procedures for seismic design of jointed precast concrete structures. The distinction between monolithic and jointed construction is defined. The need of specific code provisions for jointed precast construction is presented, and it is noted that further empirical knowledge of the seismic behavior of precast concrete structures is required before improved codes can be formulated. A current research program aimed at providing rational performance criteria, which specify conditions of loading and deformation for laboratory tests of precast concrete connections, is described. Preliminary results for a large panel crosswall building are presented.

INTRODUCTION

Precast concrete structures are of two types, "jointed" and "monolithic." In monolithic construction, precast elements are joined by well-reinforced connections possessing stiffness, strength, and ductility comparable to cast-in-place concrete. Jointed construction describes all means of connecting precast components which result in planes of significantly reduced stiffness, strength, and ductility at the interface between adjacent precast members.

In effect, current provisions of the Uniform Building Code (UBC) [1] require monolithic construction in seismic Zones 3 and 4, where major earthquake damage is anticipated. Detailing provisions of ACI 318-77 [2] can be applied in the design of monolithic precast structures, and performance comparable to that of cast-in-place concrete can be achieved.

However, over the largest portion of the United States the expected intensity of earthquake damage is moderate, at most. In seismic Zones 1 and 2, the UBC accepts jointed precast structures, but no distinction is made between jointed and monolithic construction with regard to loading criteria or reinforcement detailing requirements.

Thus, the UBC falls short of the ideal in its treatment of precast concrete construction. The lack of design guidelines for jointed precast structures could expose the public to hazards of inadequate construction. Competent structures appear needlessly expensive to an uninformed owner when inferior designs, costing less to construct, meet with no objections from the code.

On the other hand, current code provisions may waste opportunities for adequate performance at costs lower than those of cast-in-place concrete. When a requirement of monolithic behavior is imposed for precast assemblages,

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the fabrication efficiencies of jointed construction can be lost. Moreover, for low-rise panelized structures, the economics of jointed construction could favor design to force levels considerably higher than those currently specified for cast-in-place concrete, if connector ductility demands were proportionately reduced. Such possibilities are not addressed in the UBC.

Before improved code provisions can be achieved, however, significant advances in knowledge of the seismic behavior of jointed precast construction must be gained. One research program aimed at advancing the state of the art of connection design for precast concrete is being pursued by ABAM Engineers Inc. The study is funded in part by the National Science Foundation (NSF), under Grant No. CEE-8121733, "Design of Connections for Precast Prestressed Concrete Buildings for the Effects of Earthquake" [3].

This is the second phase of a three-part program. Phase 1, funded by the NSF and performed by others [4], defined the present state of the art. In Phase 2, a rational methodology for the derivation of connector performance criteria is being developed, guidelines for connection detailing are being formulated, and recommendations for physical testing of selected connection details are being prepared. Physical testing is to be conducted in Phase 3.

One aspect of the current study, developing a methodology for derivation of rational seismic performance criteria, is described below.

DEVELOPMENT OF RATIONAL PERFORMANCE CRITERIA

Due to the many uncertainties of material properties, stress distributions, and failure modes, practical design procedures for reinforced concrete evolve through physical testing. The most useful tests are those demonstrating the specimen's suitability for a particular type of structure under given service or ultimate conditions, rather than recording an exploration of the specimen's behavior under an arbitrary loading regime.

Rational Performance Criteria Prescribe Test Conditions

Thus, a methodology for estimating connector performance requirements is needed for planning the physical tests to be performed in Phase 3. Practical estimates of forces, inelastic deformation magnitudes, and number of cyclic load reversals characterizing anticipated service conditions must be made. In the present research, such requirements derived through analysis and the application of engineering judgement are referred to as "rational performance criteria."

The derivation of rational performance criteria involves specifying global seismic forces, analyzing elastic member force distributions, assessing global ductility demand, and determining the kinematics of plastic deformation. Considerations in each of these areas are presented below, with reference to a 17-story precast bearing-wall building, one of the example structures under investigation in the present research.

Specification of Global Seismic Forces

Seismic forces and deformations result from a structure's dynamic response to wildly oscillating ground displacements. The magnitude of response to a given earthquake is a complicated function of mass, stiffness, damping, and strength. Because it is impossible to predict with precision the characteristics of major earthquakes which might occur at a given location, the prediction of a structure's seismic forces and deformations is similarly uncertain.

For these reasons, modern building codes follow a practical approach to seismic design. The codes acknowledge the inherent ductility and energy absorption capacity in well-detailed structures. Depending on the framing system and construction materials, design forces significantly lower than the anticipated elastic strength demand may be specified.

Thus, the structure is expected to yield under the most severe ground shaking anticipated at the site. Code requirements for ductile construction details assure that structures can sustain significant damage without collapse, while the design forces are large enough to produce elastic behavior under minor ground shaking.

In the Uniform Building Code, design forces are specified for a structure of "average" ductility, then scaled up or down using the code's K-factor to give appropriate strength to structures with lesser or greater capacity for inelastic action. The designer is not apprised of the implied global ductility demand corresponding to a given K-value, nor of the forces that would develop if the structure remained elastic.

For these reasons, UBC seismic force provisions are not being used in the present study. Rather, a new document, "Tentative Provisions for the Development of Seismic Regulations for Buildings" [5], has been adopted for specification of global seismic forces. This volume, referred to as "ATC-3," was prepared by the Applied Technology Council and is under evaluation by design professionals and code interests in the United States.

In the ATC-3 procedure, the magnitude of reduction between the forces due to fully elastic response and the forces to be used in design appears explicitly, in the form of a force-reduction factor, R. Structures with little redundancy and ductility qualify for relatively small force reductions. For example, the recommended R for a partially reinforced masonry shear wall is 1.5. Structures with significant redundancy and ductility qualify for greater force reductions; the recommended R for a well-detailed steel frame is 8. ATC-3 does not distinguish between monolithic and jointed concrete in its specification of R-values, so these will need to be considered in the present work.

ATC-3 incorporates site-dependent response spectra for specification of elastic strength demand. Empirical period formulas of ATC-3 and the UBC may not be reliable for jointed precast construction so, in the present study, building periods are being estimated by analyses which include effects of foundation and connector flexibility.

Analysis of Elastic Member Force Distributions

Analysis assumptions commonly used for monolithic structures may not be suitable for jointed precast concrete. The ACI strength design method, in which member strengths are assigned according to relative elastic stiffnesses of the undamaged structure, relies on inelastic load redistribution to adjust for differences between actual and computed load paths. Jointed precast structures rely on discrete connectors of limited ductility and have a relatively low degree of redundancy. Consequently, their load redistributing capacity may be insufficient to accommodate inaccuracies of the simplified analyses usually performed for monolithic structures.

Among other reasons, load distributions of the strength design method are imprecise because they neglect changes in relative stiffness that occur as joints open and close during response to ground shaking, and because analysts frequently assume that foundations and floor diaphragms are rigid. These issues are being considered in the present research.

One case under investigation is a 17-story large panel precast apartment building, shown in Figure 1. This structure was designed by the PCI Committee on Bearing Wall Buildings. The building and the analyses discussed here are described in detail in Reference [7]. Elastic force distributions have been analyzed by the STRUDL [6] program, using three-dimensional computer models.

Diaphragm and foundation flexibility have been found to exert important influences on the predicted distribution of loads among lateral resisting elements. In-plane, cross-wall moments for the apartment building subject to transverse earthquake loads are shown in Figure 2. In this figure, results of a hand calculation assuming rigid diaphragms and foundations are compared with STRUDL results where diaphragm flexibility is modeled.

As can be seen from the solid and dashed lines in Figure 2, results of the hand calculation and STRUDL analyses are in close agreement when foundation flexibilities are ignored. However, as shown by the dotted lines, predicted moments in some walls increase by up to 50 percent when foundation flexibility is included. The results show that, for shear-wall structures of the proportions studied, wall moments distribute in proportion to relative foundation stiffness.

Note that the results presented are based on relative stiffnesses before cracking, and are suitable for assessing strength requirements up to the seismic damage threshold. However, this analysis may be inadequate for characterizing force distributions once nonlinear action commences. In particular, due to stiffness reductions caused by wall cracking, foundations may appear "rigid" in the damaged structure, and shears may distribute among the walls in proportion to their relative inelastic stiffnesses.

Thus, an analysis considering post-yield structural behavior must also be performed. For some structures, such studies could reveal substantial increases in diaphragm loads due to changing distributions of shear among lateral force resisting elements as damage progresses. A methodology for performing such analyses is being developed in the present research.

It has been shown that the modeling of foundation flexibility can have a substantial influence on predicted wall and diaphragm forces. Thus, to reduce the need for load redistribution, and thereby mitigate connector ductility demands, it is recommended that diaphragm and foundation flexibilities be included in analytical models for predicting lateral force distributions in jointed precast bearing-wall structures.

Specification of Global Ductility Demand

Because it employs simplified procedures based on the theory of elastic/perfectly plastic (EPP), single-degree-of-freedom (SDOF) systems, the ATC-3 approach adopted for specifying global seismic forces also enables the estimation of total inelastic displacement under earthquake loading. Elastic displacements under the design loads are scaled up by a drift coefficient, Cd, to estimate maximum displacements of the damaged structure.

Due to the particular choices of R-values and Cd-values in ATC-3, predicted inelastic displacements in all cases are practically the same as would occur if the structures responded elastically to the design earthquake. This implies that the global ductility factor depends only on the ratio of elastic strength demand to the elastic strength provided.

Studies of EPP SDOEF systems [8] have shown that the relationship between elastic and inelastic displacement magnitudes is not constant as implied by the ATC methodology, but depends on vibration period. The ATC assumption that peak elastic and inelastic displacements are equal seems to be appropriate for systems with period longer than about 3 seconds. However, for systems with period of 0.5 second or less, strength reductions below the elastic demand can result in unacceptably large ductility requirements.

For systems in the intermediate period range, between 0.5 second and 3 seconds, elastic and inelastic deformation magnitudes are related through the energy imparted to the structure by seismic ground motions. Peak inelastic deformation magnitude exceeds the peak elastic value by an amount depending on the degree to which design strength is reduced below the elastic strength demand. For example, a strength reduction factor of $R = 4$ produces inelastic displacements twice as large as the elastic values.

Mahin and Bertero [9], in an evaluation of the methods of Newmark and Hall and ATC-3, have concluded that these techniques do not reliably limit ductility demands to specified values, even for ideal EPP SDOF systems. Nonetheless, it was concluded that they can provide valuable guidelines for design if their limitations are recognized, particularly that there is large scatter in the results so ductility far in excess of predicted demands must be provided.

While the simplified approach of ATC-3 may prove adequate for design, the more refined method, in which the relationship between strength and required ductility is a function of period, will be used for computing global ductility demand in the derivation of rational performance criteria.

Kinematics of Plastic Deformation

To complete the specification of rational seismic performance criteria, inelastic deformation magnitudes of the connectors must be determined. This aspect of the methodology is still under development; however, the intended procedure can be described.

To satisfy the requirement that the structure remain undamaged under ground shaking of a magnitude expected to occur several times during the structure's life, strength will be assigned to members in the lateral force resisting system based on results of the initial elastic analysis under the 1/R-factored ATC-3 seismic loads.

In consideration of differing wall dead loads and other factors affecting the onset of nonlinear response, locations where the earliest plastic hinges are expected to form will be identified. Relative stiffnesses of lateral resisting members will be adjusted to reflect the occurrence of damage during strong ground shaking. The "second-stage" elastic model thus obtained will be subjected to imposed deformations corresponding to the predicted maximum inelastic global displacement. Estimated moment-rotation properties of the plastic hinge regions will be used to compute hinge rotation magnitudes.

Recognizing that strains tend to concentrate in the joints of nonmonolithic precast structures, kinematic models representing the spatial geometry of given joints will be used to translate plastic joint rotations into plastic deformations of specific connectors.

CONCLUSION

To a great extent, the economic and functional success of a precast concrete structure depends on the configuration and properties of its interelement connections. The design of connections which are easily fabricated, speedily erected, stable, strong, and ductile is a demanding task.

Because present U.S. building codes lack specific provisions for jointed precast construction, designers must work without desired technical guidelines and the public safety is in the hands of the lowest bidder. To overcome present code deficiencies, further knowledge of the seismic behavior and failure modes of jointed precast construction is needed.

Phase 2 of a three-part program intended to advance the state of the art of connection design for jointed precast structures has been described. A principal objective of this study is the development of a rational methodology for the derivation of connector performance criteria. Results of this effort will provide quantitative guidelines for the planning of physical tests on connectors, to be conducted in Phase 3. In time it is hoped that this work, combined with parallel efforts by others, will culminate with the adoption of improved code provisions for precast concrete construction.

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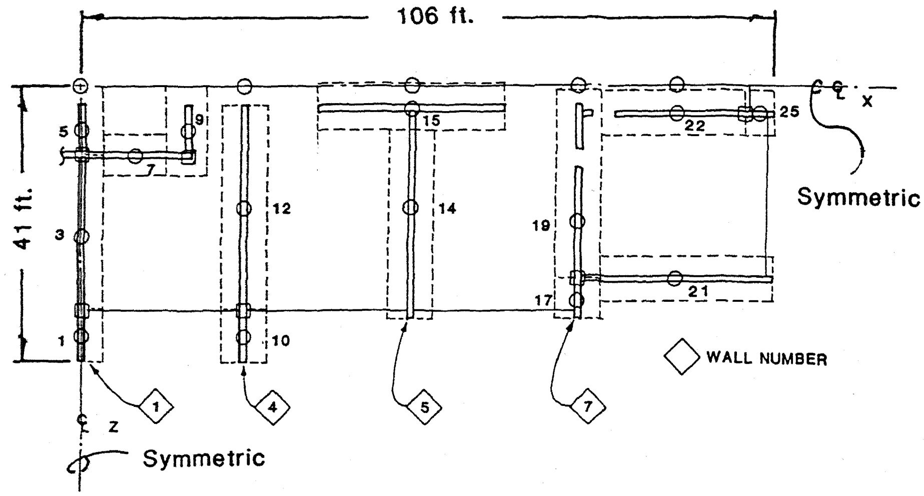

Figure 1 One–Quarter Plan Showing Arrangement of Walls and Foundations

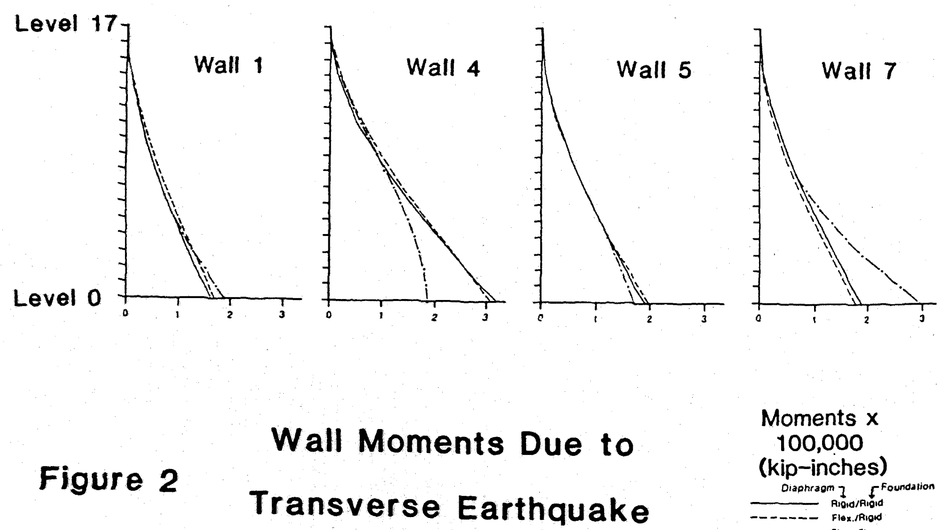

Figure 2 Wall Moments Due to Transverse Earthquake