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PB93-214724

THE JAPANESE FIVE-STORY FULL SCALE REINFORCED MASONRY BUILDING TEST

by

FRIEDER SEIBLE

JANUARY 1988

DEPARTMENT OF APPLIED MECHANICS & ENGINEERING SCIENCES
UNIVERSITY OF CALIFORNIA, SAN DIEGO, CA

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PART I—PROJECT IDENTIFICATION INFORMATION

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THE JAPANESE 5-STORY FULL SCALE REINFORCED MASONRY TEST

PART II—SUMMARY OF COMPLETED PROJECT *(FOR PUBLIC USE)*

From August 16, 1987 to December 19, 1987 Dr. Frieder Seible participated in the test program of a 5-story full scale reinforced concrete masonry building at the Building Research Institute of the Ministry of Construction of the Japanese Government of Tsukuba City, Japan. Dr. Seible was on sabbatical leave from UCSD for the indicated time period, and the research stay in Japan was financially supported by the Science and Technology Agency of the Japanese Government and the National Science Foundation under the UJNR Cooperative Research Program on Wind and Seismic Effects. Dr. Seible's participation in the Japanese Masonry Research Program resulted in four publications presented in the Appendix to this report and a commitment to complete two more joint research papers with Japanese researchers within the next six months.

PART III—TECHNICAL INFORMATION *(FOR PROGRAM MANAGEMENT USES)*

1. ITEM *(Check appropriate blocks)*	NONE	ATTACHED	PREVIOUSLY FURNISHED	TO BE FURNISHED SEPARATELY TO PROGRAM: Check (√)	TO BE FURNISHED SEPARATELY TO PROGRAM: Approx. Date
a. Abstracts of Theses					
b. Publication Citations					
c. Data on Scientific Collaborators					
d. Information on Inventions					
e. Technical Description of Project and Results		X			
f. Other *(specify)*					

Principal Investigator/Project Director Name *(Typed)*	3. Principal Investigator/Project Director Signature	4. Date
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Report on 4-Months Research Visit to the Building Research Institute at Tsukuba City, Japan

by

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Jan. 1988

SUMMARY

From Aug.16,1987 to Dec.19,1987, Dr.F.Seible participated in the test program of a 5-story full scale reinforced concrete masonry building at the Building Research Institute of the Ministry of Construction of the Japanese Government in Tsukuba City, Japan. Dr. Seible was on sabbatical leave from UCSD for the indicated time period, and the research stay in Japan was financially supported by the Science and Technology Agency of the Japanese Government and the National Science Foundation under the UJNR Cooperative Research Program on Wind and Seismic Effects. Dr. Seible's participation in the Japanese Masonry Research Program resulted in 4 publications presented in the Appendix to this report and a commitment to complete 2 more joint research papers with Japanese Researchers within the next 6 months.

BACKGROUND

The third phase of the United States-Japan Cooperative Research Program on Seismic Effects has the objective of developing comprehensive design guidelines for masonry structures in seismic zones to advance the state of technology in masonry construction in both countries. Both countries have organized individual TCCMAR (Technical Coordinating Committee on Masonry Research) programs to provide the necessary research base and data to formulate these comprehensive design guidelines. A major component of this joint research effort is the exchange of research information between the two countries in annual joint meetings as well as the exchange of researchers for certain important phases of the joint research program.

The overall JTCCMAR Research Plan calls for the final validation and verification of the proposed new analysis and design models for masonry structures by means of a full scale laboratory test of a 5-story reinforced concrete masonry building under simulated seismic loads in each country. The Japanese 5-story full scale masonry research building was tested this fall at the Building Research Institute in Tsukuba City, while the U.S. 5-story full scale building test is scheduled for 1990 at the University of California, San Diego, in the Charles lee Powell Structural Systems Laboratory, the only facility in the Nation where currently such a full scale test can be conducted.

With Dr. Seible being a member of the U.S.-TCCMAR Team and the U.S. test to be conducted at UCSD, Dr. Seible was sent as the official U.S. observer to the Japanese full scale test. The objectives of the research stay in Tsukuba City, Japan, were to coordinate the U.S.-Japan JTCCMAR effort on the full scale testing phase, to provide input to the Japanese test program and to provide the U.S. research and engineering community with the latest Japanese research data from the concluding full scale reinforced masonry building test.

RESEARCH ACTIVITIES

Dr. Seible's research activities during his 4 months research stay in Japan can be grouped into three categories based on the above outlined objectives.

The first activity was a detailed and comprehensive summary of the U.S. modular TCCMAR program, in particular the experimental tasks, to provide the Japanese researchers with the necessary information on the U.S. masonry research effort, in order to coordinate future joint research programs. This comprehensive summary of the U.S. research activities is attached in Appendix I, and was formally presented at the Third U.S.-Japan Joint Technical Coordinating Committee Meeting on Masonry Research in Tomamu, Hokkaido, Japan, Oct,1987.

As one of the direct research contributions to the Japanese Japanese 5-story full scale masonry building test, see Fig.1, Dr.

Seible evaluated the proposed loading system for the 5-story test building by means of computer simulations. This detailed evaluation was also presented at the Third U.S.-Japan JTCCMAR Meeting in Hokkaido and is attached in Appendix II. This evaluation resulted in recommendations for the prestressing level in the floor slabs, information needed prior to the full scale test.

The main activities naturally related to the full scale test itself and the dissemination of relevant research data to the U.S. masonry community. As the appropriate forum for this information dissemination the Masonry Society Journal was chosen, and a series of four papers on the Japanese 5-story full scale masonry building test was proposed, see Appendix III. Of the four reports, the first two have been completed and were submitted for review for publication to the Masonry Society Journal. These two papers are enclosed in Appendix IV and V, respectively. Work on the final two reports is still in progress and should be completed within the next six months.

While the obtained and reported test data is certainly of high interest to the U.S. research community, the experience of the Japanese full scale test and the lessons learned from this test will be an invaluable experience for the upcoming U.S.-TCCMAR full scale masonry building test in San Diego. Since the U.S. test will be the first test of a full scale building under laboratory conditions in this country, it was essential to participate in the Japanese test which constitutes the third full scale building test world wide and in Japan.

RELATED ACTIVITIES

In addition to the participation in the full scale test and the associated data reduction and report writing, Dr. Seible also visited numerous large scale structural testing facilities in Japan. Mentioned should be among others the world's largest shaking table facility in Tadotsu, the structural laboratories associated with the Universities of Tokyo and Kyoto, research laboratories of the Japanese Construction Industry, such as Okumura Corporation, as well as the Public Works Research Institute of the Ministry of Construction. Visits to these facilities and discussions with the associated researchers were of particular importance to the continued completion of a world class structural research facility here at UCSD.

On the academic level visits to the Universities of Hokkaido (Sapporo), Kyoto, Tokyo and Tsukuba provided the necessary stimulation for future research endeavors as well as invaluable personal contacts with the Japanese Academia. A formal presentation to the faculty and researchers of the Institute of Industrial Sciences of Tokyo University, see Appendix VI, was made as part of one of these visits, in an effort to exchange the latest research developments. The associated discussions not only concentrated on the building research side, but also into the field of bridge research, where large experimental projects are going on in Japan, parallel to some of the research efforts here at UCSD.

CONCLUSIONS

The lessons learned from the Japanese 5-story full scale reinforced concrete masonry building test are invaluable to the U.S.-masonry research community in general and to the UCSD researchers in particular, since the parallel U.S.-test will be performed in San Diego. The research visit also resulted in new ideas and personal contacts for future joint research activities.

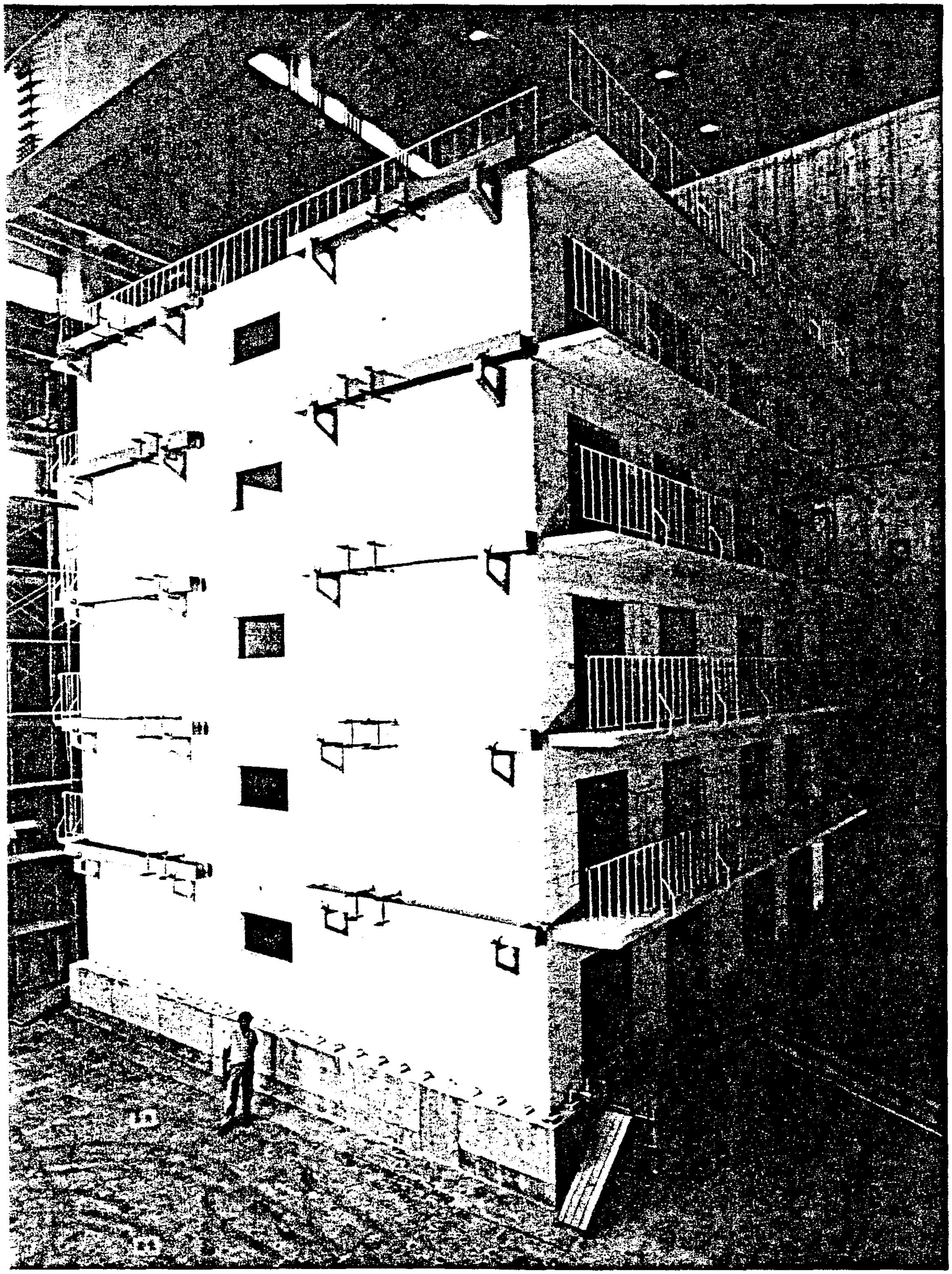


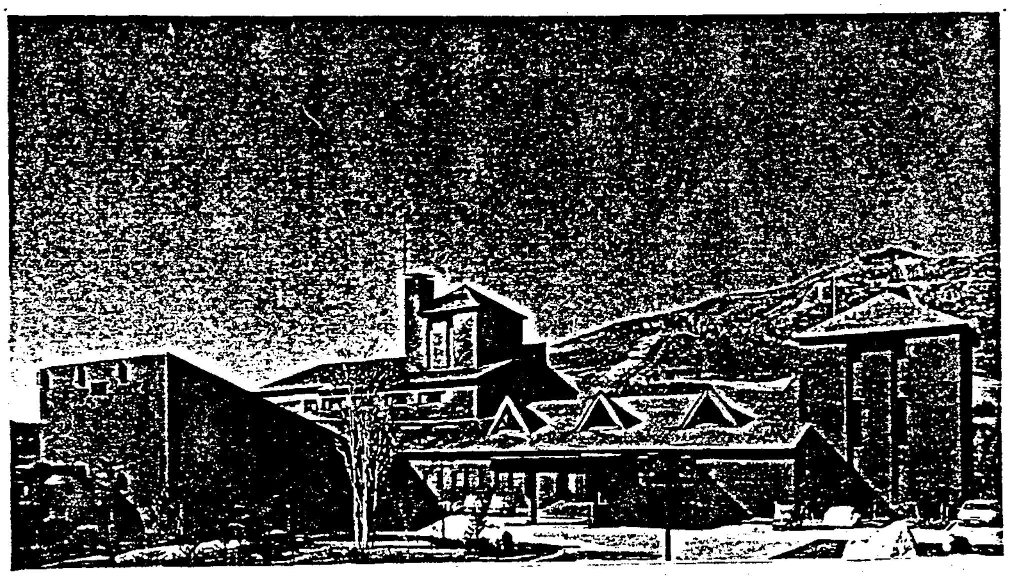
Fig.1 Japanese 5-Story Full Scale Building Test

APPENDIX I

U.S.-JAPAN COORDINATED PROGRAM FOR MASONRY BUILDING RESEARCH

THE THIRD MEETING OF THE JOINT TECHNICAL COORDINATING COMMITTEE ON MASONRY RESEARCH

Chaired By J.L.Noland and T.Okada



OCTOBER 15, 16 AND 17, 1987, TOMAMU, HOKKAIDO, JAPAN

U.S.-JAPAN PANEL ON WIND & SEISMIC EFFECTS (UJNR)

Third-Joint Technical Coordinating Committee on Masonry Rresearch
- U.S.-Japan Coordinated Earthquake Research Program -
Tomamu, Japan, October 15-17,1987

THE MODULAR U.S.-TCCMAR PROGRAM FOR MASONRY BUILDINGS IN SEISMIC ZONES - SUMMARY OF EXPERIMENTAL PROGRAM -

by

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ABSTRACT

The modular structure of the U.S. coordinated research program on masonry structures in seismic zones is outlined and the individual experimental research modules are summmarized. These individual research modules can be grouped into materials tests, component tests, sub-assemblage tests and finally the prototype test with the testing of a 5-story full scale research building representing a segment of a prototype masonry structure. The entire experimental program is interconnected by a common analytical modelling effort which draws its validation and verification from this large scale experimental data base. The complexity of predicting the behavior of masonry structures under critical seismic loads requires the experimental program to provide the corresponding complexity of data with the transition from materials, components, and sub-assemblages to the prototype test. The required changes in the test methodology for the individual experimental research modules are discussed in this paper.

INTRODUCTION

The experimental portion of the U.S. coordinated program on masonry research has to be seen within the overall TCCMAR (Technical Coordinating Committee on Masonry Research) effort of providing a broad and rational basis for the development of a comprehensive design philosophy for masonry structures in various seismic zones.

The broad data base required for this development of new design guidelines can only be established by detailed parameter studies at the materials, components, substructures and prototype structure levels. The quantity of parameter studies needed to cover all aspects of material proerties, construction types, geometry and dimensions, as well as a multitude of possible critical load combinations and load and cummulative damage histories can only be provided by efficient analytical models. These analytical models of various complexity for materials, components, substructure and prototype studies require experimental verification at each level prior to combining the individual modules to predict the overall structural behavior under seismic loads. This required syntnhesis for the analytical modeling is also reflected in the experimental program, where the same modular philosophy has been adopted, allowing the behavioral study of materials and components as well as their combined substructure and prototype performance under critical loads.

The main objectives of the TCCMAR-U.S. experimental program are thus to furnish a reliable data base which will support the development of rational design methods for masonry structures and to provide the necessary validation of computer models for seismic response analysis and design. In an effort to achieve these objectives, a phased modular program of separate , but coordinated experimental tasks has been adopted. The resulting sequence commences with basic material studies, progresses to fundamental structural elements, continues to assemblages of several such elements to substructures, and culminates with the synthesis of the above modules in a full scale laboratory test of five stories of a generic masonry building.

CONCEPT

The experimental U.S.-TCCMAR program is based on the concept that (1) large or full scale tests are needed to validate analytical models, (2) individual, self contained test modules have to be identified which maximize nation wide utilization of expertise and facilities, and (3) all the experimental modules are inter-connected through the common analytical effort to allow the complex synthesis process. Only close coordination and cooperation between the individual research tasks, the design profession, the manufacturing and construction industry, and the building officials can lead ultimately to a comprensive revision of design codes. In this process the experimental

TCCMAR program is needed to support and substantiate these comprehensive design recommendations for each individual module and, collectively for materials, components, sub-assemblages, and prototype levels as shown in Fig.1. The individual experimental modules have to be not only laterally but also upward compatible to allow the synthesis process depicted in Fig.1 with the development of a common experimental methodology which is based in concept and complexity on the analytical modelling and design requirements. It is therefore important that all experimental modules are linked together by the analytical network as schematically shown in Fig.1.

With the objective of the TCCMAR program being in the seismic design area of buildings, the analytical modelling and with it the associated experimental verification has to reflect not only strength and capacity information but more importantly a complex and probabilistic loading sequence and history which can not be addressed with only monotonic forcing functions. Thus, just like the modular development of the experimental program itself, the test methodology has to change and develop during the test program to reflect realistic conditions and behavior. It is therefore schematically shown in Fig.1 that the testing regime changes and develops from monotonic, cyclic, and cyclic Sequencially Phased Displacements (SPD) to Generated Sequential Displacement (GSD) histories which reflect the global structural loading and where possible the response history under seismic loads. (A discussion of the above terms will follow in the Metodology Section). True dynamic or real time loading will be limited to non-destructive type forced vibration testing to determine general dynamic response characteristics and to scale model and component shake table tests which will be used to verify some of the load rate dependent structural aspects.

The TCCMAR research effort concentrates on developing rational design and analysis models for the materials behavior, the component behavior and substructure behavior for relevant seismic loading environments. The research goal is to combine these individual modules and to predict the response of complete building systems. To accomplish this task, TCCMAR has adopted this modular system of concurrent experimental and analytical tasks. The purpose of the experimental phase is to assist in the development of new and to validate completed analytical models, as well as to provide some full scale data base for the formulation of design guidelines.

METHODOLOGY

The general experimental TCCMAR methodology directly results from the principal objective of the experimental program, namely the verification of analytical and design models by full scale tests. This requires on the analytical side models which can simulate structural behavior under critical seismic loads, a requirement which can only be verified experimentally if the

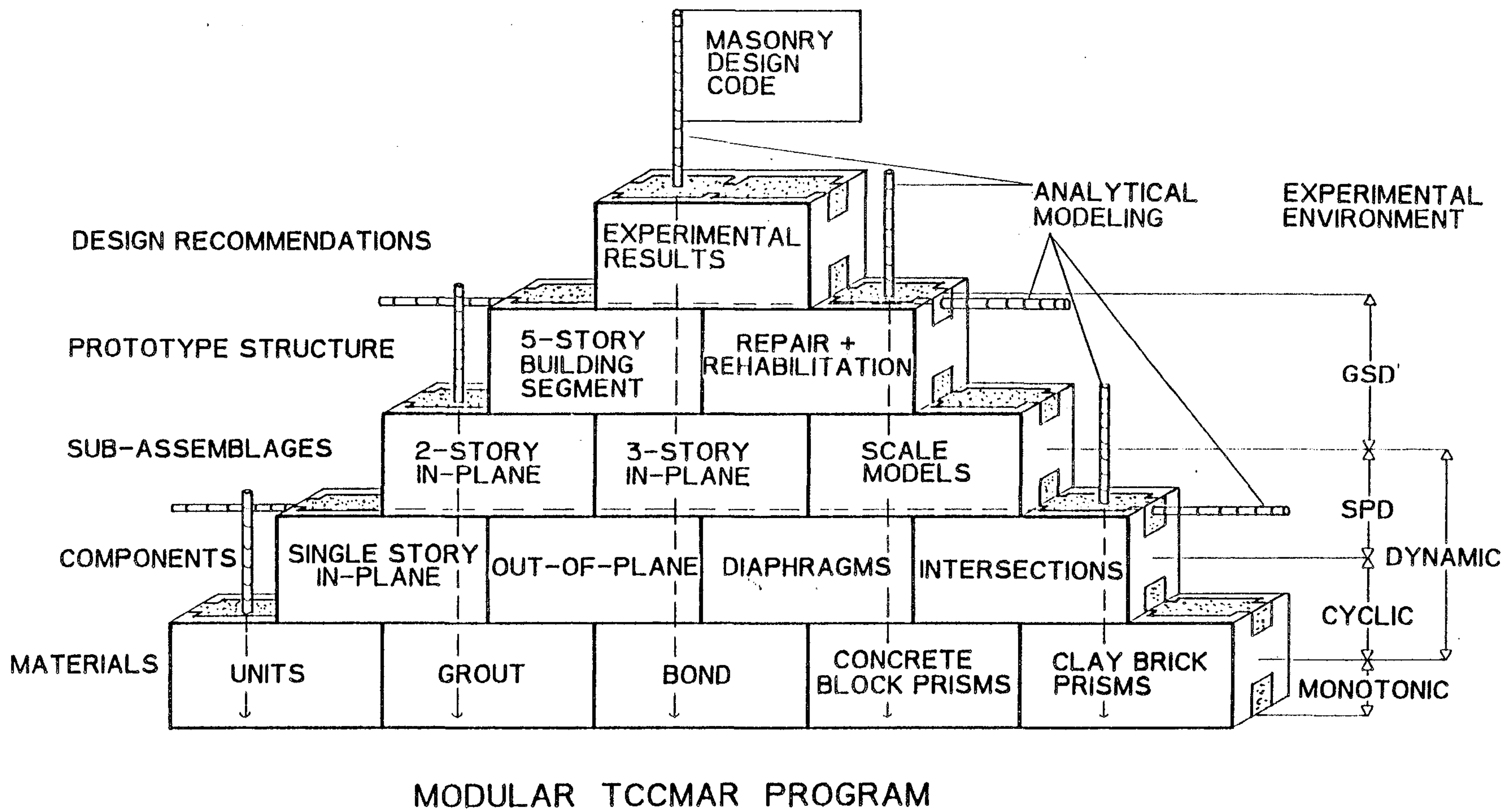

Fig. 1 U.S.-TCCMAR Program Overview

experiment can provide a realistic corresponding load environment. While during the development of the analytical models at the materials and component level simple load and deformation models are important for establishing the required element characteristics, it is equally important to extend the complexity of the forcing function to include phenomena such as low energy cycles after high peaks and and consecutive cycles of the same or similar magnitude to ensure stable conditions. Finally, questions of the dependency of the response history of a structural system with damage accumulation on the load or deformation history need to be addressed, particularly in highly redundant structural systems which offer a multitude of force redistribution capabilities. Thus, a methodolgy was adopted for the experimental TCCMAR program, which increases the complexity of the experimental procedure with the increasing complexity of the test module.

At the broadest level of experimental modules, the **materials** level, the main objective is the compilation of basic constitutive data of the individual materials as well as their combined action in manageable prism sizes. Since nominal design data on the materials will also be derived from these tests, the experimental procedure should be kept as simple as possible in order to allow duplication of tests at different locations. The primary experimental loading environment for this level is therefore monotonic displacement or load controlled. Where analytical or design models require a detailed trace of the unloading branch the complexity of the test setup and control increases while the basic loading philosophy still remains simple. Where behavioral aspects are directly controlled by load reversals such as bond, splice and anchorage performance, testing is extended to a cyclic loading environment.

The cyclic loading environment is then dominant for the **component** level of the modular system with test specimens representing structural elements which will experience load reversals, stiffness changes and damage accumulation in discrete behavior modes. In order to formulate analysis and design models the experimental procedure must allow the capturing of behavioral limit states as well as a trace of low level cyclic behavior after such a limit state has been exceeded. Thus, the forcing function for the component experiments should be a special time history which allows the investigtion of these behavioral aspects.

Such a time history was developed jointly by TCCMAR researchers in the form of a prescribed sequence of displacement cycle phases which are scaled based on the behavior of the element. The result, called Sequentially Phased Displacement (SPD) loading, was summarized by Porter [1] and is depicted in Fig.2. This scheme considers sets of increasing cyclic amplitude up to the onset of the first evitable significant damage limit state (e.g. flexure or shear cracking), which is termed the First Major Event (FME), followed by subsequent degrading and stabilization cycles for

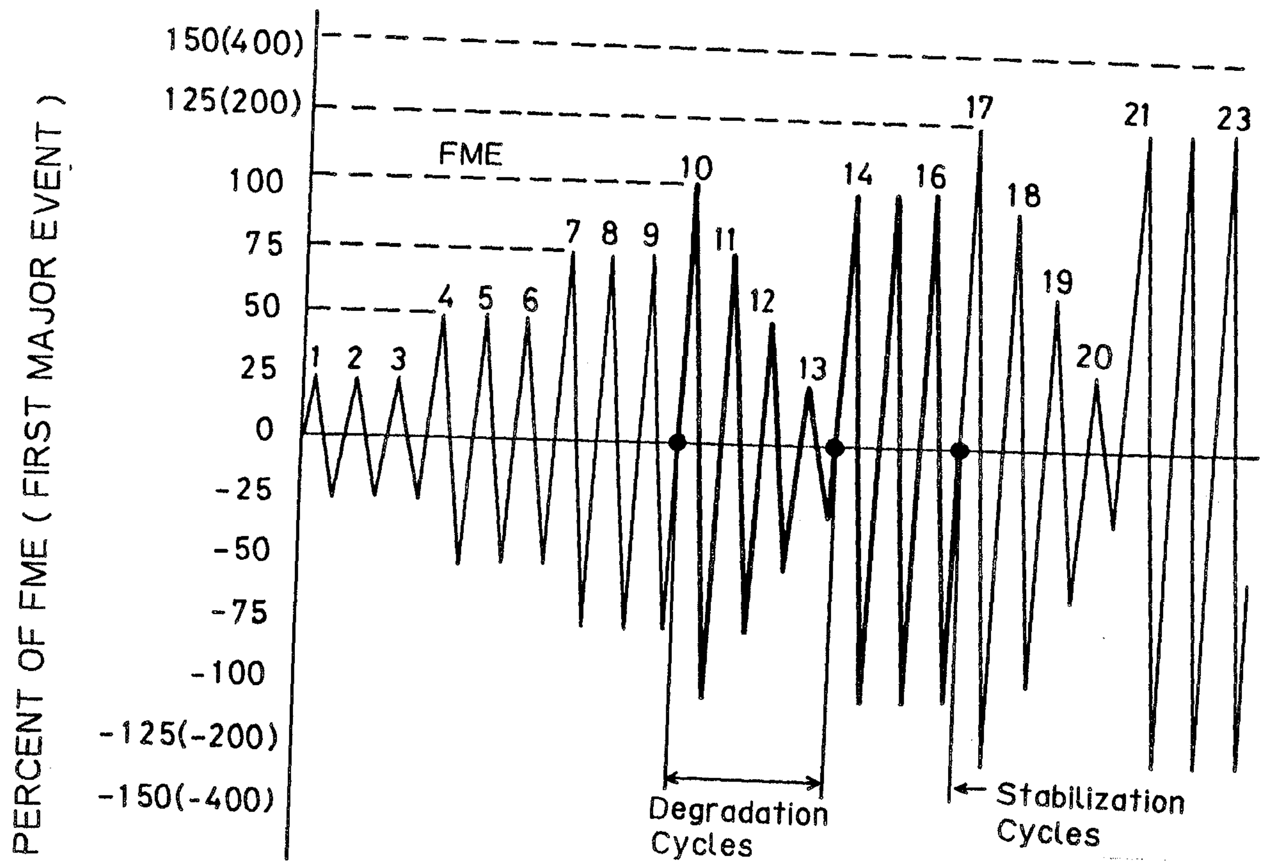

Fig. 2 Cyclic TCCMAR Displacement History

various levels of FME. This SPD procedure, which is conducted under displacement control, is an attempt to maximize relevant behavioral information needed for the analytical model development.

The SPD concept is applicable for those cases where the primary deformation of the component or substructure can be associated directly with a discrete mode of deformation, i.e. a Single Degree of Freedom (SDOF). For highly redundant Multi Degree of Freedom (MDOF) **sub-assemblages** or **prototype** structures, SPD can yield useful information only if the distribution of the significant deformation modes are known and can be 'slaved', generally in force control, to a single displacement DOF. Since these contributions of different deformation modes change with damage accumulation in the structure under real seismic loads, the SPD concept no longer approximates this behavior dependent load but rather an equivalent nominal lateral loading. This correspnds to most seismic design codes where certain lateral load distributions are assumed over the height of the structure and applied to a linear elastic structural model. Since the experimental model does not remain in the linear elastic range, the loading philosophy has to be extended to reflect the interaction between the load and the response side in the real earthquake case.

This extended experimental concept will be termed GSD (**G**enerated **S**equential **D**isplacement) method to reflect the fact that sequentially applied displacement histories which were generated based on the structural response are applied to the controlled DOF's. The form in which the prescibed displacements are generated can and will vary depending on the test structure, i.e. soft structural systems may have analytical updates at every load step, generally refered to as pseudo dynamic testing [2], or stiff structural systems may have analytical updates only after the occurance of major events based on an analytical model which can, even on a semi-empirical basis, reflect the structural stiffness changes. While it is not claimed that such a proceedure necessarily reflects the actual structural response to a certain eathquake (the next one will be different anyways), the sequential formation of damage zones and/or mechanisms in highly redundant systems,which depends on the structure-load interaction for inertia type loading, may be more realistically represented. In simplified terms the envisioned GSD procedure will use an analytical prediction for the prescribed displacement response of the test structure and load the test structure incrementally in this deformation mode using conventional techniques, e.g. displacement control in one DOF and slaved force control in the others. However, after a major event or obvious structural damage has occured, the analytical prediction of the governing global deformation mode will be updated using an analytical model which is calibrated by the current state of the test structure. This procedure can be viewed as a reboot or restart at discrete times in the forcing function time history. In the limit, if such a reboot (re-analysis of the structural response) occurs at every loading time interval, the proposed procedure could approach the well documented pseudo dynamic test method [2], if the test objective is the trace of the structural response to a particular ground motion time history. However, if the objective is the monitoring of sequential stiffness degradations and the calibration of nonlinear analytical models, some simplified or specially designed time histories can be established for the forcing function of the analytical engine which will optimize the experimental data gained from a full scale test, with possible adjustments to the load side during the test progress. The development of such a GSD procedure for masonry structures is a major component of the overall TCCMAR research effort.

EXPERIMENTAL PROGRAM

The scope of the U.S.-TCCMAR experimental effort is illustrated in Table 1, with each task corresponding to one of the individual experimental modules shown in Fig.1. Also indicated in Table 1 are the principal researchers and their respective affiliations. All tests are being conducted on six inch fully grouted full scale concrete block specimens except for correlation studies with clay brick units and scale model shaking table tests. A brief description of each experimental research module is furnished in the following sections.

The broad basis of the experimental U.S.-TCCMAR program is formed by **materials** investigations ranginging all the way from tests on individual units and core samples to composite behavior tests (unit-grout-rebar) and prism tests of various arrangements. The primary objective of the materials test modules is the establishment of constitutive data for analytical modeling and of nominal design parameters.

Basic uniaxial compressive behavior was investigated by Atkinson et al.[3], resultng in a comprehensive set of constitutive data as sampled in Fig.3. Characteristic behavior of masonry under axial, flexural and combined axial and flexural loading is under investigation by Hamid et al. at Drexel University [3,5] on concrete block prisms, utilizing a test setup as schematically shown in Fig.4. A parallel experimental module on clay brick prisms is conducted by Brown [6] at Clemson University. Component interaction at the materials level involving bond, splice and hook characteristics were evaluated by Tulin et al. [7,8] at the University of Colorado; Fig.5 presents an overview of some of the investigated experimental configurations.

The second level of test modules (see Fig.1) is comprised of **component** tests wherin the in-plane and out-of-plane behavior of elements such as walls, floor diaphragms and intersections is studied.

TABLE 1 - Experimental U.S.-TCCMAR Program

LEVEL	MODULE	P.I.	AFFILIATION
Materials	Units, Material Models	Atkinson/Noland	Atkinson, Noland & Associates
	Concrete Block Prisms	Hamid/Harris	Drexel University, Philadelphia
	Clay Block Prisms	Brown	Clemson University Clemson, SC
	Grout and Process	Tulin	University of Colorado, Boulder
	Bond and Splices	Tulin	University of Colorado, Boulder
Components	1-Story in-plane	Shing/Noland	University of Colorado, Boulder
	Walls (clay) out-of-plane	Mayes	Computech Eng. Serv. Berkeley, CA
	Walls (conc.) out-of-plane	Adham	Agbabian & Assoc. El Segundo, CA.
	Floor Diaphragms	Porter	Iowa State Univers. Ames
	Wall Inter-sections	Priestley	Univ. of California San Diego
Sub-Assemblages	2-Story in-plane	Klingner	University of Texas, Austin
	3-Story in-plane	Hegemier/Seible	Univ. of California San Diego
	Scale Models	Abrams	Univ. of Illinois Urbana
Prototype	5-Story Building	TCCMAR	Univ. of California San Diego

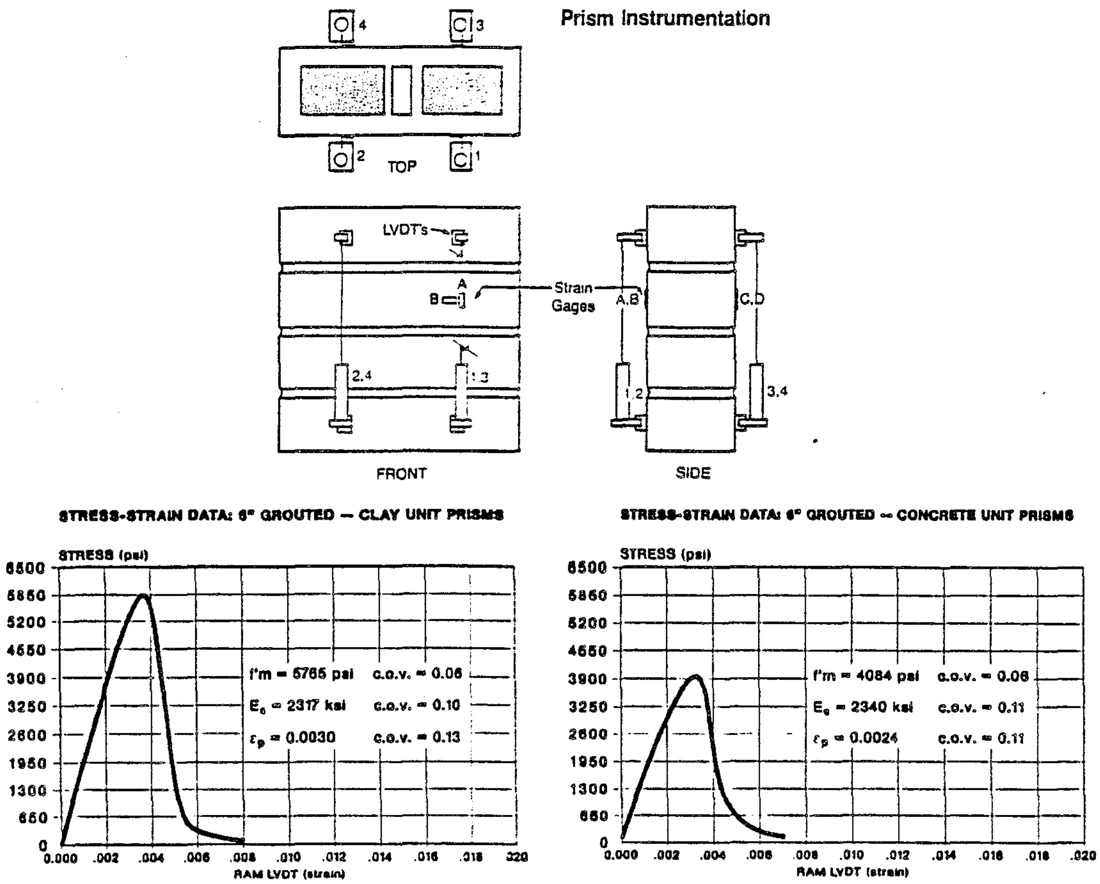

Fig. 3 Costitutive Prism Tests

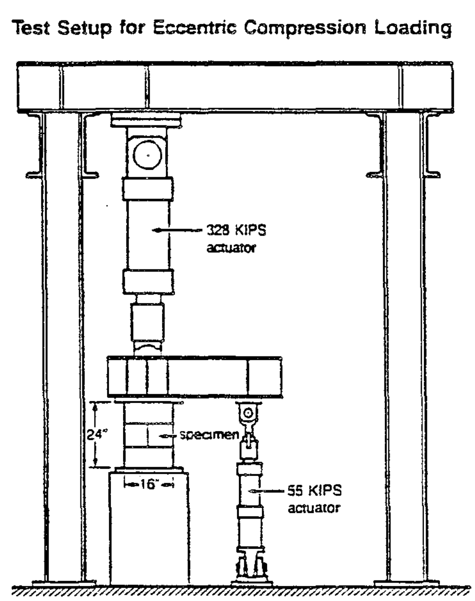

Fig. 4 Prism Tests under Eccentric Loading

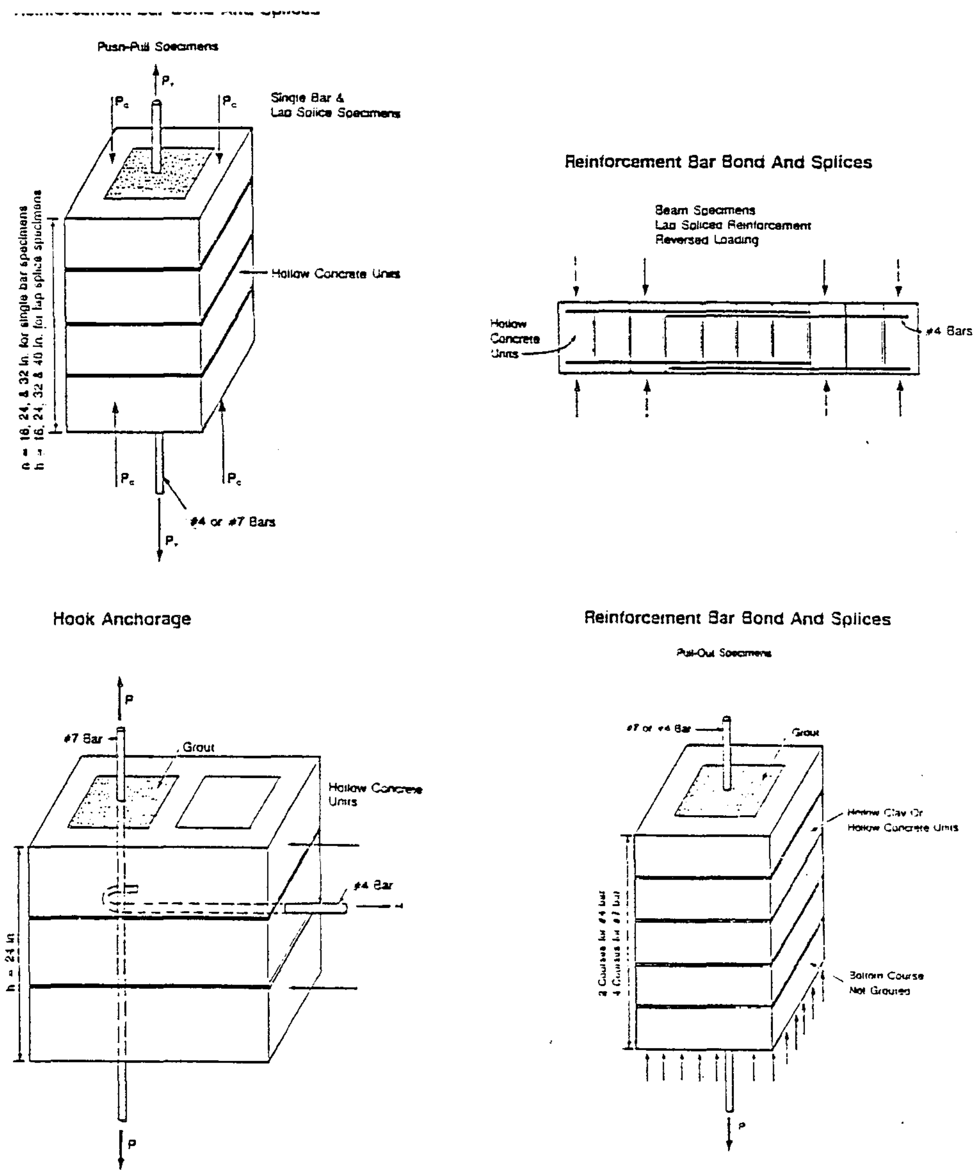

Fig. 5 Bond and Splice Tests

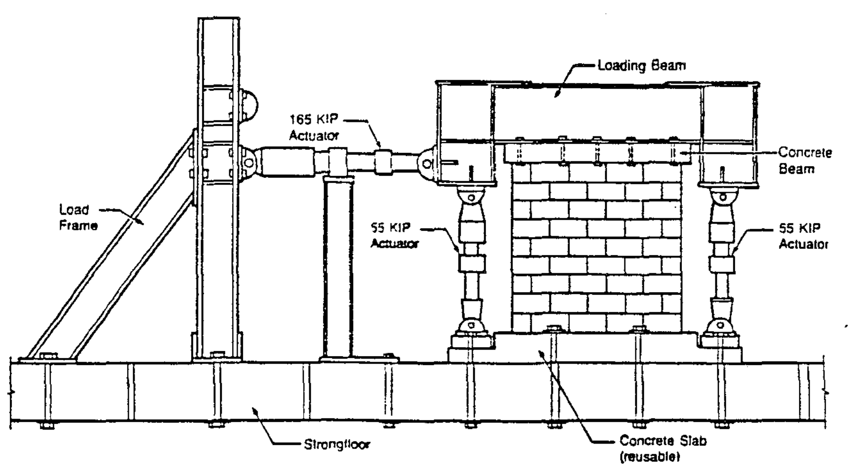

Fig. 6 Single Story Shear Walls

In-plane behavior of single story shear walls is under investigation by Shing et al. [9] at the University of Colorado using the test setup depicted in Fig.6. The test matrix includes variation of aspect ratios, horizontal and vertical reinforcement, and axial load levels to establish a full spectrum of characteristic structural component behavior. The dynamic out-of-plane behavior of reinforced clay masonry wall components is being studied by Mayes et.al. [10], utilizing a test setup at the University of California, Berkeley (see Fig.7). A parallel test module on dynamic out-of-plane response of concrete block masonry walls is scheduled to be conducted by Agbabian et al. at the University of Southern California. Comprehensive tests on floor-to-wall intersection,Fig.8, have been conducted and reported on by Hegemier et al.[11], and dynamic shaking table tests on flanged wall segments or wall-to-wall intersections are proposed by Priestley at the University of California, San Diego. Floor diaphragms consisting of full scale hollow core prestressed concrete planks and cast-in-place topping (a commomnly used floor system in masonry construction in the U.S.) are under investigation by Porter at Iowa State University using the test setup shown in Fig.9.

Sub-assemblages of complete reinforced masonry structural elements are investigated at the third level of the experimental sequence ,Fig.1. The sus-assemblages are directly tied to the precedeing and subsequent module levels in that they represent sub-structures of the prototype level comprised of individual elements from the component level. Thus, the experimental and with it also the analytical synthesis process of assembling structural systems from components will be tested at this stage. Also, with the introduction of overall structural behavior, the previously discussed GSD concept needs to be finalized and implemented.

Sub-assemblage modules of the five story full scale research building are being investigated at the University of Texas, Austin and at the University of California, San Diego respectively. Two story shear wall assemblages are under study by Klingner at Austin in an effort to obtain basic data on coupled shear walls and shear walls with openings, see Fig.10, and three story shear wall type substructures of the prototype research building are under investigation by Hegemier et al. in San Diego. The specimens in this test series are extracted from the five story research building as depicted in Fig.11b and will be tested in a test setup as shown schematically in Fig.11a. Parallel to the sub-assemblage test program described above is an experimental study, conducted by Abrams at the University of Illinois, on scale models. The models are scaled building substructures to be tested in a dynamic shaking table environment and compared to large scale quasi static tests. The test configuration is depicted in Fig.12.

The final level of the test sequence involves an experiment on a full scale five story research building, Fig.13, Wich, in itself, represents only a section of a multi-story reinforced

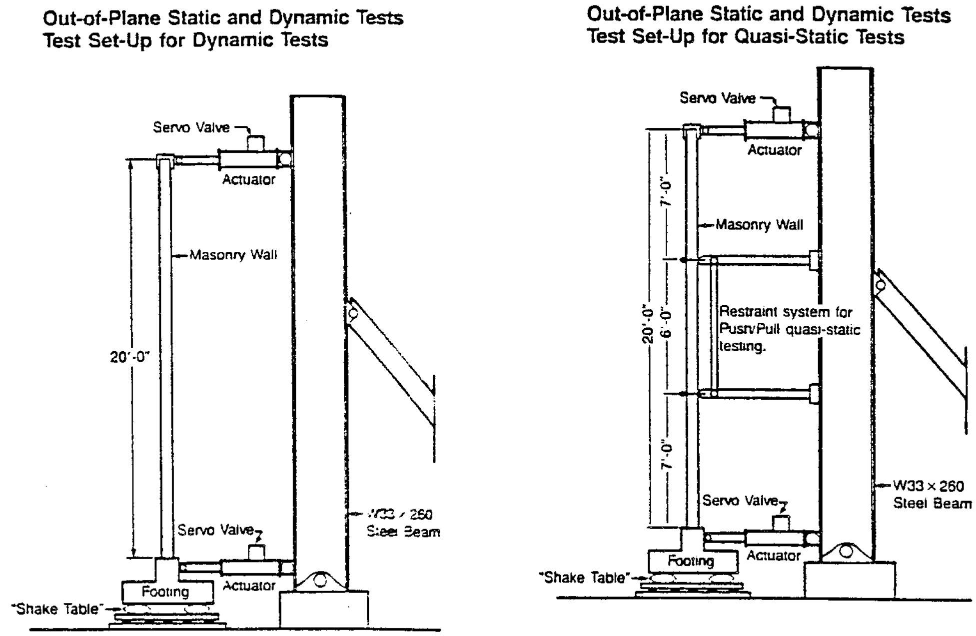

Fig. 7 Out-of-plane Wall Tests

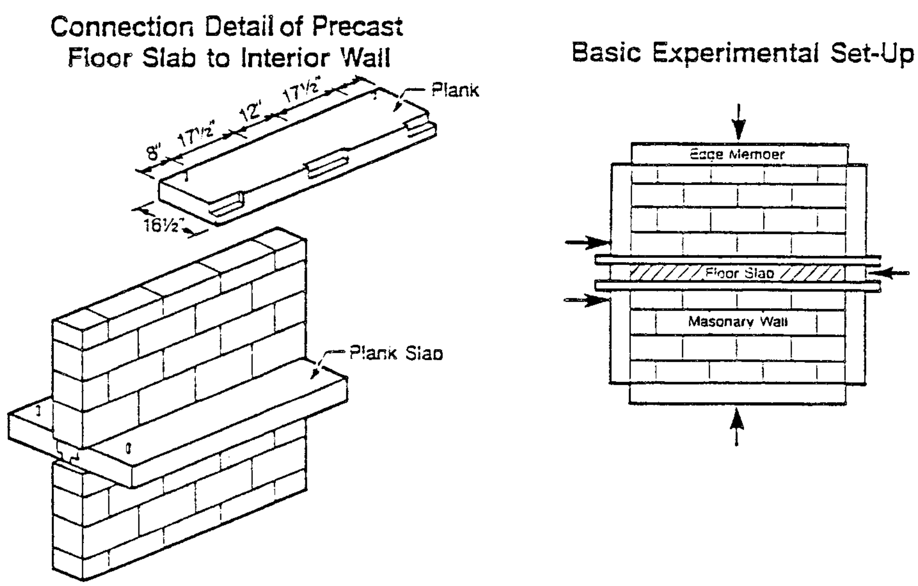

Fig. 8 Floor to Wall Intersections

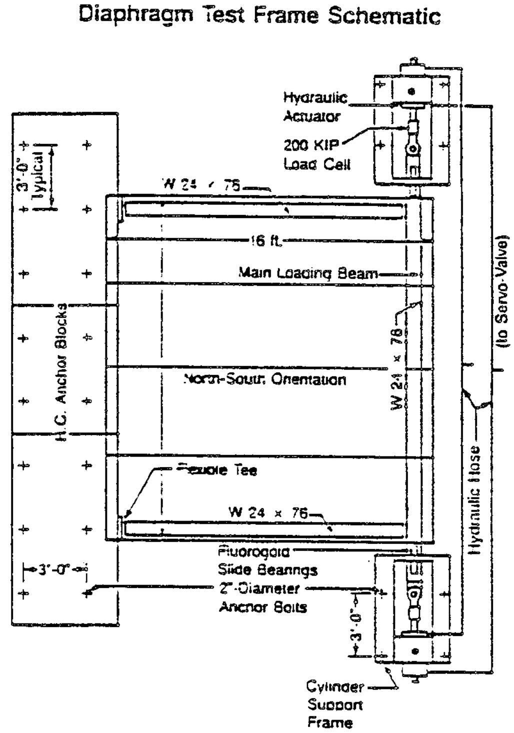

Fig. 9 Floor Diaphragm Tests

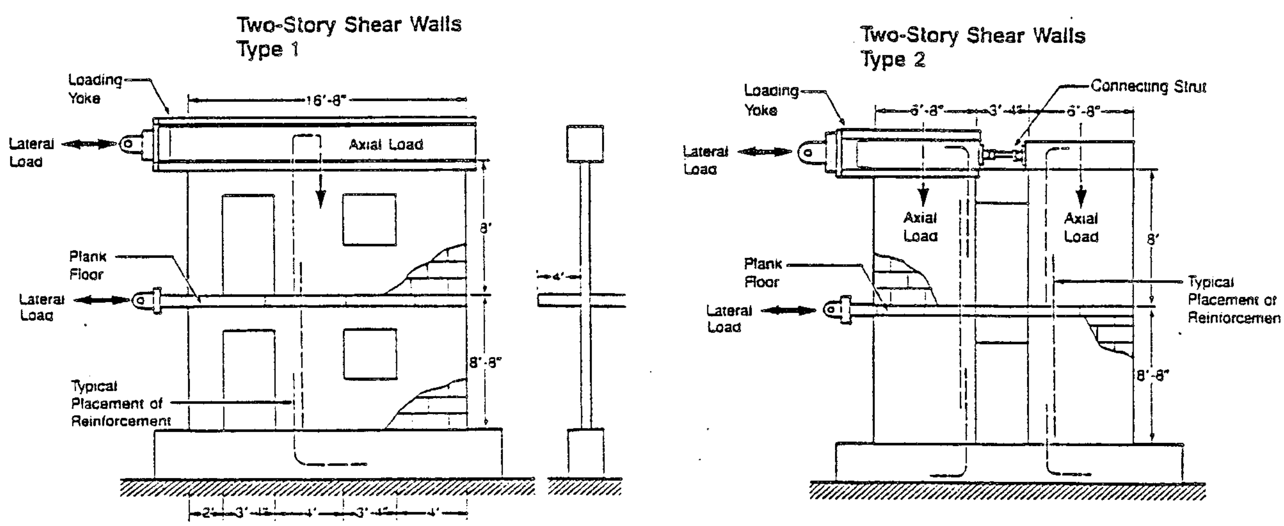

Fig. 10 Two Story Shear Wall Tests

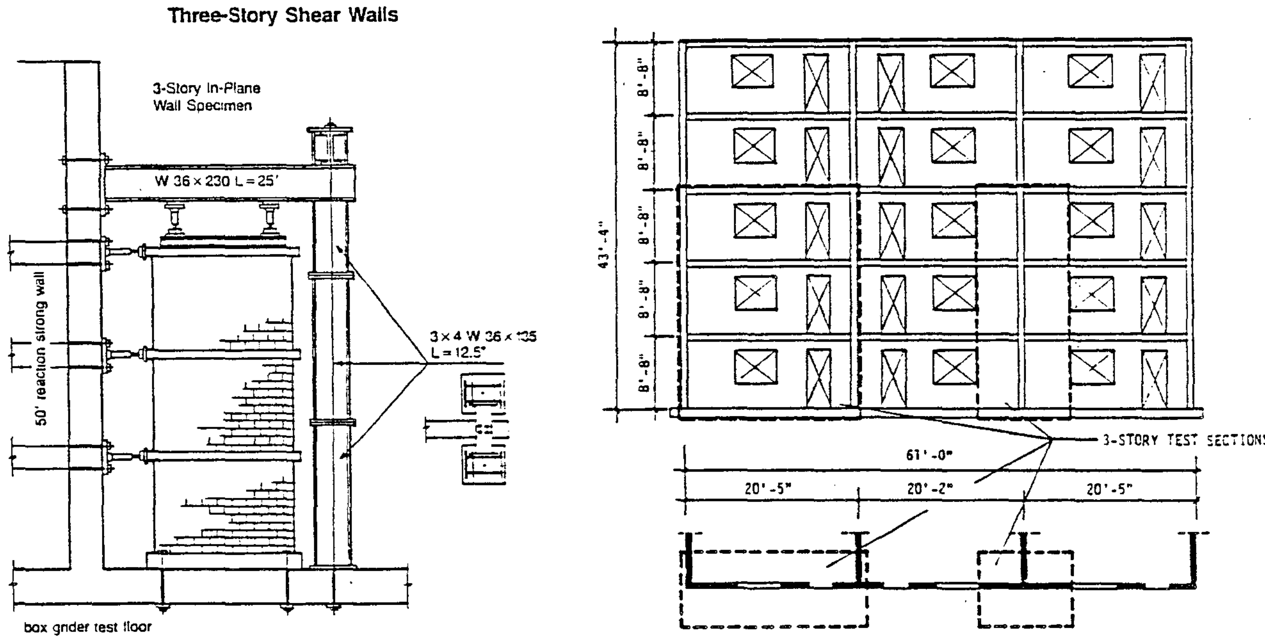

Fig. 11 Three Story Shear Wall Tests

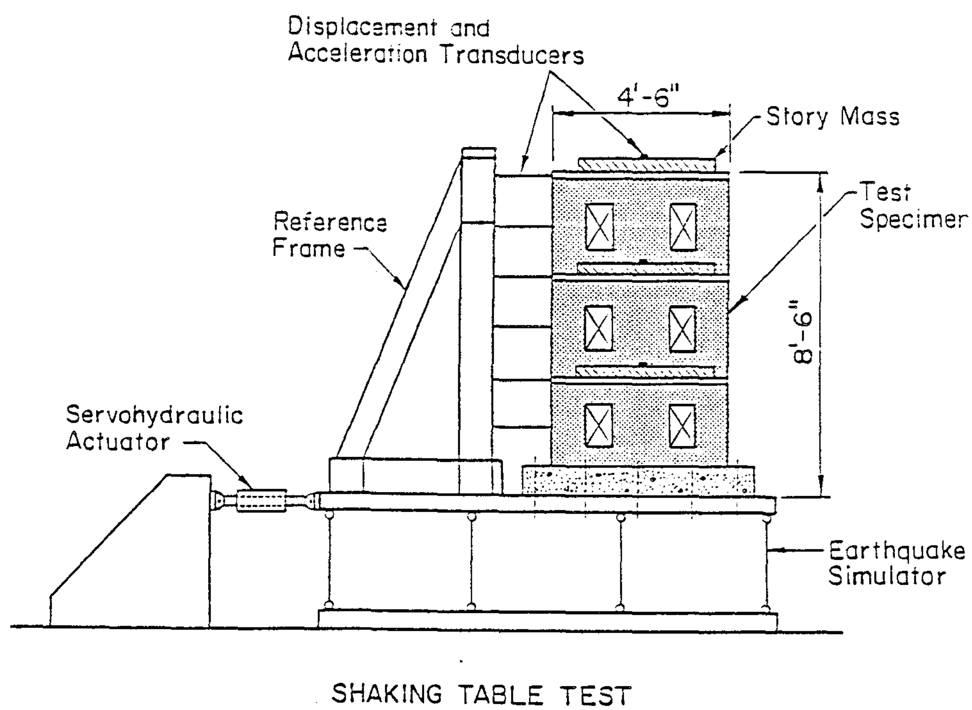

Fig. 12 Dynamic Response of Scale Models

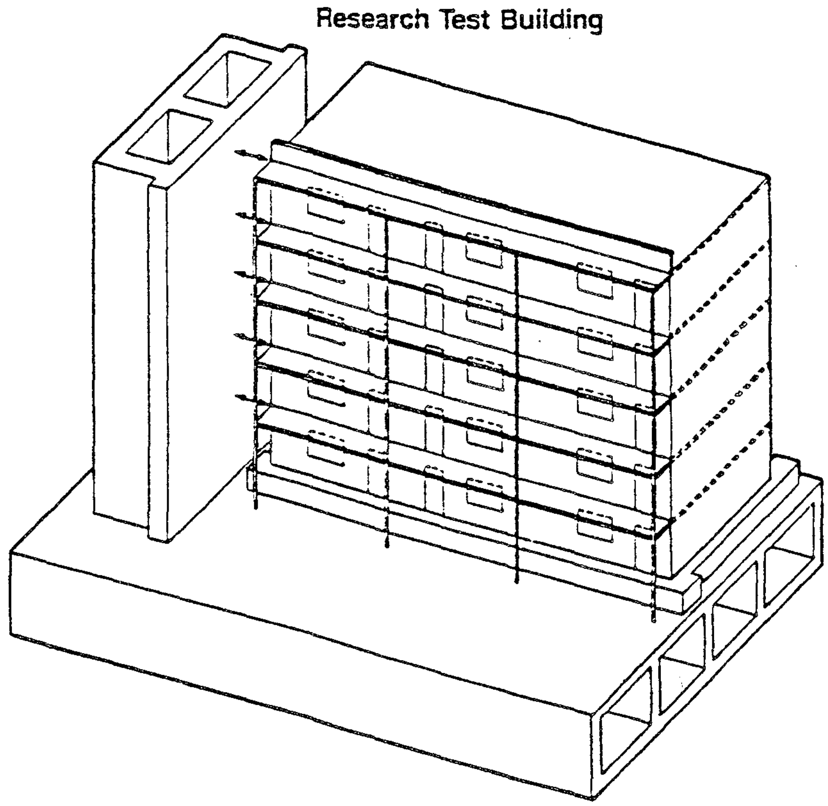

Fig. 13 Full Scale Research Building Section

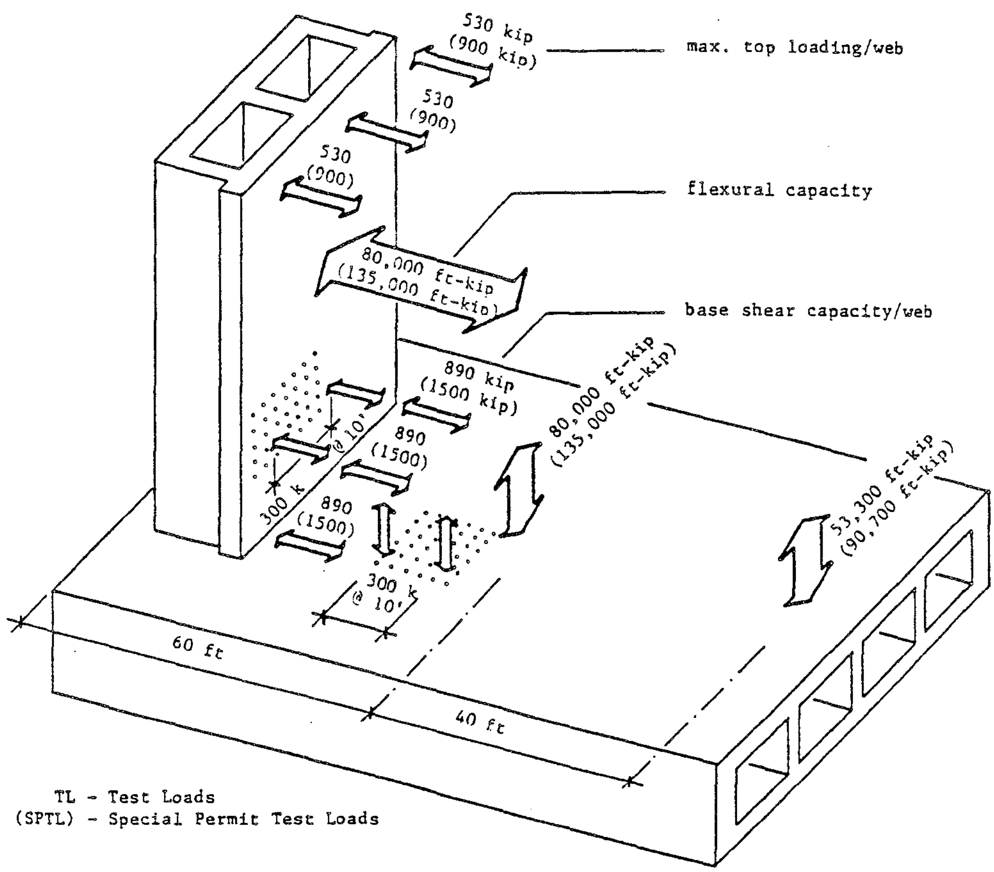

Fig. 14 Reaction Wall and Test Floor Capacities

masonry **prototype** structure. This test module will be a joint TCCMAR research effort with contributions from all the individual researchers. The experiment will be conducted in the Charles Lee Powell Structural Systems Laboratory at the University of California in San Diego. This reaction wall facility [12] was constructed in support of the U.S.-TCCMAR experimental program. The test load capacities of the heavily post-tensioned 50ft high two cell box girder reaction wall and the 120x50ft four cell box girder test floor are summarized in Fig.14. This full scale prototype test will indicate the degree to which overall structural behavior can be predicted under simulated seismic loads by a synthesis procedure which utilizes experimental and analytical component modules.

The availability of a five story prototype building which has been damaged from test loads, also offers a unique opportunity to study the effectiveness of possible repair and/or retrofitting procedures. This represents a potential expansion of the experimental prototype test level.

The combination of all of the above individual research modules by means of a coordinated analytical modeling effort will form the basis for the development of design models and detailed design recommendations.

CONCLUSION

The modular TCCMAR approach is an effort to systematically process and prepare the scientific data base for comprehensive design guidelines for masonry buildings in seismic zones. The large scale experimental research modules form a consistent structural system from materials,components and sub-assemblages to the prototype building. The principal objective of the modular experimental program is the development and validation of analytical and design models for masonry structures subjected to critical seismic loads. In order to achieve this objective the experimental testing methodology has to be developed from monotonic and cyclic loading to load histories which depend on and are interactive with the state of the test structure.

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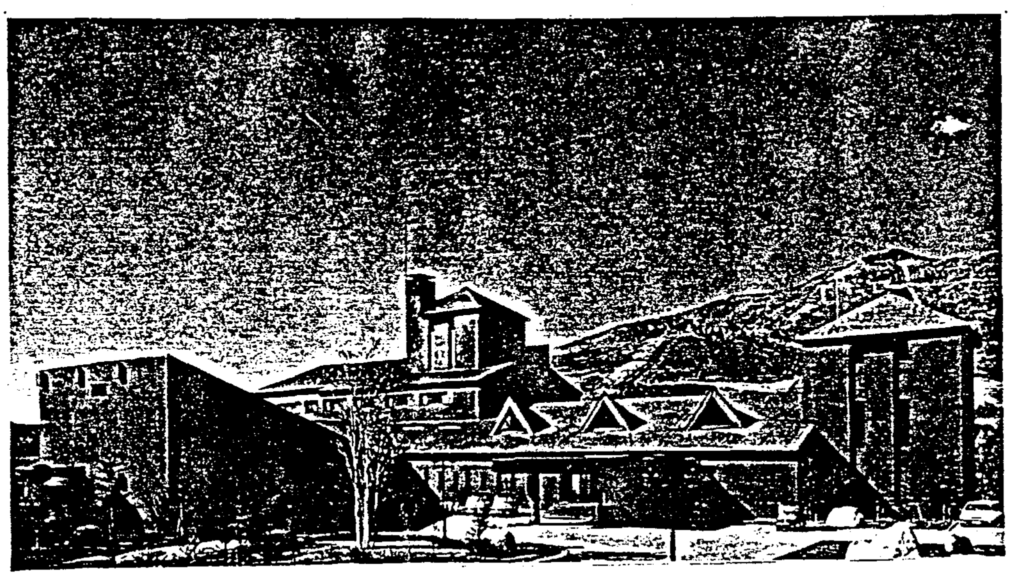
APPENDIX II

U.S.-JAPAN COORDINATED PROGRAM FOR MASONRY BUILDING RESEARCH

THE THIRD MEETING OF THE JOINT TECHNICAL COORDINATING COMMITTEE ON MASONRY RESEARCH

Chaired By J.L.Noland and T.Okada



OCTOBER 15, 16 AND 17, 1987, TOMAMU, HOKKAIDO, JAPAN
U.S.-JAPAN PANEL ON WIND & SEISMIC EFFECTS (UJNR)

Third Joint Technical Coordinating Committee on Masonry Research
- U.S.-Japan Coordinated Earthquake Research Program -
Tomamu, Japan, October 15-17, 1987

EVALUATION OF THE LOADING SYSTEM OF THE JAPANESE 5-STORY FULL SCALE MASONRY RESEARCH BUILDING

by

Frieder Seible[1], Yutaka Yamazaki[2], and Masaomi Teshigawara[3]

ABSTRACT

Full scale structural testing utilizing a reaction wall facility generally necessitates the load application at discrete points even though loads to be simulated are often of the inertia or mass proportional type. This approximation in the loading system requires a careful evaluation particularly in shear wall type structures where the stiffness of the horizontal load distribution system (floor slabs) is of similar order of magnitude as the lateral stiffness of the vetical support system. While post-tensioning of the floor system can preserve its stiffness integrety by eliminating possible critical tensile regions, post-tensioning of cast-in-place RC floors can introduce significant horizontal forces to the stiff vertical support system which may lead in turn to behavior modifications in the vertical support elements. The present study is an evaluation of the loading system of the Japanese five story reinforced concrete masonry research building to determine load approximation effects and optimum post-tensioning levels of the floor systems.

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INTRODUCTION

The full scale test of a 5-story reinforced concrete masonry research building at the Building Research Institute in Tsukuba Science City requires the load application at discrete points even though the critical inertia forces encountered during a real earthquake loading would be distributed proportional to the mass of the building. Solid reinforced concrete floor slabs provide a significant mass concentration at the floor levels which makes the load application through the floor slabs a realistic assumption. In frame type or flexible structural systems the load application at discrete points in the floor slab is a reasonable assumption since the in-plane or membrane stiffness of the reinforced concrete floor is significantly higher than the column stiffness in the adjacent stories. Thus, the floor slab acts as a rigid body providing similar horizontal displacement conditions to all vertical support members. In the case of shear wall type structures, the in-plane stiffness of the support elements can be in magnitude similar to the membrane stiffness of the floor slab which makes possible stiffness changes due to cracking in the floor system critical for the load transfer and force distribution to individual vertical support members.

Cracking and stiffness changes in the floor system can be eliminated by appropriate post-tensioning. However, it should be kept in mind that post-tensioning of an integrated structural part such as a cast-in-place RC floor always poses the problem of prestressing force transfer redundancy. Since some portion of the prestress will be transfered into the vertical shear wall elements as horizontal forces, the behavior of the wall elements may be artificially modified by this additional horizontal stress state.

A detailed evaluation of the floor system with appropriate consideration of the support system stiffness is therefore essential to determine the amount of approximation introduced by the concentrated load application and to design an appropriate external post-tensioning scheme which will allow a minimization of potential tensile or stiffness degradation zones in the horizontal load distribution system without modifying the behavior of the lateral support system.

GEOMETRY AND LOADING

For the evaluation as outlined above a typical floor system (floor levels 1 through 4) is being investigated rather than the top floor (level 5), since floors 1 through 4 feature a two point load application while the top floor is loaded at three discrete points. The basic philosophy of the loading of the full scale Japanese test structure is the application of increasing cyclic loads in a distribution directly obtained from the Japanese Seismic Design Code [1,2,and 3] (inverse triangular lateral force distribution with 12.3%, 15.1%, 18.4%, 22.4% and 31.8% for floor levels 1 to 5 respectively). The cyclic forcing function for the builing is applied in displacement control of the top story actuators while the remaining floor level loads are slaved to the top floor loading in force control mode using the above force distribution. Thus, the two versus the three actuator arrangement in the 4th and 5th story respectively will generate slightly larger concentrated loads at the 4th floor level and, with only two concentrated loads applied, the more critical force distribution for the floor slab.

Overall geometry and loading arrangement of a typical floor plan with two point load application is shown in Fig.1. Loads are transmitted from the reaction wall and the actuators through structural steel wide flange girders to 1.00m x 0.90m x 1.60m reinforced concrete blocks which are monolythically connected to the RC floor slab along the centerline (X2 axis) of the building.. The continuous cast-in-place floor slab has a uniform thickness of 150mm except for a 2.99m wide spine along the centerline which has been increased in thickness to 200mm for load distribution purposes. The load application is perpendicular to the centerline (axis X2) at the two load points as indicated in Fig.1. Detailed information on the dimensions of the test building can be obtained from [4] and for a typical floor slab from Fig. 3.

With estimates for the ultimate lateral load capacity of the building ranging from 600 to 800tons a 22.4% floor level contribution at floor level 4 can result in 180tons or 90tons per actuator. Since actuators of 100ton capacity are employed, this maximum actuator capacity of 100tons should be used to assess critical stress level in the floor system conservatively. The total load to be applied to the floor system for a detailed membrane analysis is therefore 2x100tons or 2MN.

PRELIMINARY EVALUATION OF THE LOADING SYSTEM

A first estimate of the possible approximations in the loading system due to the concentrated load application of simulated seismic loads at a typical floor level can be obtained from a comparative analysis of a rigid floor system and a floor system with in-plane flexibility. With symmetry of the structure employed to analyze only one half of the floor system, the denominations of individual shear walls are given in Fig.2, together with the schematic assumptions for the preliminary analysis.

Individual shear wall stiffnesses for lateral in-plane loads can be derived from the combined flexural and shear deformation capabilities under the conservative assumption that no significant relative member end rotations per story height occur.

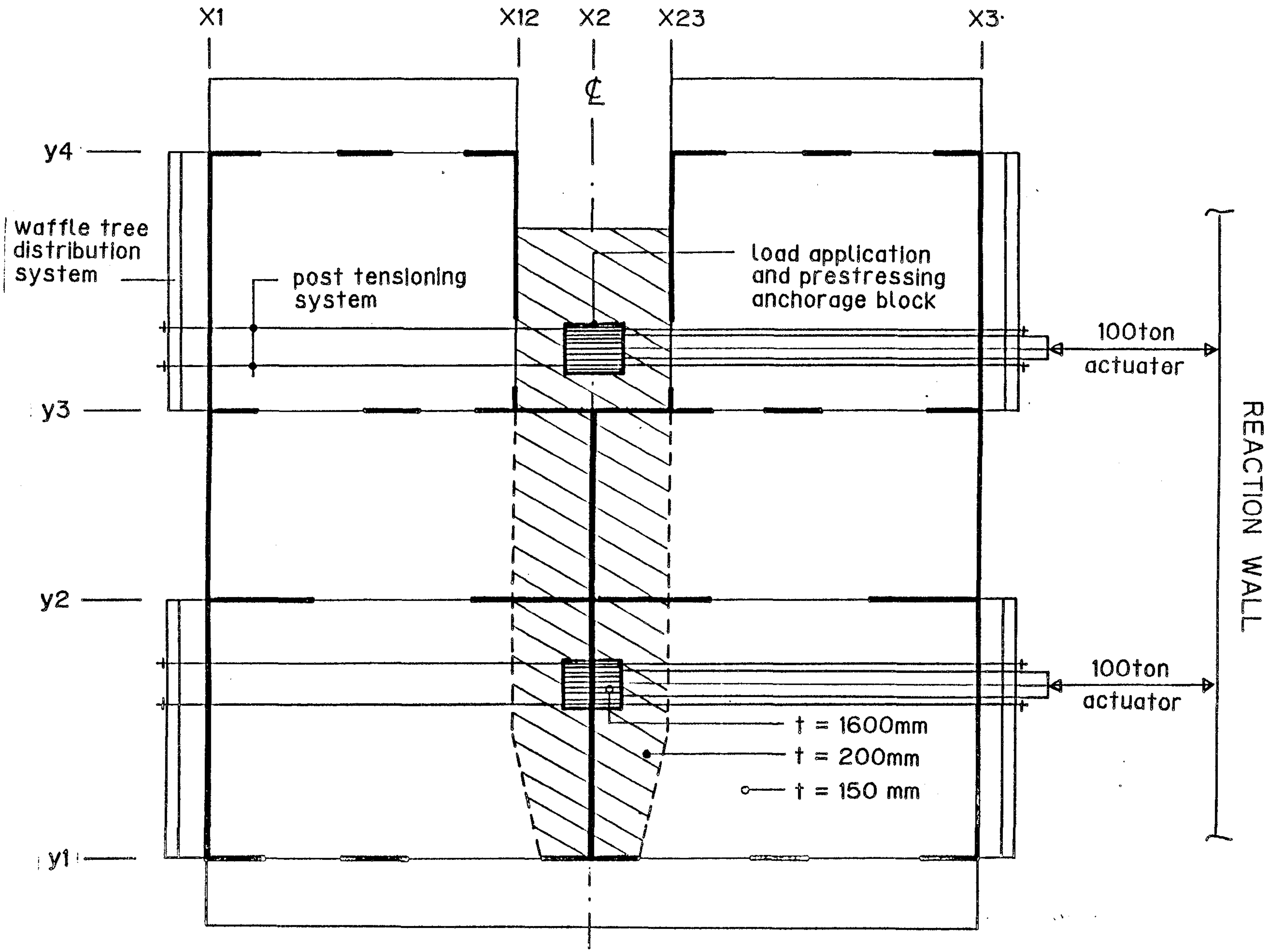

Fig. 1 PLAN VIEW OF TYPICAL FLOOR SLAB

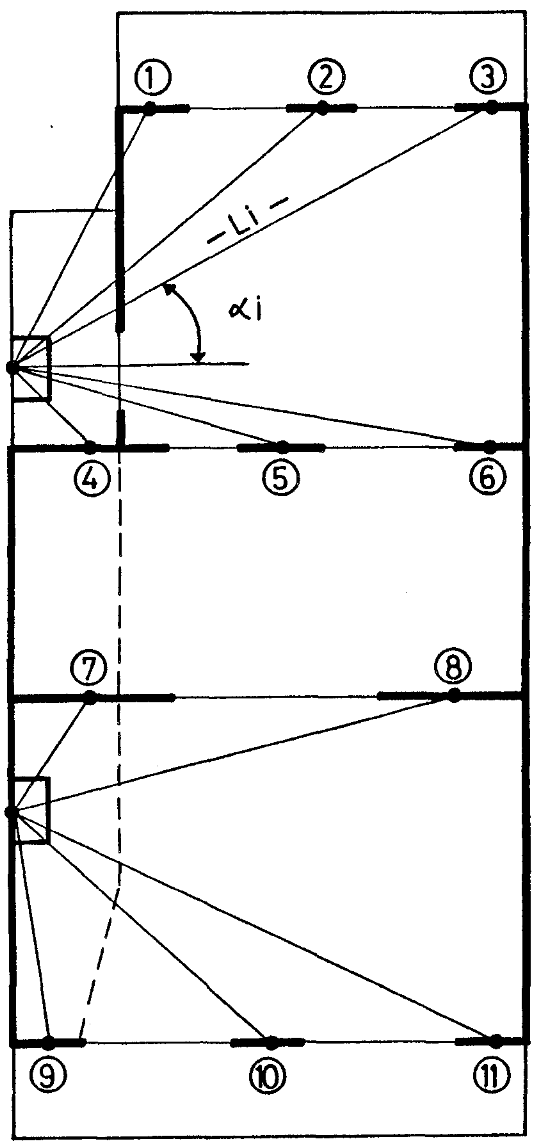

Fig. 2 SHEAR WALL DESIGNATION

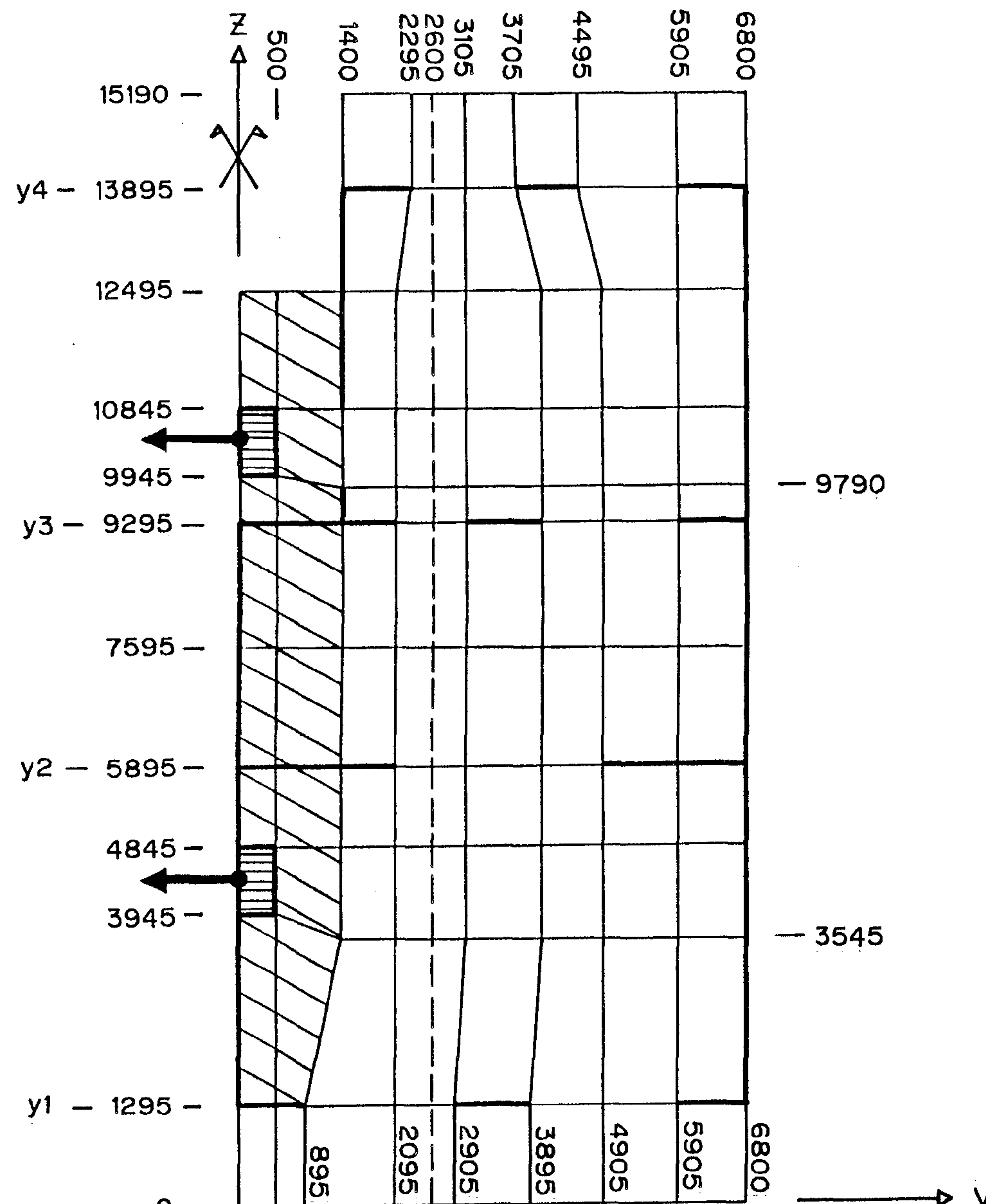

Fig. 3 PLANE STRESS ANALYSIS MODEL

Based on classic structural theory [5], with the flexural deformation in a wall element expressed as

$$H^3/(12E_mI) \quad (1)$$

and the shear deformation expressed as

$$H/(GA_v) \quad (2)$$

(where H=the story height of 2.80m, E_m=the modulus of elasticity for the fully grouted masonry walls, I=the moment of inertia for in-plane bending, G=the shear modulus and A_v=the shear area), an estimate of the initial combined flexibility and with it the overall in-plane stiffness of each wall element can be obtained. A summary of the individual stiffness parameters is given in Table 1.

For the preliminary evaluation of the loading system a first assumption of a rigid floor diaphragm can be made which simply distributes the applied loads to the individual wall elements proportional to their relative stiffness and probably very similar to inertia type loading. The other extreme can be obtained by assuming a flexible (in-plane) floor slab with displacement contributions to the individual walls in the direction of and inverse proportional to their centroidal distance from the point of concentrated load application (see Fig.2). Thus, a set of relative reaction forces can be obtained which are summarized in Table 2, together with the rigid floor slab results, in terms of percentage contribution of the total applied floor load. From this simple comparison of extreme cases the loading problem becomes obvious with differences in the largest reaction forces (wall element 4) of 17%, which, in terms of a max. applied floor load of 180tons, costitutes a load difference of 31tons for this one wall element (total) alone. With the actual conditions being somewhere between these two extreme cases, but with a clear tendency towards the rigid case, it is obvious that a detailed plane stress analysis [6,7] of the floor and support system is needed to evaluate the approximation involved.

EVALUATION OF LOADING SYSTEM

The plane stress model investigated is depicted in Fig.3 with dimensions as shown and a concrete modulus of elasticity and a poisson's ratio of 23000N/mm^2 and 0.15 respectively. Shear walls in the y-direction (loading direction) were introduced in terms of boundary element springs equivalent to the individual wall stiffness values listed in Tabel 1 and a modulus of elasticity for the fully grouted masonry of E_m=20000N/mm^2. Displacements in the z-direction are constraint along the building symmetry axis and the shear walls in the z-direction, since torsional effects are mostly eliminated by the top floor actuator control.

Table 1 - INDIVIDUAL SHEAR WALL STIFFNESS

Wall No.	Length L [mm]	$I=tL^3/12$ [mm^4]	$A=tL$ [mm^2]	Flexibility x E_m			Stiffness [N/mm^2]
				flex. [mm]	shear [mm]	total [mm]	
1	990	1.54 E10	188 E3	.119	.041	.160	6.25xE_m
2	790	0.78 E10	150 E3	.235	.052	.287	3.48xE_m
3	990	1.54 E10	188 E3	.199	.041	.160	6.25xE_m
4*	2095	116.5 E10	398 E3	.003	.020	.023	44.2xE_m
5	990	1.54 E10	188 E3	.119	.041	.160	6.25xE_m
6	990	1.54 E10	188 E3	.119	.041	.160	6.25xE_m
7*	2095	116.5 E10	398 E3	.003	.020	.023	44.2xE_m
8	1990	12.4 E10	378 E3	.015	.021	.036	27.8xE_m
9*	895	9.08 E10	170 E3	.040	.046	.086	11.6xE_m
10	990	1.54 E10	188 E3	.119	.041	.160	6.25xE_m
11	990	1.54 E10	188 E3	.119	.041	.160	6.25xE_m

* $L=2L_i$ and $I=I_i/2$ for flexure due to symmetry
$H=2800mm$ $t=190mm$ $G=0.43E_m$

Table 2 - COMPARISON OF INDIVIDUAL SHEAR WALL CONTRIBUTIONS

Wall No.	Distance [mm]	Angle α [Deg]	Rigid Floor %P	*Flex. Floor %P
1	3958	62.2	3.70	1.70
2	5391	40.5	2.06	1.10
3	7253	28.9	3.70	1.70
4	1591	46.4	26.19	43.30
5	3764	17.0	3.70	3.60
6	6447	9.8	3.70	2.20
7	1830	55.1	26.19	31.20
8	6042	14.4	16.47	10.10
9	3132	81.8	6.87	1.20
10	4601	42.4	3.70	2.30
11	7068	26.0	3.70	1.80

*$(K_i \cos\alpha)/L_i$

Three load cases were analyzed, namely (1) two concentrated loads pulling at the loading points indicated in Fig.3, (2) a corresponding uniformly distributed in-plane load over the entire floor slab and (3) a set of prestressing forces as indicated by the post-tensioning system in Fig.1. Load levels in load cases (1) and (2) were set at the maximum possible floor actuator capacities of 2MN or 200tons.

Individual wall element reaction percentage contributions to the total applied floor loading are summarized in Table 3 and show only small differences between the point load and the distributed load application. In addition, load case (1) wall contributions in Table 3 are close, as expected, to the rigid floor slab contributions in Table 2, which indicates that the rigid floor slab assumption and with it the concentrated point load application is still admissible for the present stiff lateral support system.

The approximation level for individual wall elements due to the concentrated load application can best be determined by

$$\nabla\ [\%] = \frac{[\%](1)\ -\ [\%](2)}{[\%]\mathrm{rigid}} \tag{3}$$

with reference to the rigid floor contribution of the particular element. While some of the lesser loaded elements show percentage differences of up to 17% between the point load and the distributed load application, the majority of wall elements features differences well below the 10% level, in particular wall elements 4,7 and 8 which carry together about 70% of the applied floor loading.

An in-depth study of the sensitivity of this loading approximation to the assumed stiffness parameters and their possible changes is summarized in Table 4 and Fig.4 on the selected case of wall element No.4 (Fig.2), which attracts the largest portion of the applied load. Rather than evaluating wall element 4 in terms of the total applied floor loading, Table 4 and Fig.4 are presented in terms of the rigid floor load contribution of element 4 or 26.2% of the total applied floor load. For various stiffness levels of the overall lateral support system, changes in wall element 4 contributions are evaluated and show for the selected reference stiffness of 1.0 (derived in Table 1) a 5.19% increase over the rigid floor case for concentrated load application and a 2.48% decrease for distributed load application. The total difference between concentrated and distributed load application in terms of rigid floor contribution is 7.67%, which can be also obtained from equation (3). This represents a good measure for the approximation introduced by the selected loading arrangement. The sensitivity of tis approximation to the proper determination of the lateral support stiffness parameters can be seen in Fig.4 and is quantified in Table 4, with values ranging from 4.04% to 13.81% for one half and double the

Table 3 - INDIVIDUAL SHEAR WALL CONTRIBUTION IN PERCENT OF THE TOTAL FLOOR LOADING

Wall No.	Load Case (1) Point Loads [%]	Load Case (2) Distributed [%]	Load Case(3) Prestressing [%]
1	3.77	4.35	0.11
2	2.11	2.45	0.12
3	3.69	4.33	0.64
4	27.56	25.55	4.15
5	3.78	3.90	0.10
6	3.64	4.02	1.65
7	26.37	24.36	3.58
8	15.01	15.94	4.65
9	6.77	7.24	0.22
10	3.83	3.95	0.03
11	3.62	3.93	0.66
Total	100 %	100 %	16 %

Table 4 - CHANGE IN WALL ELEMENT 4 LOADING WITH STIFFNESS VARIATION IN THE SUPPORT SYSTEM

Lateral Stiffness Factors	0.1	0.2	0.5	1.0	2.0	5.0	10.0	100.0
*Two Point [%] Loading	0.57	1.18	2.82	5.19	8.85	15.46	20.80	29.54
*Distri-buted [%] Load	-0.27	-0.50	-1.22	-2.48	-4.96	-11.11	-17.90	-38.63
*Load Diffe- [%] rence	0.84	1.68	4.04	7.67	13.81	26.57	38.70	68.17

*100% reference load is the rigid floor loading of wall element 4

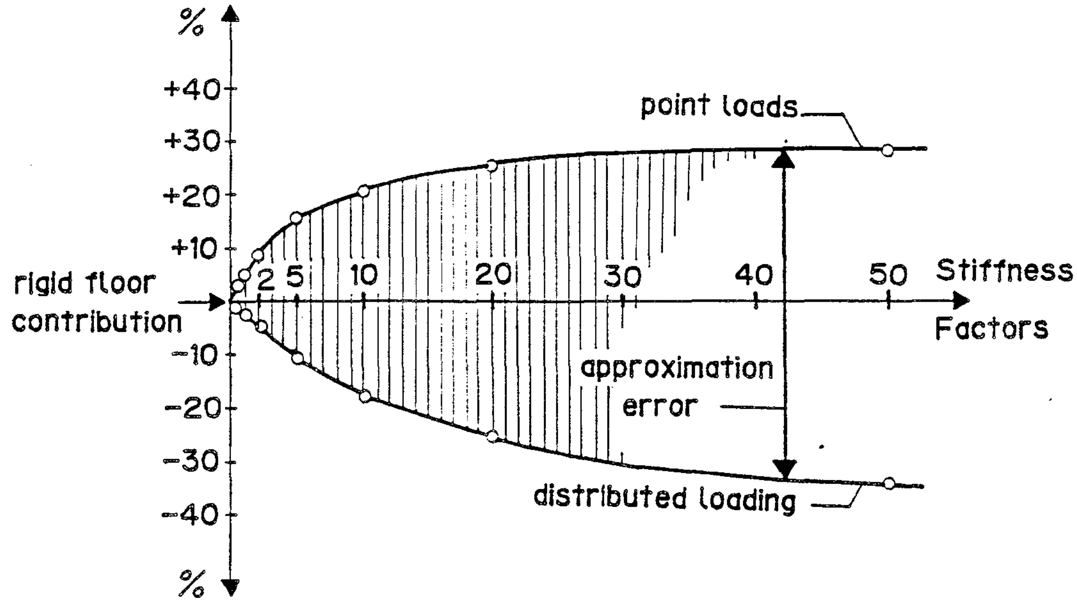

Fig. 4 LOADING APPROXIMATION DEVELOPMENT FOR WALL ELEMENT 4

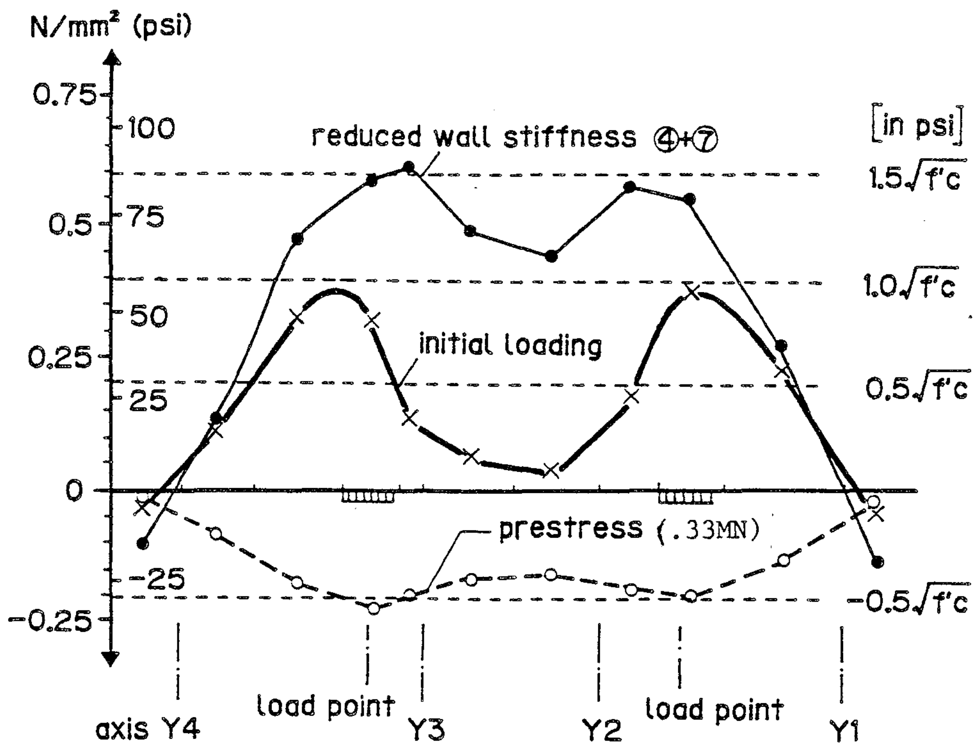

Fig. 5 FLOOR STRESS LEVELS AT Y=2600mm UNDER 2P=2MN FLOOR LOADING

assumed lateral stiffness respectively. Effects of stiffness degradation in the lateral support system and in the floor slab can be qualitatively evaluated based on the indicated behavior in Fig.4, with initial wall deterioration reducing the approximation error followed by increases with stiffness degradations in the floor slab.

Based on this evaluation it can be concluded that the approximations introduced by the discrete loading system to represent inertia type seismic load distributions are generally less than 10% which can still be considered quite satisfactory.

DISCUSSION OF PRESTRESS LEVEL

Prestressing of the floor system as schematically indicated in Fig.1 was proposed to preserve the structural integrety and load distribution capacity of the floor system even at significant damage levels in the building. Any such prestressing however requires consideration of the effect on the lateral support system. In particular, how much of the applied prestressing force is directly transfered to the lateral support system, and the axial stress levels in the floor system have to be determined.

Reaction contributions of the idividual wall elements to the total applied prestressing force are given in Table 4. It can be seen that a total of 16% of the applied prestress is transfered to the supporting wall elements and that the largest individual wall contribution is at wall element 8 with 4.65% of the total applied prestressing force. It should be noted that this analysis only considers the lateral stiffness contributions from a single story level and that contributions from two story levels can be estimated from the tendencies in Fig.4 for twice the lateral stiffness values to more than 1.5 times the values indicated in Table 4. Thus, only about 75% of the applied post-tensioning force will be transfered as effective prestress into the floor system at an intermediate floor level, with up to 25% being absorbed as reaction forces in the wall elements.

Even more important are stress levels in the floor system particularly along a line of y=2600mm (see Fig.3), where large tensile forces have to be transfered in load caes (1) to the wall elements y>2600mm. Fig.5 depicts the y-stress levels along this line with a maximum tensile stress of approximately o.4N/mm^2 or $1\sqrt{f_c'}$ in American [psi] units. If the two major contributing wall elements 4 and 7 experience significant structural deterioration (e.g. $K_i/100$) the stress levels in the floor system along line y=2600mm would only increase to about 0.6N/mm^2 or $1.5\sqrt{f_c'}$ [psi]. These stress levels in the floor system are so small that virtually no prestress is required to eliminate potentially critical tensile zones in the floor system. If the intend is to effectively eliminate these initial tensile stress regions, a total prestress force of approximately .67MN or 67tons would be required. The stress distribution for half that level or a total prestress force of about 33tons is indicated for comparative purposes in Fig.5, which would reduce initial tensile stress levels in the floor slab below $0.5\sqrt{f_c'}$ [psi]. Since axial stress level in critical elements for the formation of yield hinges, such as floor-beam connections over doorways and openings, may artificially influence the member behavior, and since the overall tensile stress levels are clearly well below critical cracking limit states, only a nominal level of prestressing is suggested which would preseve the structural integrety of the floor slab at large deformation levels beyond the maximum lateral force limit state of the test building.

CONCLUSIONS

The detailed analysis of the loading system for the Japanese five story full scale reinforced masonry building test has shown that the approximations introduced by simulating distributed inertia type forces with concentrated loads at the floor levels are mostly within 10% of the total applied floor loading and therefore still acceptable. Based on strictly linear elastic floor system evaluations, the potential tensile stress levels in the horizontal load distribution system under the applied concentrated loads are of magnitudes which do not warrant any significant level of prestress. In order to preserve the floor slab integrety at large deformation levels and after the formation of plastic hinges, a nominal level of prestress may be selected which does not significantly influence the initial behavior of the horizontal load transfer system.

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APPENDIX III

THE JAPANESE 5-STORY FULL SCALE REINFORCED CONCRETE MASONRY TEST - Design and Construction of the Test Building

by Frieder Seible, Tsuneo Okada, Yutaka Yamazaki and Masaomi Teshigawara

THE JAPANESE 5-STORY FULL SCALE REINFORCED CONCRETE MASONRY TEST - Loading and Instrumentation of the Test Building

by Frieder Seible, Yutaka Yamazaki, Takashi Kaminosono and Masaomi Teshigawara

THE JAPANESE 5-STORY FULL SCALE REINFORCED CONCRETE MASONRY TEST - Forced Vibration and Cyclic Load Test Results

by Frieder Seible, Yutaka Yamazaki, Hatsukazu Mizuno and Takashi Kaminosono

THE JAPANESE 5-STORY FULL SCALE REINFORCED CONCRETE MASONRY TEST - Ultimate Load Test Results and Analytical Correlation Studies

by Frieder Seible, Yutaka Yamazaki, Takashi Kaminosono and Masaomi Teshigawara

APPENDIX IV

THE JAPANESE 5-STORY FULL SCALE REINFORCED CONCRETE MASONRY TEST - Design and Construction of the Test Building

by Frieder Seible, Tsuneo Okada, Yutaka Yamazaki, and Masaomi Teshigawara

KEYWORDS: Bonding System, Construction Technology, Design Guidelines, Experimental Testing, Full Scale Test, Limit State Design, Reinforced Masonry

ABSTRACT: The Japanese 5-story full scale reinforced masonry building test at the Building Research Institute (B.R.I.) in Tsukuba City, Japan, is a key component of the U.S.-Japan Joint Cooperative Research Program on Masonry Buildings under Seismic Loads. The full scale test building was designed following the new Japanese guidelines for medium rise reinforced masonry buildings. The key features of this new limit state based design standard are summarized, and the design of the test specimen is discussed.

The test building was constructed with a newly developed bonding system which utilizes openended standard units for a complete modular, fully grouted, reinforced, running bond system. The construction of the test specimen is described and quality control data and material properties are summarized.

THE JAPANESE 5-STORY FULL SCALE REINFORCED CONCRETE MASONRY TEST - Design and Construction of the Test Specimen

by

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INTRODUCTION

Under the auspices of the UJNR (United States - Japan Cooperative Program on Natural Resources) on Wind and Seismic Effects, both, the U.S. and Japan are working since 1984 on a coordinated research program on masonry structures in seismic zones. The JTCCMAR (Joint Technical Coordinating Committee on Masonry Research) is the third U.S.-Japan coordinated earthquake research program, preceded by comprehensive joint research efforts into the seismic behavior of steel and reinforced concrete frame structures.

The objective of the JTCCMAR program is the development of improved technology for masonry structures in the materials, construction and design areas in order to make masonry structures an economical as well as reliable alternative for buildings in zones of various seismicity. Based on a detailed understanding of the structural behavior of masonry buildings derived from common analytical and experimental research programs on materials, components, sub-assemblages and prototype structures, each country will develop modern design guidelines which reflect the state-of-the-art in masonry technology, applicable to the individual country. While the U.S. program focuses on the development of general design guidelines for generic masonry buildings of various geometry and size for different regions in the U.S., the primary goal of the Japanese program is the development of comprehensive design guidelines for medium rise masonry structures, in particular, the five story apartment building, to meet that countrie's need of high density residential construction. The JTCCMAR research plan calls for the verification of analytical and design models by means of a 5-story full scale prototype test on a masonry building segment in each country, reflecting the individual masonry technologies.

Even though there are some differences in the design philosophies for the U.S. and Japan full scale research buildings, the similarities in the analytical modeling effort and in the experimental research programs on materials, components and sub-assemblages, will provide invaluable generic behavioral data on reinforced masonry structures subjected to simulated seismic loads. Therefore, an attempt is made in this paper and in subsequent reports to summarize the design, construction, instrumentation, loading, structural response and analytical correlation studies of the Japanese 5-story full scale reinforced masonry building test, see Fig.1, in order to stimulate independent interpretation and analytical modeling by other researchers.

In this first paper, the basis for the design of the Japanese 5-story full scale test building is established and the construction of the test specimen is documented.

DESIGN OF THE TEST BUILDING

Geometry and Dimensions

The test specimen represents one module of a typical 5-story apartment building as cited in a design example [1] of the A.I.J.(Architects Institute of Japan). The typical floor plan and elevations of a 5-story building module are shown in Fig.2. Prototype apartment buildings generally consist of one to three of these building modules as indicated in Fig.2 in the east-west direction which is also the selected loading direction for the full scale test, in order to investigate the structural behavior of walls Y1-Y4 which are heavily penetrated with openings.

The door and window openings effectively produce frame

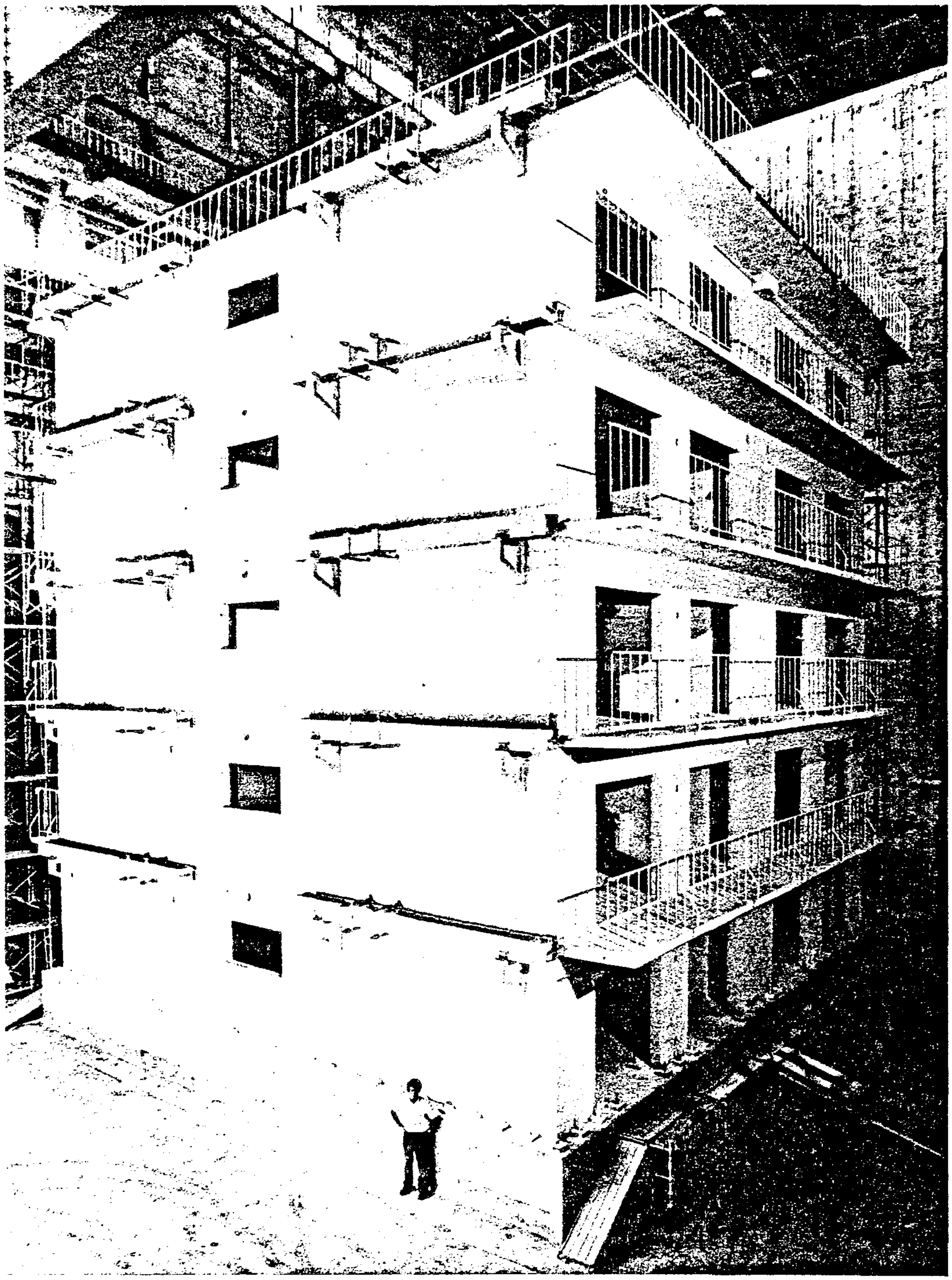

Fig. 1 5-Story Full Scale RM Test Building

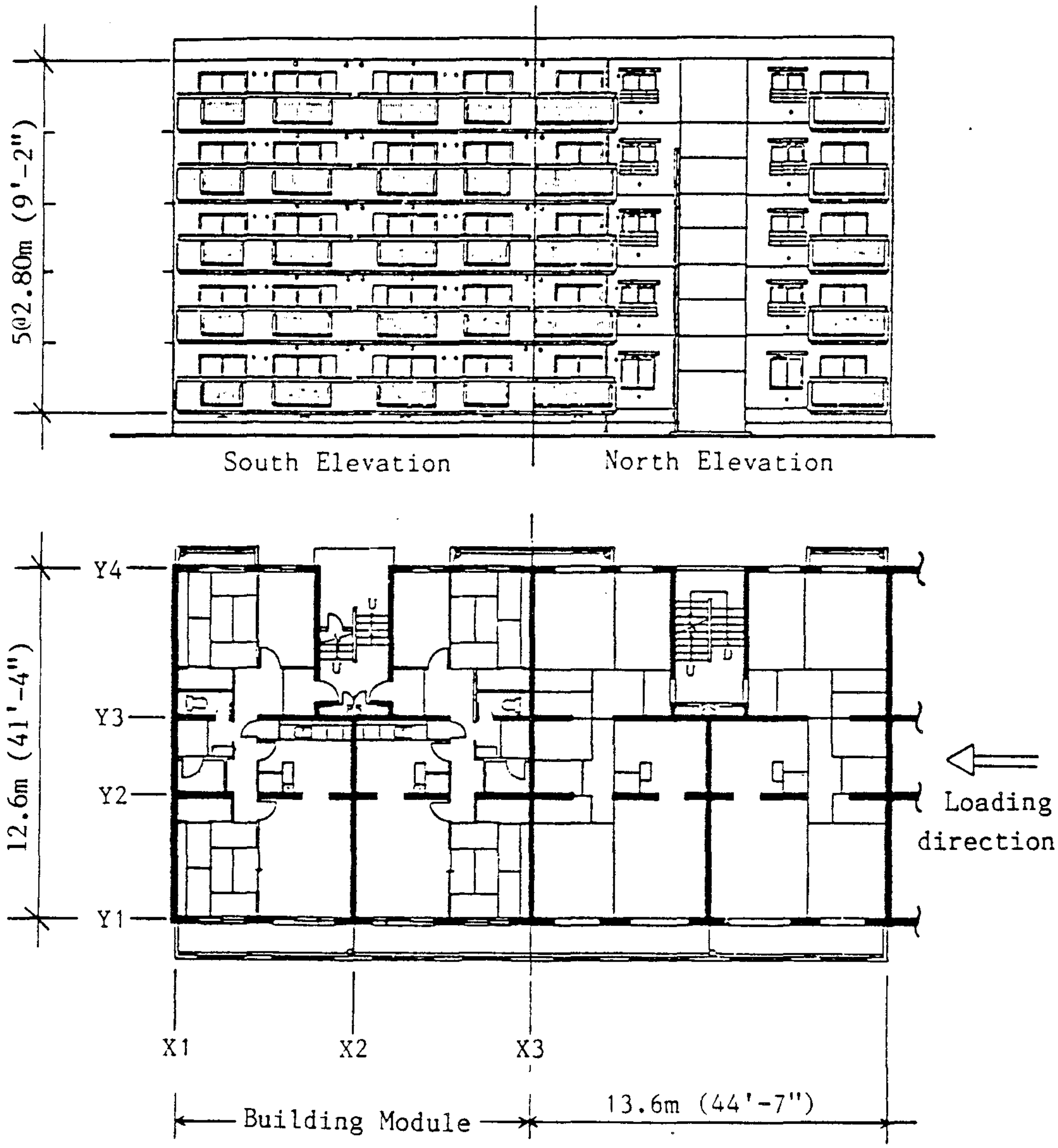

Fig. 2 Prototype Building Modules

type or coupled shear wall type planar structures in the loading direction and are thus referred to in the following as lateral load resisting frames Y1-Y4. Geometry and key dimensions of these four lateral load resisting frames are indicated in Fig.3. Each of the four frames consists of a series of wall/column(W) and girder/beam(G) elements with their individual designations shown in the typical floor plan of the test building, Fig.4.

The typical story height is 2.80m(9'-2") including a 150mm(6") reinforced concrete floor slab which increases in thickness to 200mm (8") along the central loading strip, see Fig.4, and above the masonry walls to accommodate the vertical 200mm (8") module (T) in masonry concrete block construction.

General Design Criteria

The design of the Japanese 5-story reinforced masonry test building reflects the new proposed Japanese design guidelines for medium rise masonry structures [2] which differ significantly from the current Japanese design practice. The previous design guidelines for masonry structures were predominantly in the form of structural specifications such as e.g. strict limitations on floor

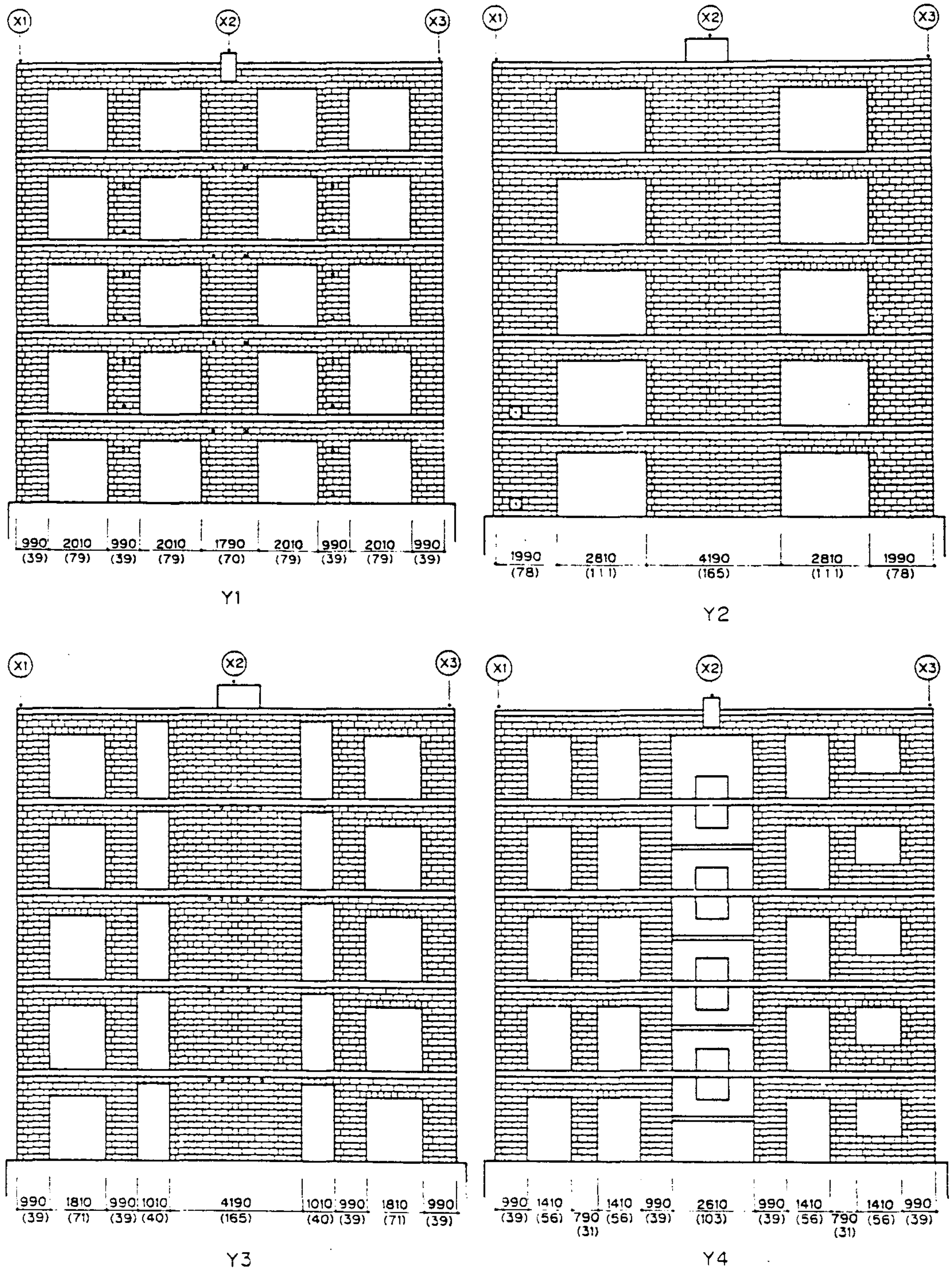

Fig. 3 Lateral Load Resisting Frames Y1 through Y4 [mm(in.)]

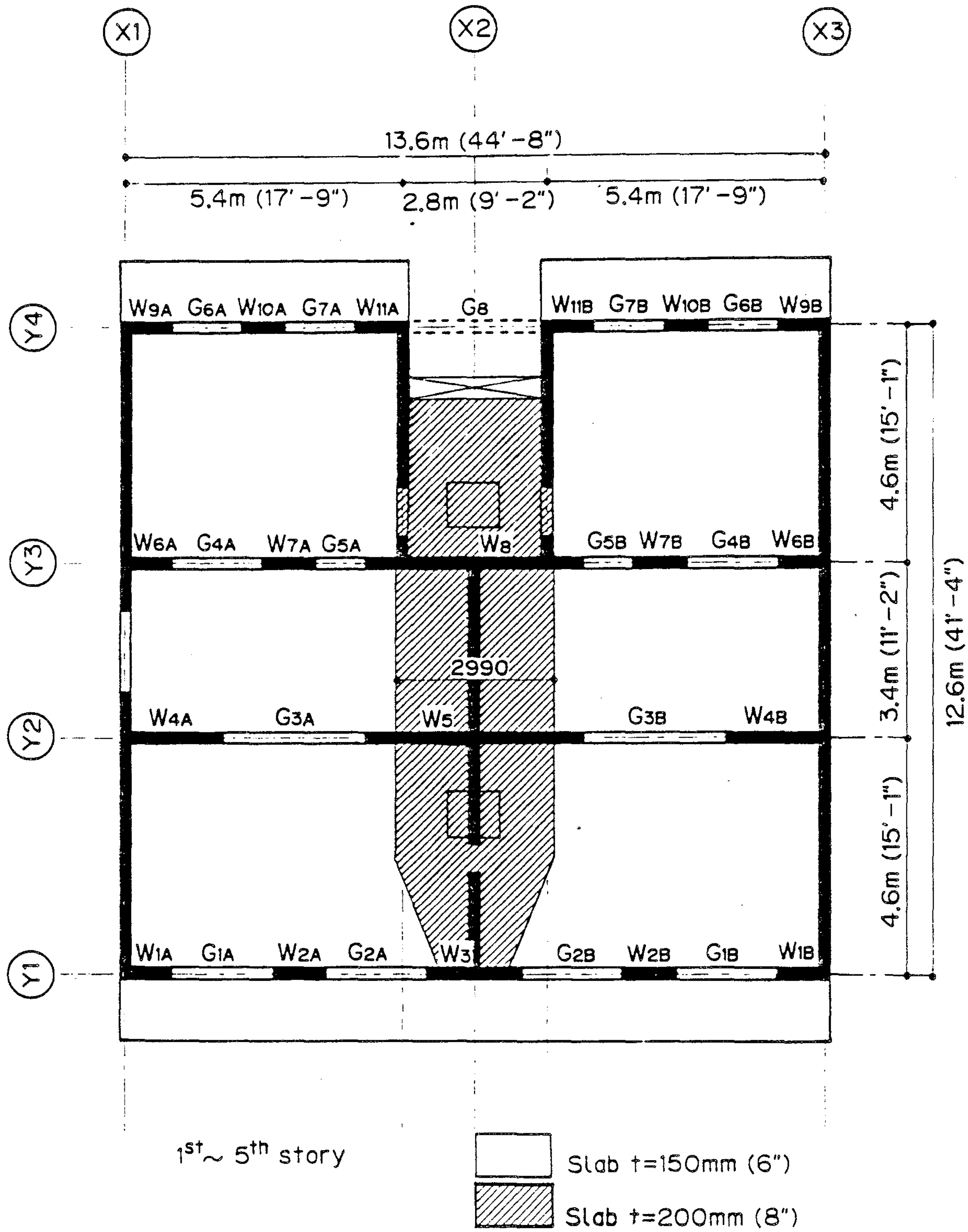

Fig. 4 Plan of Test Building and Member Designation

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area to total wall length in a given direction to ensure certain stiffness and with it certain dynamic response characteristics, limitations on size and shape of wall elements and openings, as well as detailed reinforcement specifications for individual elements, thus, effectively leading to non-engineered but specified structures.

The new design guidelines, while not completely free from general structural specifications, allow a morality-level rational engineering design approach based on the three fundamentals of structural specifications, allowable working stress design and ultimate strength design. Remainders from the structural specification concept are the height limit of the building to 16m(52'-9") or 5 stories, a somewhat relaxed total wall length to floor area ratio, and minimum and maximum reinforcement limits, as outlined in Appendix II.

Allowable stress design concepts are employed to ensure undamaged structural performance for small to moderate seismic excitation which can be expected several times per year. The unmodified standard lateral load coefficient on the weight of the structure for this service stress limit state is 0.2.

Ultimate limit state concepts are employed to prevent collapse of the building under the highest credible seismic excitation during the life of the structure. Global collapse mechanisms should basically be produced by local flexural hinge formation in the beam elements and by flexural plastic hinge formation at the base. For this ultimate limit state an unmodified base shear coefficient of 1.0 is assumed, multiplied by a structural performance coefficient of 0.5 for structures comprised of elements which are detailed to meet certain inelastic deformation criteria. In all other cases a factor of 0.6 is to be used for reinforced masonry structures compared to 0.25-0.5 for steel and 0.3-0.55 for reinforced concrete buildings, as specified in the Japanese seismic design code [3].

Rather than in terms of a hard to define "ductility", the deformation capacity is expressed in terms of a critical drift angle (lateral displacement/member length) of 1/100 up to which point no significant strength degradation is allowed. Based on extensive experimental component tests [4], this can be assured by limiting the nominal shear stress level at a local flexural mechanism to 1.8N/mm^2(260psi), by requiring the shear capacity to acting shear force ratio at the mechanism to be greater than 1.1 for beams and flanged walls and 1.2 for walls without transverse walls, and by providing special reinforcing details such as spiral reinforcement and/or joint ladder reinforcement in the expected plastic hinge regions, in order to confine the compression toe.

Details of the new proposed Japanese design code for medium-rise reinforced masonry structures can be obtained from [2].

Design of the Test Structure

Following the new Japanese design guidelines as outlined above, the 5-story full scale test specimen, Fig.1, was designed for a service limit state seismic base shear coefficient of 0.2 and an ultimate limit state seismic base shear coefficient of 0.5, with a lateral load distribution along the building height as specified by the Japanese Building Code and explained in [3]. The building weight includes a permanent live load portion (approximately 1/6 of the service design live load) for a typical floor load contribution of $5.10 kN/m^2$ (106psf) and a roof load of $5.29 kN/m^2$ (110psf). Including walls and other permanent fixtures the average gravity loading for the earthquake load case is $11.07 kN/m^2$ (231psf) for the first through fourth story level and $9.60 kN/m^2$ for the roof level, respectively, based on a plan area of $180.6 m^2$ ($1949 ft^2$), for one building module or two apartment units as shown in Fig.4. A summary on the determination of the lateral seismic design loads is given in Table 1.

A linear elastic frame model with rigid zones in the beam-column intersection regions was employed to determine member design loads, while member capacities were estimated from empirical formulas [2], developed originally for reinforced concrete members and verified for masonry members through the extensive materials and component Japan-TCCMAR program [4]. The effective width of flanged elements was taken as the smaller value of 1m (3.3ft) from the member face or 0.25 times the clear spacing between adjacent load resisting frames. The ultimate horizontal strength at each story level was obtained from a virtual work plastic hinge mechanism approach and the resulting frame and story capacities are shown in Table 2. An overview of the reinforcement for frames Y1-Y4 is given in Fig.5, and detailed lists of reinforcement for individual wall and beam elements are depicted in Tables 3 and 4, respectively.

Table 1 DETERMINATION OF LATERAL DESIGN LOADS

Level i	Weight W_i	$\sum_{i}^{R} W_i$	Shear Force Distribution Factors [3]	Shear Force Q_i	Lateral Loads P_i	Service Limit State $0.2 \times P_i$	Ultimate Limit State $0.5 \times P_i$
	MN (kips)	MN (kips)		MN (kips)	MN (kips)	MN (kips)	MN (kips)
R	1.73 (390)	1.73 (390)	1.70	2.95 (662)	2.95 (662)	0.59 (132)	1.47 (331)
4	2.01 (451)	3.74 (840)	1.39	5.19 (1,168)	2.25 (506)	0.45 (101)	1.12 (253)
3	2.01 (451)	5.74 (1,291)	1.23	7.06 (1,588)	1.87 (420)	0.37 (84)	0.94 (210)
2	2.01 (451)	7.75 (1,742)	1.11	8.60 (1,934)	1.54 (345)	0.31 (69)	0.77 (173)
1	2.01 (451)	9.76 (2,193)	1.00	9.76 (2,193)	1.15 (259)	0.23 (52)	0.58 (130)
Total	9.76 (2,193)	—	—	—	9.76 (2,193)	1.95 (438)	4.88 (1,097)

Table 2 CALCULATED LATERAL CAPACITIES in [MN (kips)]

Frame / Story	Y1	Y2	Y3	Y4	Total
5th	0.36 (82)	0.50 (113)	0.41 (93)	0.31 (69)	1.58 (356)
4th	0.64 (144)	0.88 (198)	0.34 (75)	0.54 (121)	2.79 (627)
3rd	0.87 (195)	1.20 (269)	0.99 (222)	0.73 (164)	3.78 (851)
2nd	1.06 (238)	1.45 (326)	1.20 (271)	0.90 (202)	4.61 (1,037)
1st	1.20 (271)	1.65 (371)	1.37 (307)	1.01 (225)	5.32 (1,175)

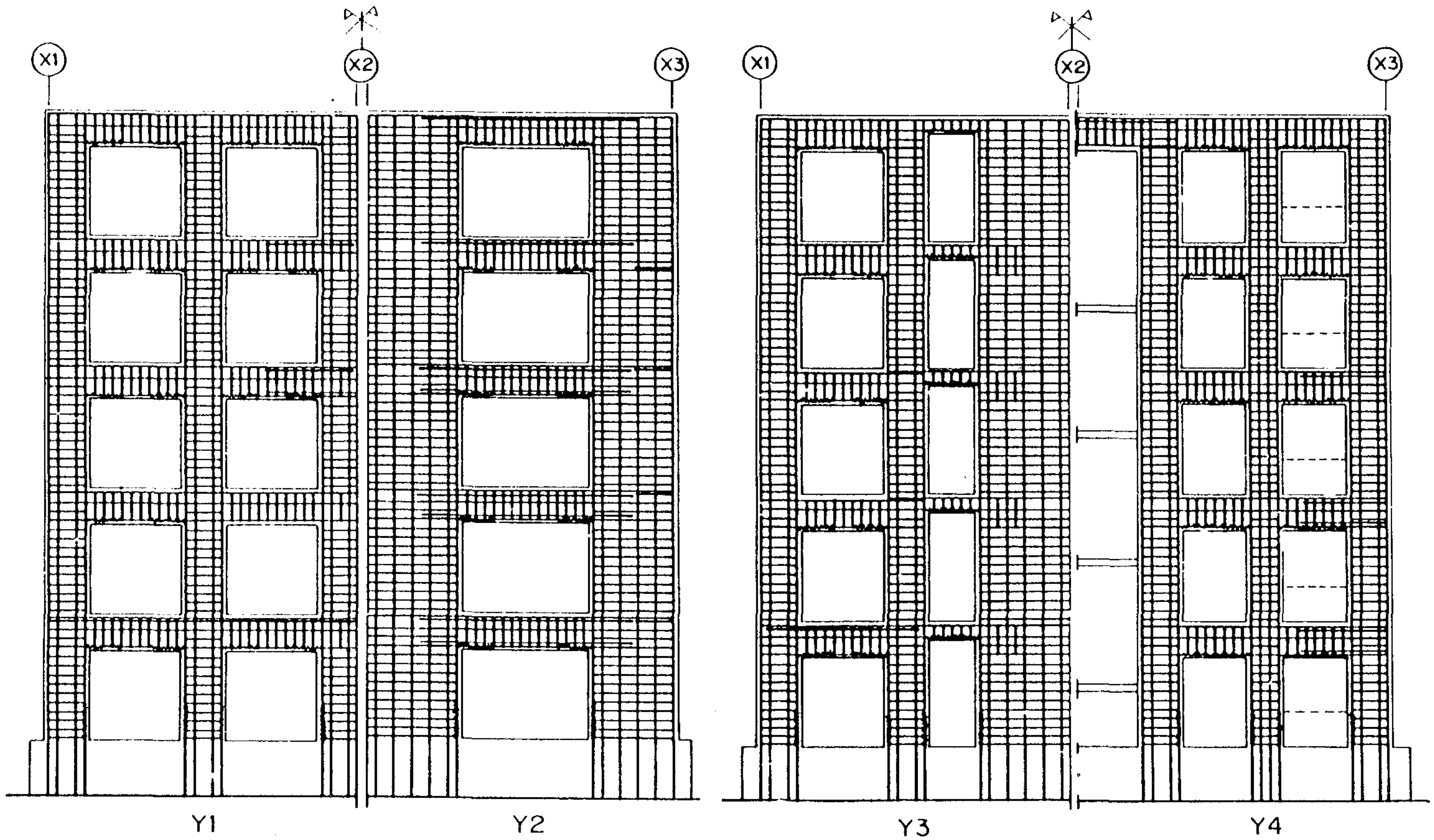
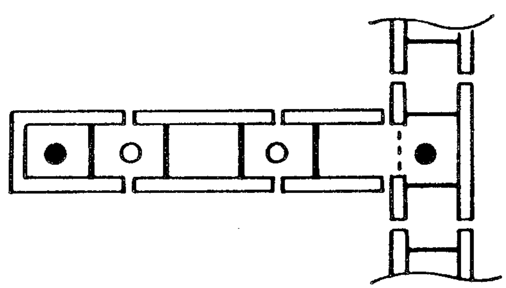


Fig. 5 Reinforcement Layout in Lateral Load Resisting Frames

Table 3 LIST OF WALL REINFORCEMENT

Story \ Wall		W1AB, W6AB W9AB, W11AB	W2AB W7AB	W3, W5 W8	W4AB	W10AB
5th	Main Flexural	D19, D16*	D16	D16	D19, D16*	D16
	Vertical	D16 @400	D16 @400	D16, D13 @400 alternately	D16, D13 @400 alternately	D16 @400
	Horizontal	D10 @200	D10 @200	D10 @200	D10 @200	D10 @200
4th	Main Flexural	D19, D16*	D16	D16	D19, D16*	D19
	Vertical	D16 @400	D16 @400	D16, D13 @400 alternately	D16, D13 @400 alternately	D16 @400
	Horizontal	D10 @200	D10 @200	D10 @200	D10 @200	D10 @200
3rd	Main Flexural	D19, D16*	D16	D16	D19, D16*	D19
	Vertical	D16 @400	D16 @400	D16 @400	D16 @400	D16 @400
	Horizontal	D13, D10 @200 alternately	D13, D10 @200 alternately	D13, D10 @200 alternately	D13, D10 @200 alternately	D13, D10 @200 alternately
2nd 1st	Main Flexural	D19, D16*	D19	D16	D19, D16*	D19
	Vertical	D16 @400	D16 @400	D16 @400	D16 @400	D16 @400
	Horizontal	D13, D10 @200 alternately	D13, D10 @200 alternately	D13, D10 @200 alternately	D13, D10 @200 alternately	D13, D10 @200 alternately

* Flanged end

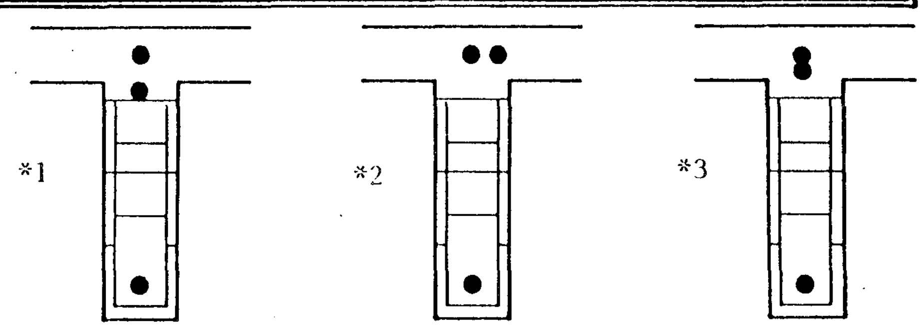


● Main Flexural Reinforcement
○ Vertical Reinforcement

Table 4 LIST OF BEAM FLEXURAL REINFORCEMENT

Story	Beam	G1	G2 W2 side	G2 W3 side	G3	G4	G5	G6 W9 side	G6 W10 side	G7
5th	T	D19	D19		2-D19*3 (2-D19*2)	D19	D19	D19		D19
	B	D19	D19		D22	D16	D16	D19		D19
4th	T	D19	D19	D19,D16*1 (D22)	2-D22*1 (2-D22*2)	D19	D19	D19		D19
	B	D19	D19		D25	D16	D16	D19		D19
3rd	T	D22	D22	D22,D16*2 (D22,D16*1)	D25,D22*1 (D25,D22*2)	D22	D19	D16, D22*1	D22	D22
	B	D19	D19	D16,D19*1	2-D19*1	D19	D16	D19		D19
2nd	T	D25	D25		D25,D22*2 (D25,D22*1)	D22	D19	2-D19*1 (D22, D16*1)	(D22)	2-D19*3 (D22)
	B	D22	D22		D22,D19*1	D19	D16	D16, D22*1	D22	D22
1st	T	2-D19 (D25)	2-D19 (D25)		D25,D22*2	D16,D19*3 (2-D19*1)	D19	2-D19*3 (D16, D22*1)	(D22)	2-D19*3
	B	D22	D22		D19,D22*1	D19	D16	D16, D22*1	D22	D22

T:TOP B:Bottom
( ):Dwelling unit B



All reinforcement, except spiral and joint ladder reinforcement, consisted of deformed bars with nominal $350N/mm^2$ yield strength for bars Ø16 (#5) and larger, and $300N/mm^2$ (grade 40) for bars Ø13 (#4) and smaller. The denomination in Tables 3 and 4 is given by diameter in [mm], e.g. D19 is a deformed bar with 19mm nominal diameter(#6 rebar). The vertical reinforcement in wall elements is divided into end or main flexural reinforcement which is arranged in the extreme cells of the wall element and standard vertical reinforcement arranged in interior cells. In addition to the member reinforcement indicated in Tables 3 and 4, spirals were provided around the bottom flexural reinforcement in all beams, where flexural yield hinges are expected. Spirals of length L=800mm (31.5"), which corresponds to 40Ø for a D19 (#6) bar, consisted of S4 (0.16") undeformed reinforcement with 100mm (4") inside diameter and 40mm (1.6") pitch. Only one beam, namely G2A in the second story (2G2A), was not provided with spiral reinforcement, in order to determine behavior differences. Spirals were also provided at the base of all walls at the ground floor level around the splices of the main flexural reinforcement (extreme bars) and starter bars except for the flanged wall ends. In addition, joint reinforcement consisting of S4 (0.16") diameter bars was placed in walls 1W2A and 1W5 and in the lower parts of beam 1G2A. This joint reinforcement was ladder shaped for the walls and U-shaped for the beam.

All elements satisfy the inelastic deformation capacity criteria as outlined above, except for beam 4G3 and walls 2W10 and 1W10 which have calculated shear capacity to flexural hinge shear force ratios at ultimate of 1.0, 1.1, and 1.0 respectively, and thus have to be considered members of limited inelastic deformation capacity. However, still a deformation capacity factor of 0.5 was applied to the lateral ultimate limit state design forces as proposed in the new Japanese design guidelines for structures comprised of ductile members.

Transverse walls X1-X3 were typically reinforced with D13@400mm (#4@16") vertically and D10@200mm (#3@8") horizontally with one D19 (#6) bar horizontally at every floor level. In the 1st-3rd story, however, every alternate horizontal reinforcing bar was changed from D10 to D13 (#3 to #4) to increase the horizontal reinforcement ratio from 0.18% to 0.25%. Flexural floor reinforcement consisted typically of D10 (#3) in both directions, top and bottom.

The short beams G5A and G5B were reduced in depth to two courses or 39cm (15.35"), in order to prevent shear failure at low load levels. Spandrel walls with various connection details to the adjacent wall elements W9B and W10B were arranged in frame Y4 to study the respective design details. Steel frames and doors were installed in frame Y3 between walls W7A and W8 to study their performance at various deformation levels. Additional design details to be investigated during the full scale test are the top flexural rebar arrangement in the bond beam-R/C slab assemblages, see Table 4, and the effect of small openings in structural components arranged at various locations throughout the test building.

A more detailed summary on the design of the 5-story test building can be found in [5].

CONSTRUCTION OF THE TEST BUILDING

General Construction Principles

In an effort to advance the technology in fully grouted reinforced masonry construction a new modular bonding system [6] was developed at the Building Research

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Institute (B.R.I.) in Tsukuba City, Japan, which utilizes open ended H-shaped standard units and two types of special end units as shown in Fig.6. The development of the B.R.I. concrete block system was based on the following general principles: (1)usage of open end units to avoid web-to-web joints and with it problems of water penetration, (2)modular coordination in wall spacing, (3)modular coordination in size of walls and openings, (4)modular coordination in reinforcing bar spacing, (5)running bond in all parts of the wall including wall-to-wall intersections, (6)minimization of number of special units except for on-site cut units, and (7)sufficient space for reinforcement and grouting at wall intersections.

The modular system utilized in the construction of the 5-story test building and based on the above principles has nominal modular standard unit dimensions of 2T=400mm (16") in length, T=200mm (8") in width and T=200mm (8") in height, including a nominal mortar joint thickness of 10mm (0.4"). The actual unit dimensions and examples of horizontal bonding patterns produced with these units and utilized in the test building are given in Fig.7. The important modular dimensions for the reinforcement layout are also T=200mm (8") both, vertical and horizontal.

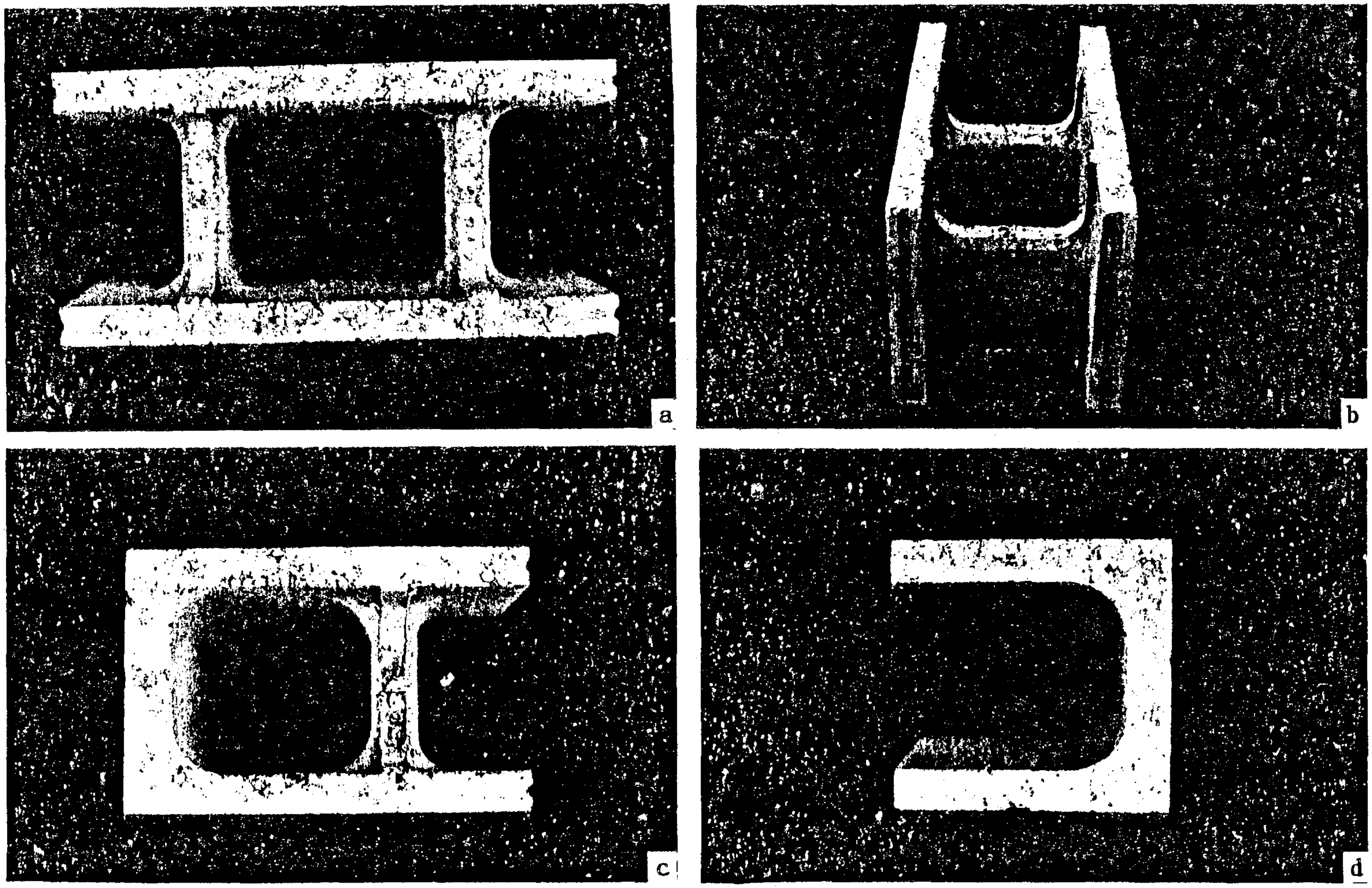

Fig. 6 Concrete Block Units a,b) Standard Openended Unit, c,d) End Units

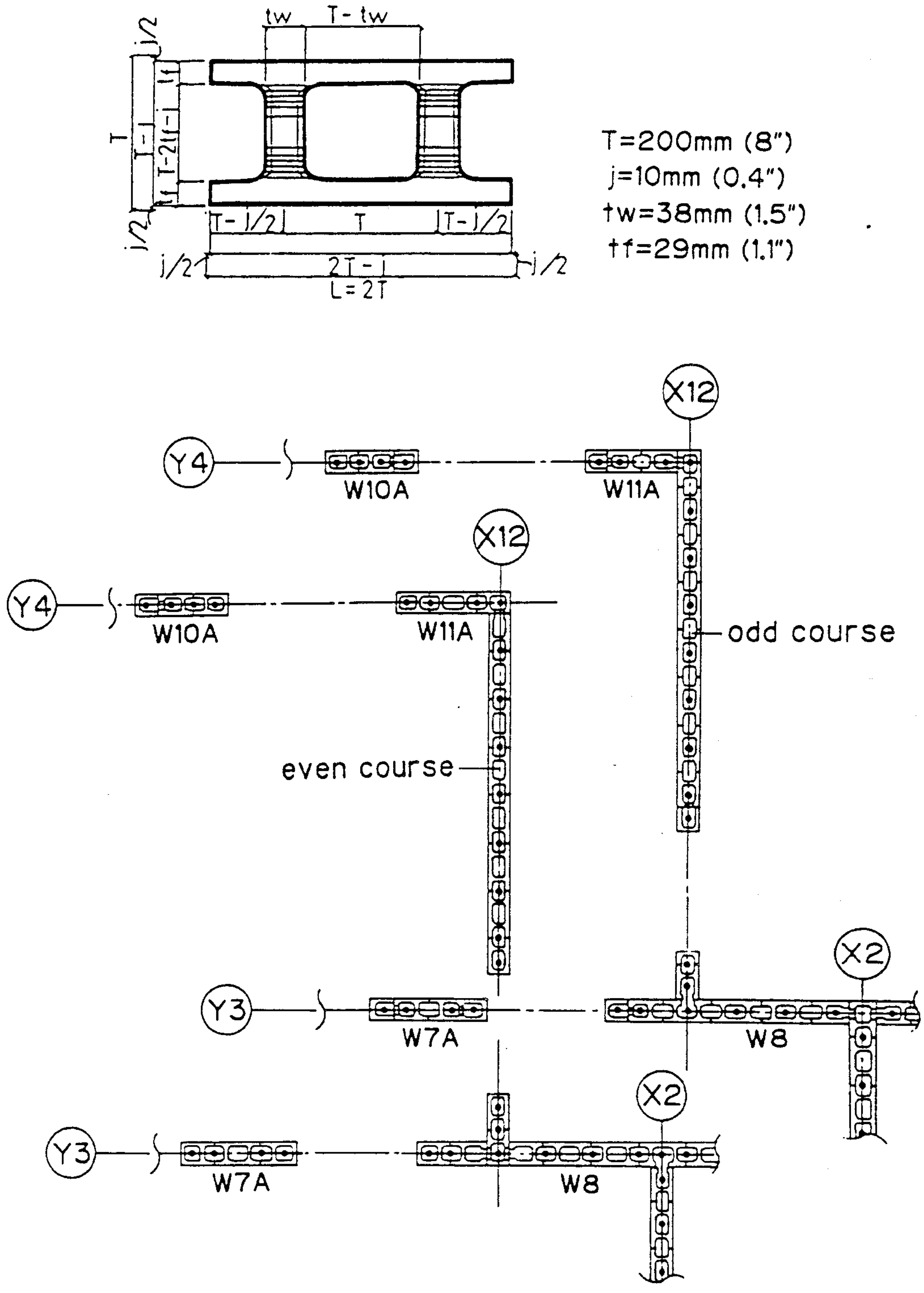

Fig. 7 Dimensions of Standard Unit and Typical Bonding and Reinforcing Patterns

Construction of the Test Specimen

The starter bars for the lateral load resisting masonry frames were anchored in a 1.2m (46") deep and 0.9m (35") wide reinforced concrete foundation beam grillage which was tied to the box girder test floor by means of 310 high strength 32mm (1-1/4") diameter post-tensioning bars, prestressed with 400kN (88kips) each.

Construction of the masonry walls commenced by placement of the story high vertical main flexural reinforcement prior to unit placement, in order to protect rebar strain gages and to allow the installation of spiral reinforcement around the 40d (d=diameter of reinforcing bar) lap joints in the critical hinge zones of the first story walls. Details of the construction process are depicted in Figs.8 and 9. Starter bars , lap joint, and spiral reinforcement can be seen in Fig.8a. Examples of L and T-shaped wall-to-wall intersections are shown in Figs.8c and 9c, respectively, and joint ladder reinforcement, as shown in Fig.9d, was provided in all horizontal joints in walls 1W2A and 1W5.

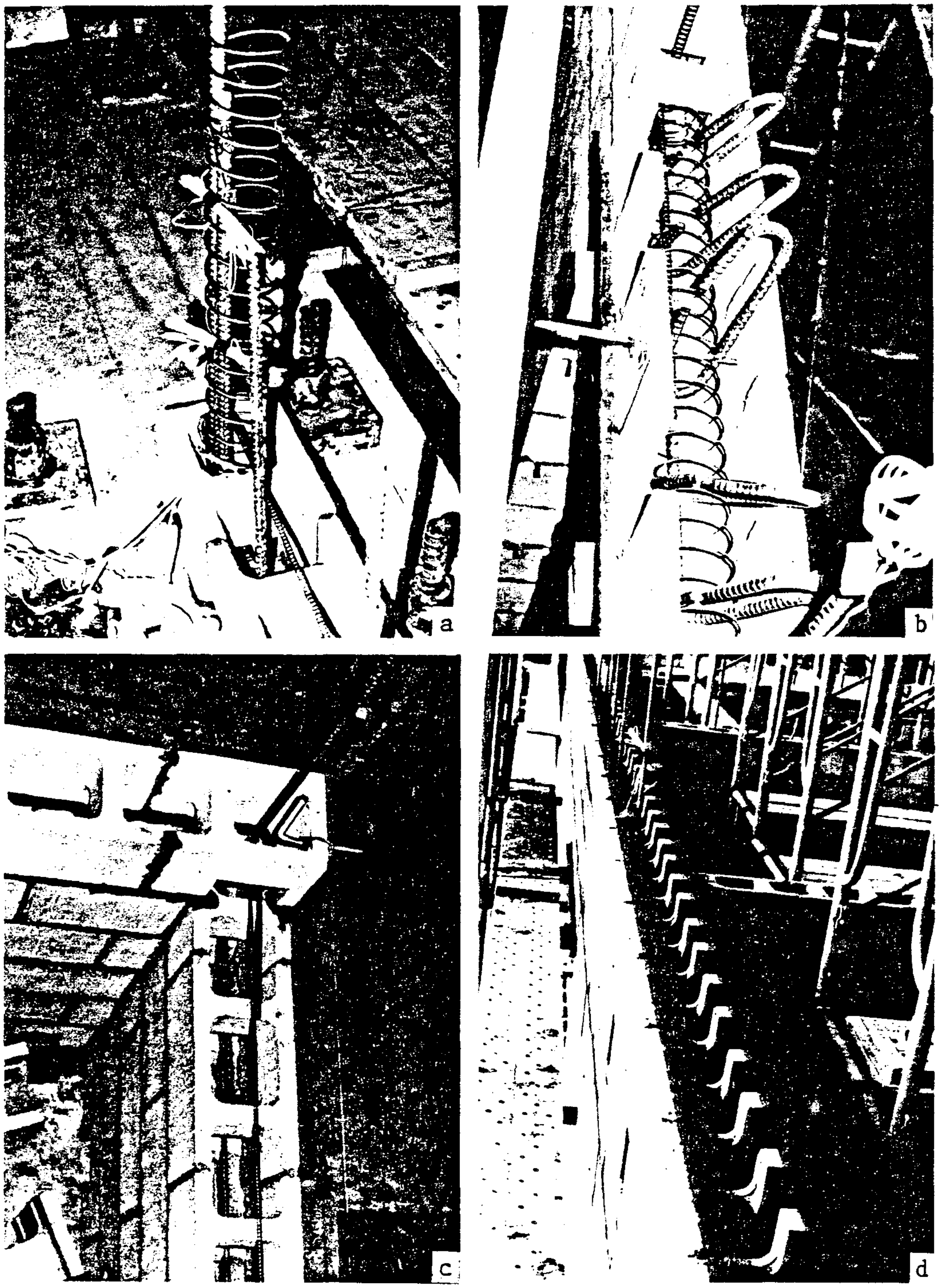

Fig. 8 Construction Details

a) Spirals in Walls b) Spirals in Bond Beams

c) L-Intersection d) Floor Level Course

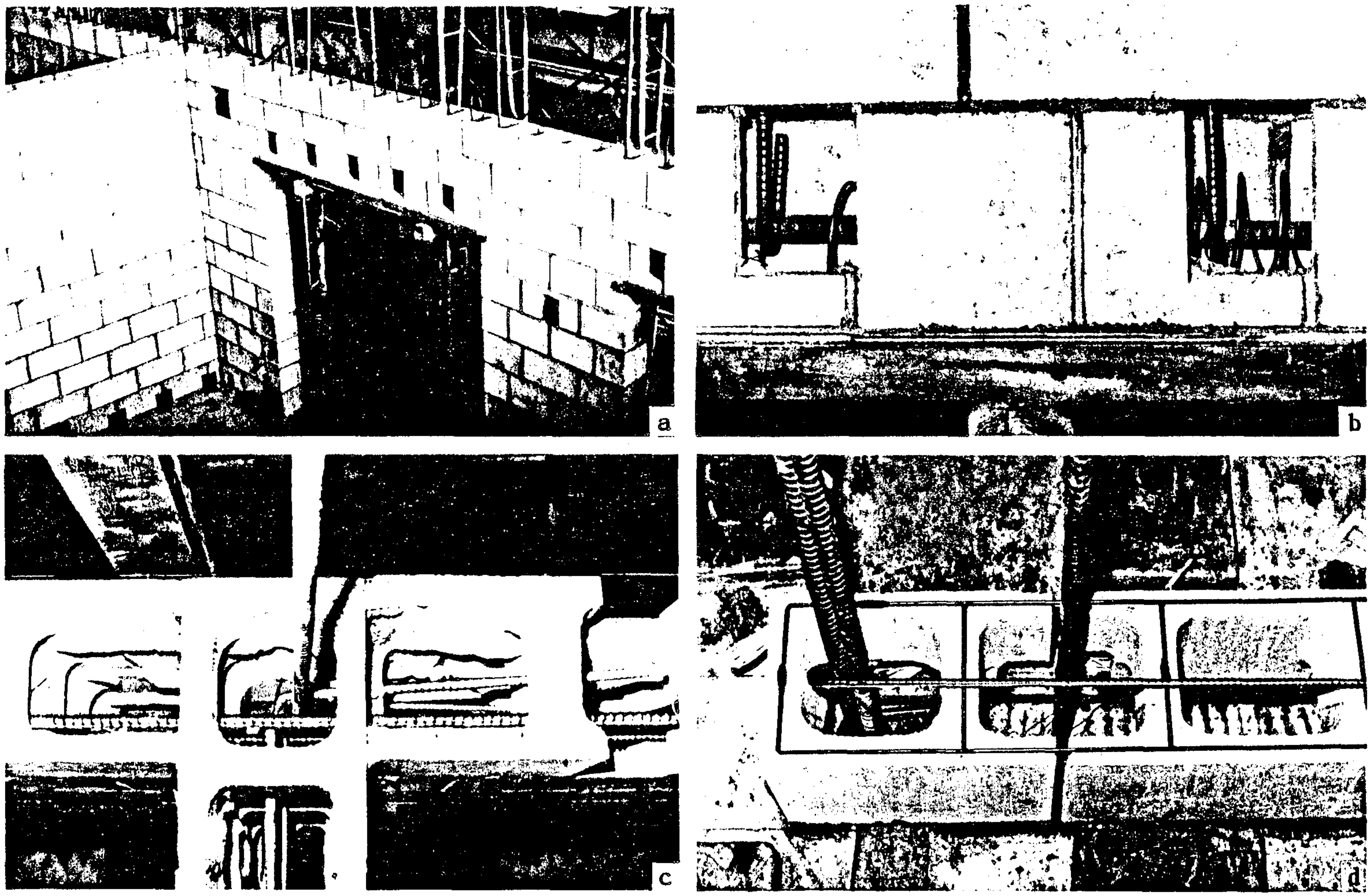

Fig. 9 Construction Details a)Clean Outs b)Bond Beam Detail c)T-Intersection d)Ladder Joint Reinforcement

Shear keys, 25mm deep x 80mm wide @ 400mm (1"x3"@16"), perpendicular to the wall axis were formed at the top of the foundation beams and at every floor level in the reinforced concrete slab to prevent slippage between walls and slabs at high load levels. Inspection holes (clean-outs) were provided at the foot of the walls and the bottom of the bond beams in order to clean out construction debris and joint mortar residue prior to grouting, and to inspect the proper placement of the reinforcement. These clean-outs, as shown in Fig.9a, were arranged every 2T=400mm (16") or where vertical rebar was located. The bond beam spiral reinforcement, vertical joint reinforcement, and clean-outs are depicted in Figs.8a and 9a,b.

Prior to grouting of the walls, approximately 25mm (1") of high slump mortar was poured into the cells to minimize segregation effects of coarse aggregate at the slab interface during the placement of the grout concrete. Grouting commenced in two lifts of 1/2 the story height to allow partial setting of the first grout layer prior to subsequent grouting. The design strength of the grout concrete was 24N/mm^2 (3400psi), the maximum aggregate size was 20mm (3/4"), and 4% of air entrainment was used to obtain slump levels of approximately 210mm (8.3"). Compaction of grout was achieved by internal vibrating of each cell. Grouting was stopped 100mm (4") below the top of each wall to allow slab concrete to penetrate into the walls. Masonry units, where part of the inside face shell was cut out, were used at the floor levels, see Fig.8d, to establish additional bond between the reinforced concrete floor slab and the walls. The completed test specimen is depicted in Fig.1.

Material Properties and Quality Control

After construction of the test specimen, ultrasonic grout inspections in the vicinity of the spiral reinforcement in the first story walls revealed grout flaws in 6 of 28 inspection points. These grout flaws indicate potential problems with grout placement due to the reinforcement congestion in the end cells, caused by the spiral reinforcement. The detected problem areas were subsequently injected with cement grout.

Twenty seven cylinders, 100mm Ø and 200mm high (4"x8"), of grout concrete for standard material tests and twelve three course, stack bonded, grouted masonry prisms were produced for each story; all test specimens were air cured in the laboratory next to the test building. Compressive tests of the concrete cylinders were carried out 7 and 28 days after the grouting operation, and the prisms were tested in compression after 28 days. Additional materials tests half way through the full scale test program were performed to determine the actual properties at the time of testing. The 28 day tests, as well as test results obtained during the static load testing of the research building, are summarized by story level in Table 5. While the grout for each story was designed to reach the nominal design strength at the beginning of the static load test, the strength distribution in Table 5 indicates a slightly reduced strength level in the first story, not only as expected and designed for at the 28 day mark, but also in the middle of the static load test. The concrete mix design for the floor slabs directly followed the grout mix design of the corresponding story.

Compressive strength tests of joint mortar specimens, 40x40x160mm (1.5x1.5x6"), after 28 days are also summarized in Table 5. The mechanical properties of the utilized reinforcing bars, obtained experimentally from 6 specimens for rebar diameter, showed an actual yield stress level of 380N/mm^2 (54ksi) independent of the rebar size.

Table 5 COMPRESSION TEST RESULTS [N/mm^2(psi)]

Test	Concrete Grout		Masonry Prisms			Mortar
Time	28days	testing	28days	testing	testing*	28days
Number of Specimens	6/story	**	6/story	**	3/story	13/story
5th story	25.7 (3,725)		20.6 (2,989)		—	40.8 (5,916)
4th	25.0 (3,619)		18.7 (2,705)		—	45.7 (6,627)
3rd	24.7 (3,583)		15.7 (2,282)		—	36.3 (5,264)
2nd	23.6 (3,427)		17.1 (2,473)		—	36.5 (5,293)
1st	20.3 (2,936)		15.4 (2,227)		17.5 (2,538)	41.2 (5,974)
Mean	23.9 (3,459)		17.5 (2,538)		17.5 (2,538)	40.1 (5,814)

* cut from test building at the beginning of the test program
** will be supplied as soon as available

CONCLUSIONS

The Japanese 5-story full scale reinforced concrete masonry test building was designed based on the proposed draft of the new Japanese design guidelines for medium rise (up to 5 stories) masonry structures. The important features of this limit state design approach is the tri-level design concept of specified dimensional and reinforcement limits, the service stress limit for structural damage mitigation at moderate earthquake levels, and the ultimate limit state to prevent structural collapse under the most severe seismic conditions.

An important feature at the ultimate limit state level is the deformation capacity design concept which allows a reduction of the required lateral load capacity if structural components are designed and detailed to allow drift angles of 1/100rad without significant loss of lateral load capacity.

One of the proposed detailing methods to ensure the necessary rotation capacity in the plastic hinge region is the placement of spiral reinforcement to increase the ultimate compressive strain limit and to prevent premature buckling of the compression reinforcement by limited confinement of the surrounding concrete. However, ultrasonic quality control checks on the grout density in the spiral region showed potential problems with grouting of these regions due to the reinforcement congestion. To minimize these potential problem areas, the latest draft of the new design guidelines limits the provision of spiral reinforcement in walls to hinge regions which feature nominal axial compressive stress levels of more than 10% of the prism design strength.

The test structure also features a newly developed bonding system which is based on a H-shaped standard open ended unit and two special end units which allow a consistent running bond pattern with a uniform module for walls, openings, and reinforcement.

The instrumentation and loading of the Japanese 5-story full scale reinforced masonry test building as well as the structural response and analytical correlation studies are presented in separate papers.

APPENDIX I

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APPENDIX II

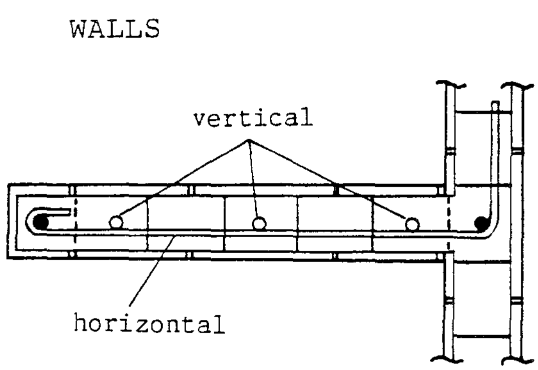
REINFORCEMENT SPECIFICATIONS FOR RM-COMPONENTS [2]

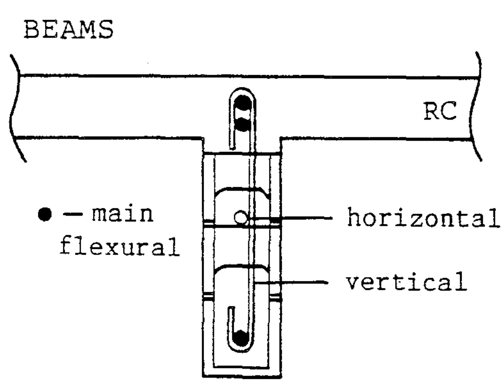
II.1 Wall Element Reinforcement Requirements

type	reinforcement		
	main flexural	vertical	horizontal
1^{st}–3^{rd} story	≥1-D16(#5) and ≤2-D25(#8)	≥0.2% or 1-D13(#4)	≥0.25% or 1-D13(#4)
4^{th}+5^{th} story			≥0.2% or 1-D10(#3)
spacing @	—	≤unit length or 400mm (16")	≤unit height or 200mm(8")

II.2 Beam Element Reinforcement Requirements

type	reinforcement		
	main flexural	horizontal	vertical
size	D16 ~ D25 (#5 ~ #8)	D10 ~ D16 (#3 ~ #5)	D10 ~ D16 (#3 ~ #5)
amount top or bottom	≥D16(#5) and ≤2-D25(#8)	≥0.25%	≥0.25% (≥0.3% for short beams $\ell/h \leq 1.5$)
spacing @	—	≤400mm(16")	≤200mm(8")





APPENDIX V

THE JAPANESE 5-STORY FULL SCALE REINFORCED CONCRETE MASONRY TEST - Loading and Instrumentation of the Test Building

by Frieder Seible, Yutaka Yamazaki, Takashi Kaminosono, and Masaomi Teshigawara

KEYWORDS: Data Acquisition, Full Scale Test, Instrumentation, Load History, Reinforced Concrete Masonry, Servo Controlled Testing, Test Plan

ABSTRACT: The overall test plan for the 5-story reinforced concrete masonry research building investigated at the reaction wall facility of the Building Research Institute (B.R.I.) in Tsukuba City, Japan, is presented. The three main phases of the test program, the static (cyclic) loading tests, the forced vibration tests, and the pseudo dynamic test are summarized together with the loading history and test sequence. The servo controlled hydraulic loading system is described and the individual actuator control modes for the various test phases are discussed. A detailed assessment of the external reference instrumentation and the individual component instrumentation is presented, together with an outline of the data acquisition and monitoring systems, in order to assist in the interpretation of the test data.

THE JAPANESE 5-STORY FULL SCALE REINFORCED CONCRETE MASONRY TEST - Loading and Instrumentation of the Test Building

by

Frieder Seible[1], Yutaka Yamazaki[2], Takashi Kaminosono[3], and Masaomi Teshigawara[4]

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INTRODUCTION

The U.S.-Japan Joint TCCMAR (Technical Coordinating Committee on Masonry Research) plan calls for a concluding full scale test of a 5-story prototype masonry structure in each country. While the research buildings will be specific to the individual technology and design requirements, the synthesis process of predicting the prototype response based on extensive materials, components, and sub-assemblage tests, as well as multi-level analytical modeling efforts is mutual to both countries and forms the basis for the joint research program. An independent evaluation of the test data of the Japanese 5-story full scale research building requires detailed knowledge of the instrumentation, the computer controlled loading system, and the load history assumptions in order to correlate research findings with the expected structural behavior of masonry buildings under seismic loads.

The methodology behind the Japanese 5-story building test is based on the objective to obtain experimental verification of limit state models described in the draft of the new Japanese design guidelines for medium rise masonry structures at service load limit states,

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the yield limit state, and at the ultimate load and deformation limit states of buildings under corresponding equivalent seismic loads as defined by the Japanese Building Code and described in [1]. Therefore, in the main portion of the full scale reaction wall test, the test structure is subjected to increasing cyclic lateral load and deformation levels, with the lateral load distribution derived from [1], in order to obtain the test structure response under this prescribed design load distribution.

The dynamic response characteristics of the test structure are determined for each of the outlined limit states by means of forced vibration tests to allow checks on analytical correlation studies.

An attempt to study the response of a masonry structure to a specific ground motion time history is made by means of a pseudo dynamic test [2] on the research building. Due to well documented difficulties with this type of simulated seismic testing for stiff structural systems, this test is conducted during the inelastic response phase of the test building after substantial damage accumulation and associated stiffness degradation.

The overall test plan, the servo controlled loading system, and the instrumentation of the test structure are described in this paper. The design and construction of the Japanese full scale reinforced masonry test building are summarized in [3], and test results as well as analytical correlation studies will be presented separately.

TEST PLAN AND LOAD HISTORY

Overall Test Plan

The load history of the 5-story full scale reinforced masonry test building was designed to provide behavioral information at the code limit states under the corresponding code load distribution, to provide basic dynamic characteristics for state determination of the test structure, and to provide information on the structural response to a specific ground motion time history. In addition, following the ultimate deformation limit state, various integrated structural details, such as floor to wall connections, are being investigated to obtain realistic prototype data, and to establish a basis for comparison with separate component tests.

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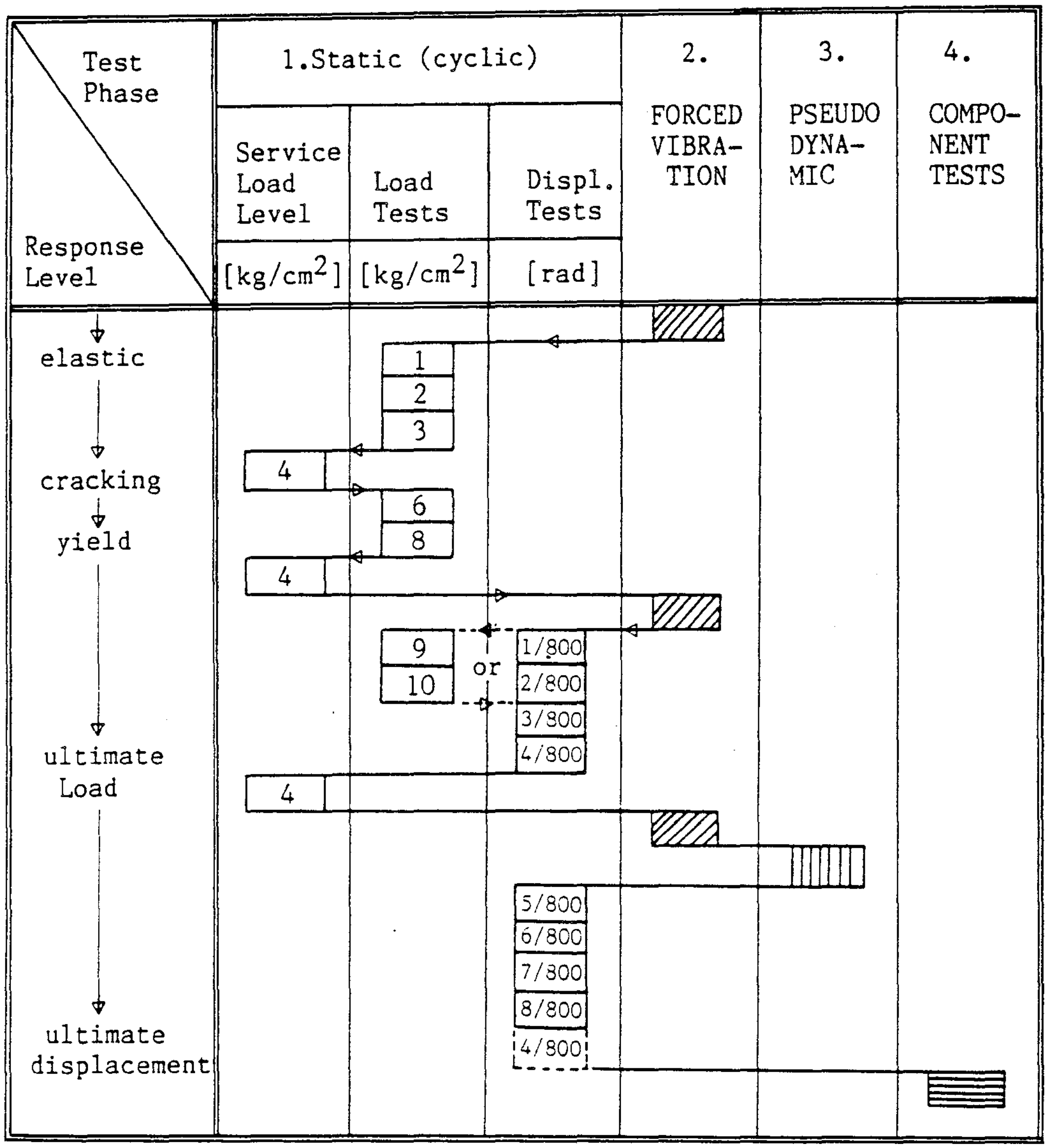
The overall test plan, as described in [4], consists of the following four phases: 1)the static loading test, 2)the forced vibration test, 3)the pseudo dynamic test, and 4)the integrated structural component tests. The overall test sequence is summarized in Table 1.

During the static load testing, increasing cyclic lateral loads following the Japanese Building Code distribution [1] are applied to the test structure by means of servo controlled hydraulic actuators, as schematically shown in Fig.1. The loading history follows the pattern depicted in Fig.2, with the first phase of cyclic load levels controlled by nominal base shear stress levels of 0.1,0.2,0.3,0.4,0.6, and 0.8 N/mm^2 (14,28,43,57,85,and 114psi), respectively. The $0.4N/mm^2$ (57psi) level represents approximately one of the design limit states described in [3], namely the service limit state with a corresponding base shear coefficient of 0.2. Two cycles at this level of service limit stress are repeated at the onset of the yield limit state and the ultimate load limit state of the structure, as shown in Fig.2, to simulate moderate seismic loading subsequent to various levels of damage accumulation. In addition to these service load level cycles after each of the outlined limit states, low

Table 1 TEST AND LOADING SEQUENCE



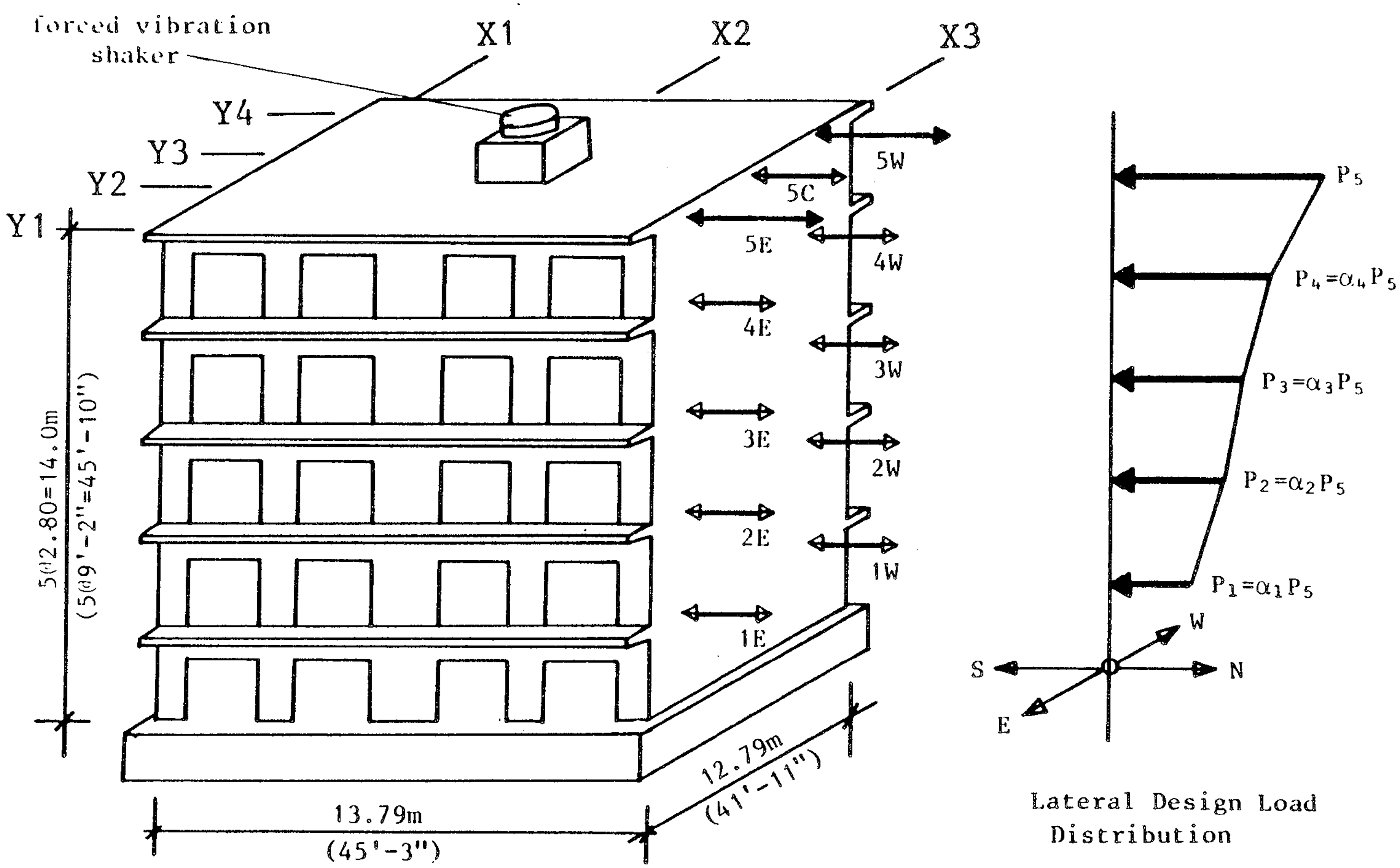


Fig. 1 Loading Sketch of Full Scale RM Building

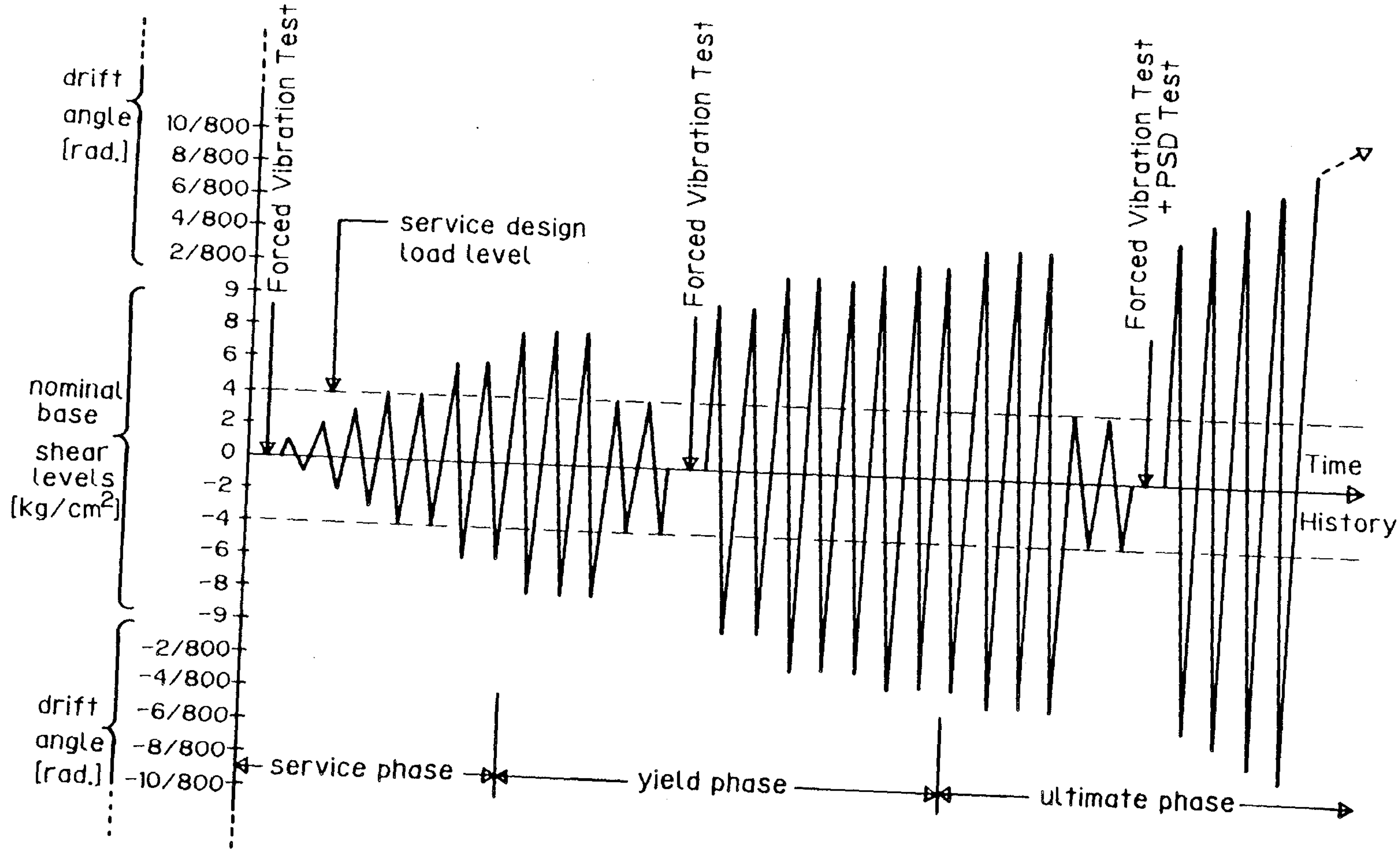


Fig. 2 Test Plan and Load History

level individual floor loads are applied in order to establish flexibility coefficients and thus a global stiffness matrix for the lateral floor degrees of freedom.

In the inelastic deformation phase of the structure, the load cycles, while still following the code load distribution, are generally determined by total building drift angle (horizontal roof displacement/height of the structure), rather than by nominal base shear stress levels. However, initial inelastic response tests may still be controlled by nominal base shear stress levels of 0.9 and 1.0N/mm^2 (129 and 142psi) until a drift angle of 1/800 is reached; subsequently only the drift angle will determine the cycle apex. This allows a detailed investigation of the deformation capacities even at decreasing structural stiffness levels. This static load sequence provides information on the building response up to service load levels, a trace of the resistance mechanism for increasing lateral force and deformation levels including maximum lateral load capacity, strength degradation and deformation capacity characteristics, as well as a performance assessment of special design details such as small openings in walls, spandrel wall design, flexural rebar arrangement in beams and service performance of doors at various deformation levels.

The main objective for the forced vibration tests is a state determination of the test structure at various design limit states by establishing the dynamic response characteristics. Actual stiffness data of the test structure at different damage accumulation levels is important for the calibration of analytical correlation models, and natural frequency and damping information is used in the assessment of subsequent seismic response of the structure. A 10ton,1-15Hz oscillator permanently installed at the roof level of the test structure and 9 horizontal and 5 vertical acceleration pickups, as shown in Fig.3, are used to measure the dynamic response of the test structure including torsional modes and base rocking through frequency sweep steady state vibration and rundown tests. Component characteristics of floor slabs are also determined through drop weight tests in selected areas at the third floor level.

The PSD (pseudo dynamic) test phase has two objectives, namely to check the application of the B.R.I.(Building Research Institute) PSD loading system to relatively

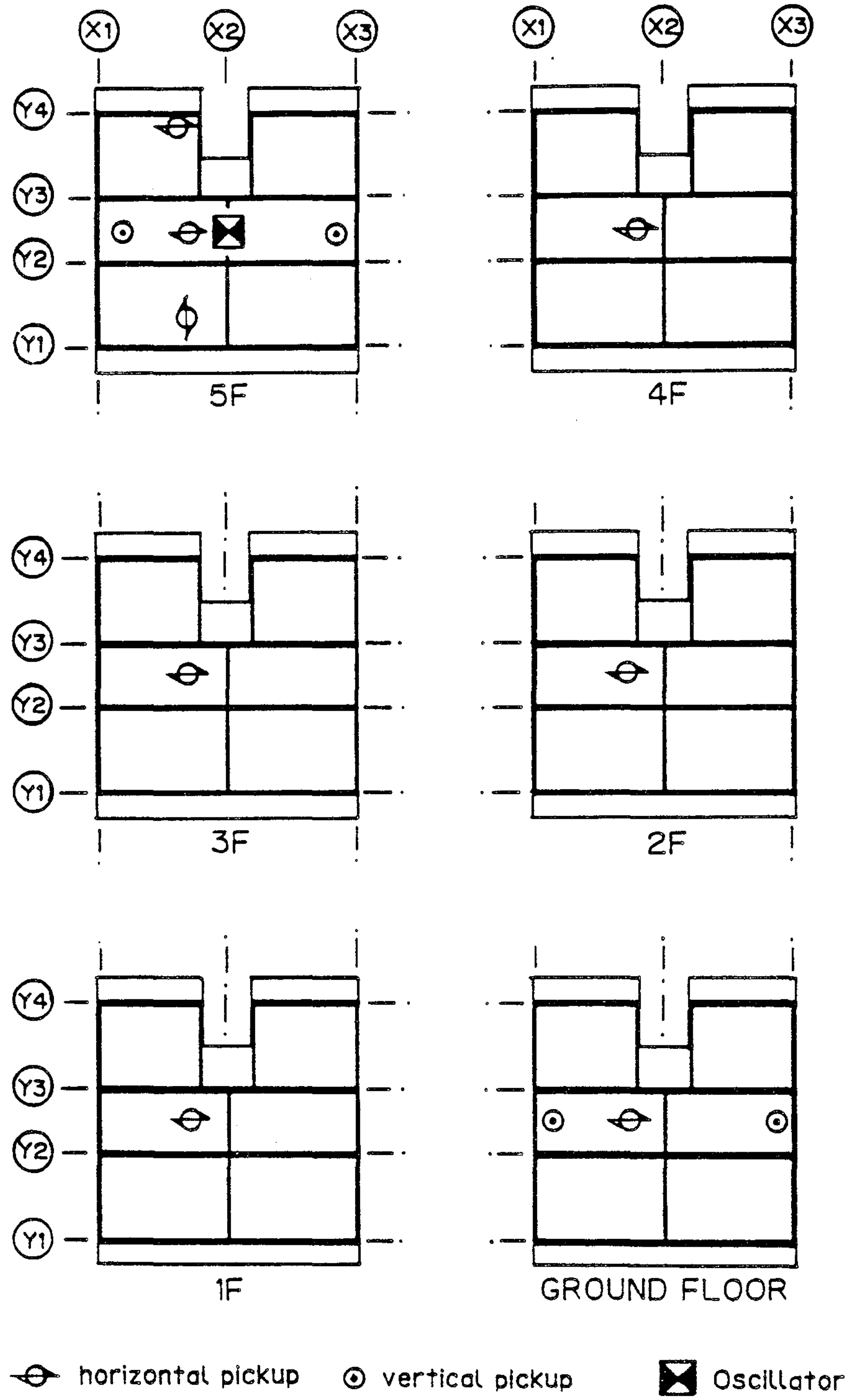


Fig. 3 Forced Vibration Test Setup

rigid structures such as this reinforced masonry shear wall structure, and to determine the structural response behavior during a subsequent earthquake or aftershock, following significant damage accumulation during the main event. The pseudo dynamic test follows the ultimate load limit state and precedes a series of large deformation cycles to establish the strength degradation characteristics, see Table 1 and Fig.2.

Upon conclusion of the static (cyclic) load testing, the full scale test structure is utilized to determine integrated structural component behavior, such as out-of-plane floor slab stiffness with various support conditions, torsional stiffness of floor slab and edge beam assemblies, and out-of-plane wall stiffness.

Loading and Control Loop

The 5-story full scale test structure is loaded by 11 one hundred ton (220kip) capacity actuators, 3 at the roof level and 2 at the first through fourth floor level, as schematically shown in Fig.1. All actuators have a stroke of ±500mm (20") except for the two exterior roof level actuators 5E and 5W which have stroke ranges of ±1000mm (40"). Each actuator is attached at one end to the reaction wall by means of a

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100mm (4") thick base plate which allows horizontal and vertical actuator positioning based on a 100mm (4") bolt hole pattern. At the other end the actuators are connected to a structural steel load beam assembly, as shown in Fig.4, which transfers the loads to 0.9x1.0x1.6m (35x39x63") reinforced concrete load blocks connected monolithically to the 200mm (8") thick and 2.99m (9'-10") wide reinforced concrete floor loading strip along the centerline of the building. Actuator connection and placement are depicted in Fig.5.

In order to preserve the structural integrity of the concentrated load application system [5] after the development of cracks in the floor slab, a high strength thread bar system connecting the load blocks with exterior load distribution beams, as shown in Fig.4, is provided. The sixteen 32mm (1-1/4") diameter high strength bars at each floor level were only hand tight and not post-tensioned, to limit the introduction of unrealistic axial stress levels in the horizontal beam elements.

The servo valves for the 11 hydraulic actuators are controlled by a MX-3000 Super Mini Computer which receives feedback control information from the load

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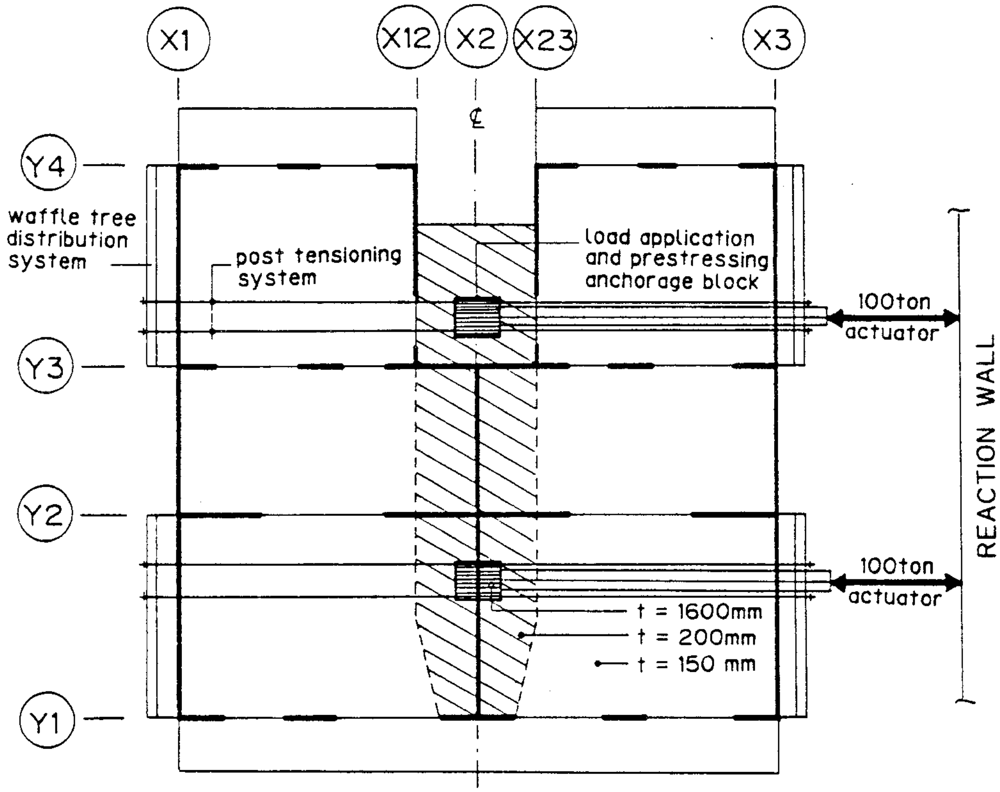
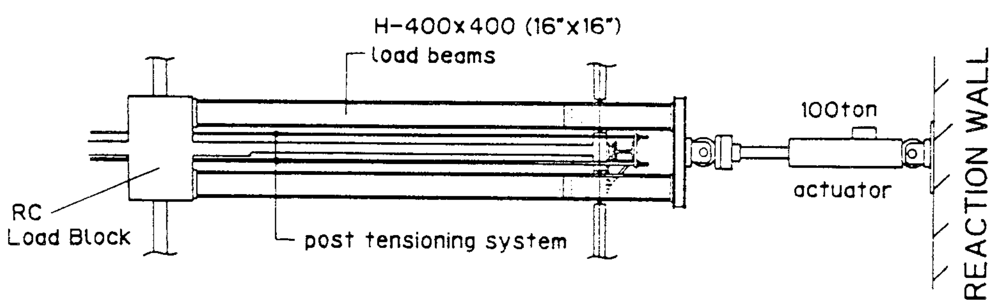


a) plan view of typical floor slab



b) vertical section through load assembly

Fig. 4 Load Application System

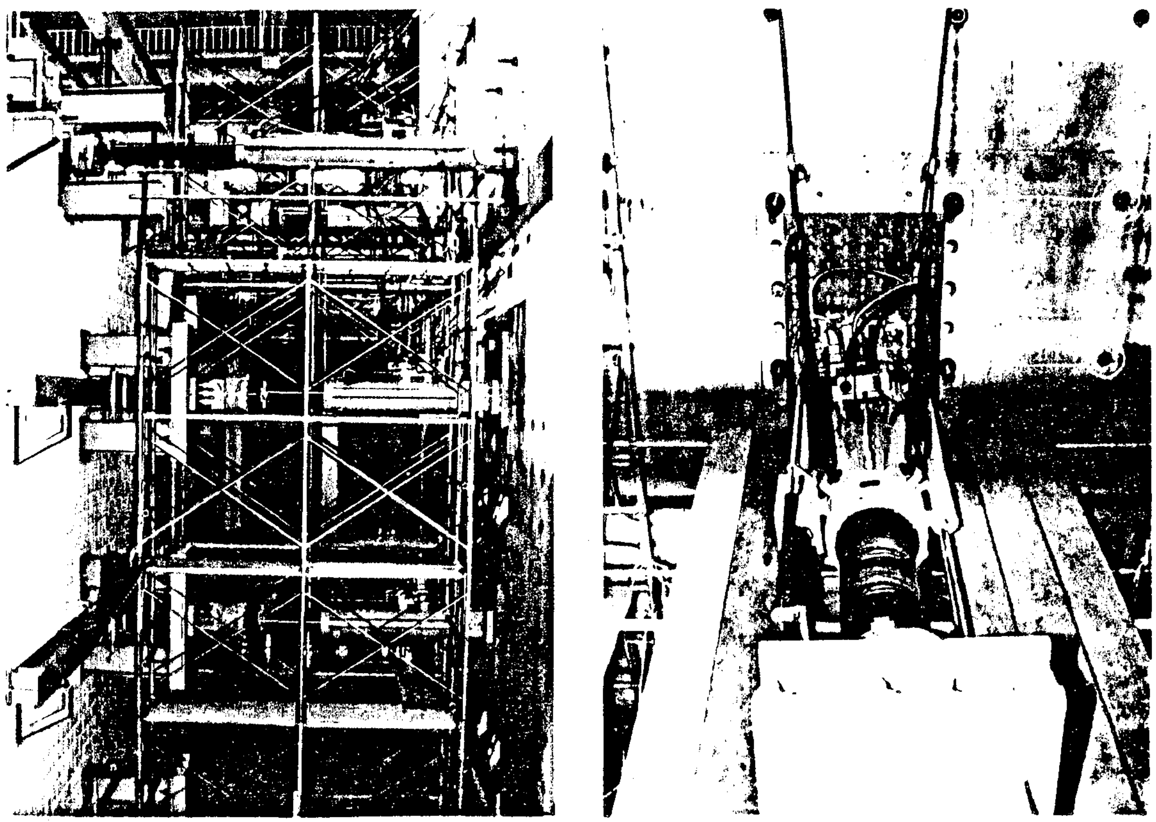


Fig. 5 Actuator Connection and Loading System

cells incorporated into the actuator assembly and from external magnescales which measure the displacements of each floor level with high accuracy (see Section on instrumentation). The magnescales are positioned, one per floor level, between a stiff external reference frame and the geometric center of the test building plan. Only at the roof level, two additional magnescales are installed to measure also the displacements of the two exterior load application points. Each actuator can be computer adjusted in a force or displacement control mode.

For the static (cyclic) load testing, the two exterior actuators at the roof level, see Fig.1, are operated under displacement control to prevent the introduction of a torsional deformation mode, and all other actuators are force slaved to the total measured roof load following the lateral load distribution for the actual test building as specified in the Japanese Building Code. The total roof load contribution factors for the individual floor loads, as indicated in Fig.1, are 0.40, 0.49, 0.59, and 0.72 for floor levels 1 through 4, respectively.

In the pseudo dynamic test, the actuators are basically displacement controlled from the magnescale readings.

However, at floor levels 1 to 4, where only one magnescale measures each center displacement of the floor slab, only one of the two floor actuators is directly displacement controlled from the magnescale with the other actuator in a force slaved mode with respect to the displacement controlled actuator at this floor level. A detailed summary of individual actuator control for the different test phases is presented in Table 2.

The super mini computer system allows an update of the servo control loop every 0.1sec with control limit checks on preselected displacement and force tolerances. Displacement increments during the initial loading phase of the undamaged building are 0.012mm (0.0005") and systems shut off limits are set at 300N (66kips) or 20mm (0.8") for individual actuators per load step. An automatic shut off of an individual servo valve is set for a 5% error between the command and feedback for the corresponding actuator. These load control tolerances can be reset as the structural system softens through damage accumulation. In addition to the on-line computer control, visual control of the magnescale displacement measurements is provided by digital readouts on the test floor and through a remote

Table 2 ACTUATOR CONTROL OVERVIEW

Location	Actuator P		Actuator Control			
			Static Load Test		Pseudo Dynamic Test	
			mode	level	mode	level
5th floor (roof)	East	5E	Displ.	active	Displ.	active
	Center	5C	Load	$(P_{5E}+P_{5W})/2$	Load	$(P_{5E}+P_{5W})/2$
	West	5W	Displ.	active	Displ.	active
4th floor	East	4E	Load	$\alpha_4 \cdot P_5/2$	Displ.	active
	West	4W	Load	$\alpha_4 \cdot P_5/2$	Load	P_{4E}
3rd floor	East	3E	Load	$\alpha_3 \cdot P_5/2$	Displ.	active
	West	3W	Load	$\alpha_3 \cdot P_5/2$	Load	P_{3E}
2nd floor	East	2E	Load	$\alpha_2 \cdot P_5/2$	Displ.	active
	West	2W	Load	$\alpha_2 \cdot P_5/2$	Load	P_{2E}
1st floor	East	1E	Load	$\alpha_1 \cdot P_5/2$	Displ.	active
	West	1W	Load	$\alpha_1 \cdot P_5/2$	Load	P_{1E}

α_i= determined by Japanese Building Code
active= predetermined computer input
$P_5 = P_{5E} + P_{5C} + P_{5W}$

controlled video camera in the control room.

Data Acquisition

The data acquisition is performed through 19 switch boxes with 50 channels each for a total of 950 channels and a data logger with 1kHz sampling rate. The data is transmitted from the test floor to the super mini computer in the control room via an optical fiber cable.

Connected to the super mini computer is a micro computer based graphical data monitoring system comprised of a PC (personal computer) master which controls 4 independent PC slaves. The PC master can display 6 simultaneous load-deformation and 7 time-history graphs derived from the magnescale and load cell information, and each PC slave can display 24 x-y type data sets in groups of 4 autoscaled screen plots. A summary of the data acquisition and monitoring system is depicted in Fig.6.

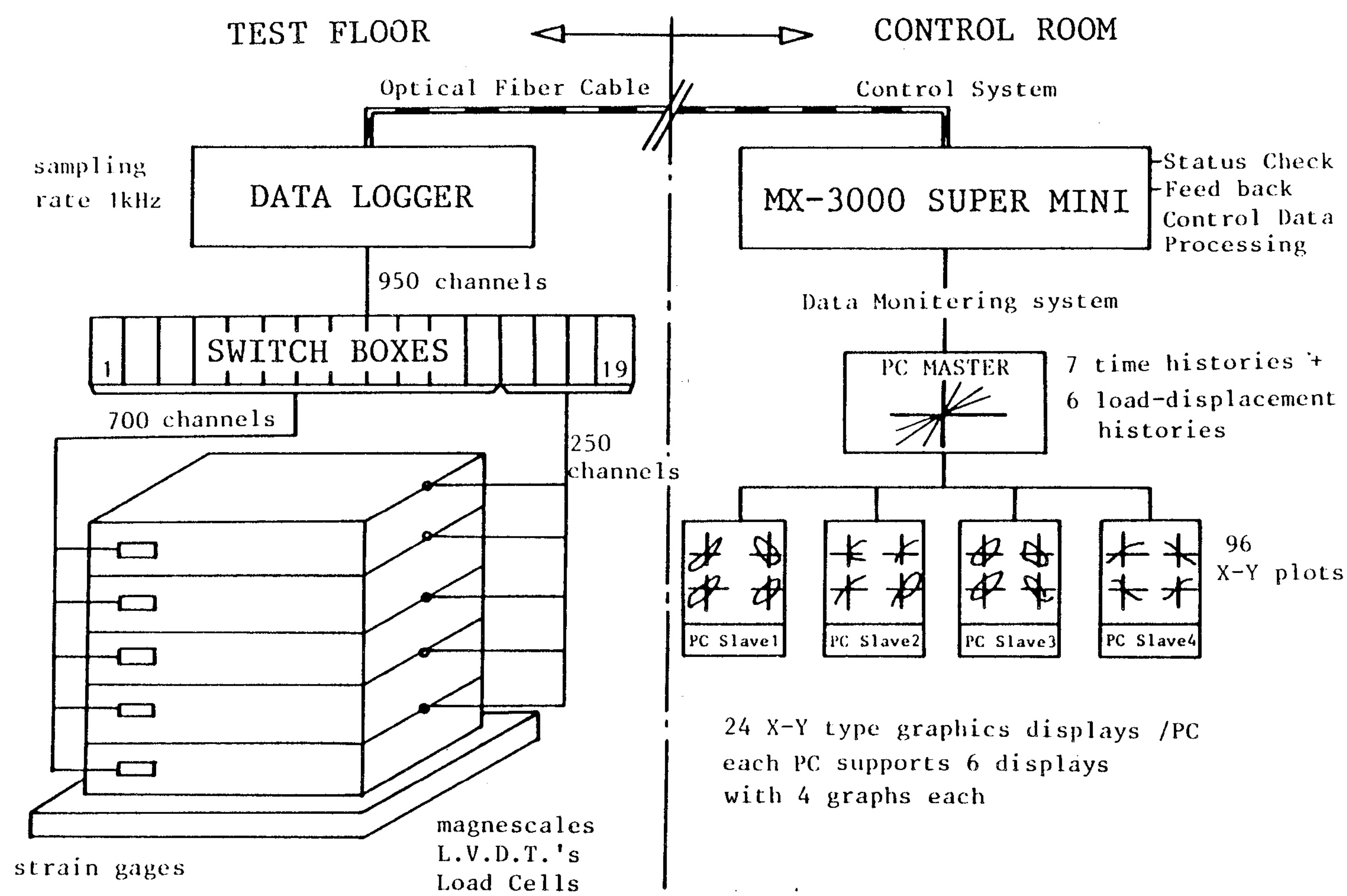


Fig. 6 Data Acquisition and Monitoring System

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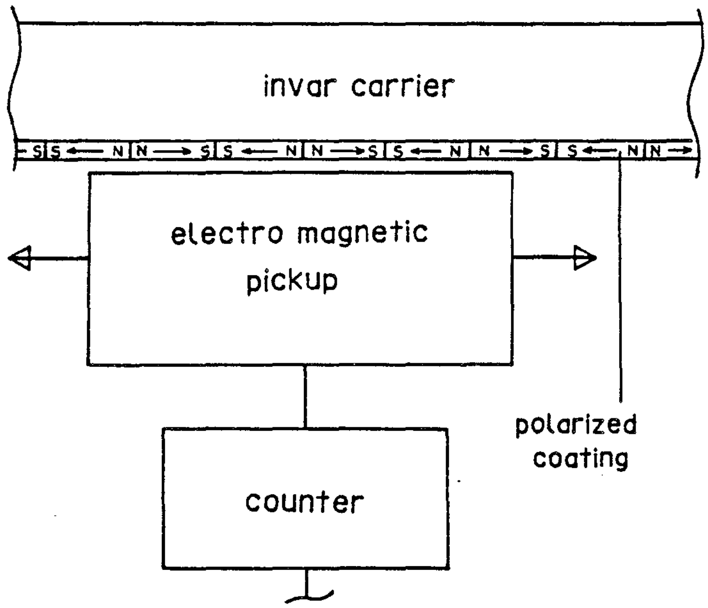
INSTRUMENTATION OF THE TEST SPECIMEN

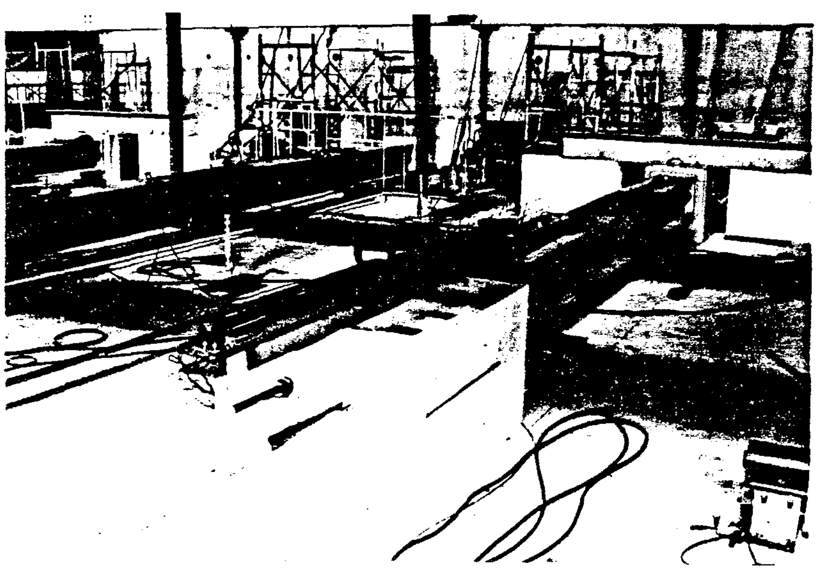
Externally Referenced Instrumentation

The global state of the test structure is monitored by a series of reference gages which measure the overall structural response relative to external (independent) reference points. This reference instrumentation is comprised of three instrument types, namely load cells, magnescales and LVDT's (linear variable displacement transducers).

Load cells and magnescales are used, as described above, for the interactive actuator control. The load cells, which are an integral part of the actuator assembly, measure the reactive force between the actuator and the reaction wall with a resolution of 0.2% or 2kN (0.4kip). Magnescales with ±1000mm (40") range for the two exterior roof level positions and ±500mm (20") at the geometric center of each floor level are connected with lightweight aluminum tubing to stiff external reference frames. The magnescale is a digital displacement transducer (DDT), see Fig.7, with a resolution of about 0.01mm (0.004in.) for the employed ranges. The position of the 11 load cells corresponds to the actuator positioning depicted in



Magnescale Principle



2000mm (6.5ft) Magnescale

Fig. 7 Digital Displacement Transducer

Fig.1, and the 7 magnescales are located as shown in Fig.8.

An assortment of analog displacement transducers, most with 100 and 200mm (4 and 8") range, are installed between the test structure and external reference frames in order to measure the global deformation response of the test building. In addition to capturing the horizontal deformation mode of the test building under the applied lateral loads, instruments are also positioned, as indicated in Fig.8, to monitor transverse displacements, vertical displacements and torsional deformation modes.

Structural Component Instrumentation

In order to correlate the full scale reinforced masonry building test data with analytical and design models, as well as with the component test data obtained from the comprehensive Japanese TCCMAR component test program [6], the individual structural components such as walls, beams, and intersection elements have to be instrumented. Due to the large number of individual elements comprising the test structure, the limited number of instrumented structural components was determined based on the expected deformation limit

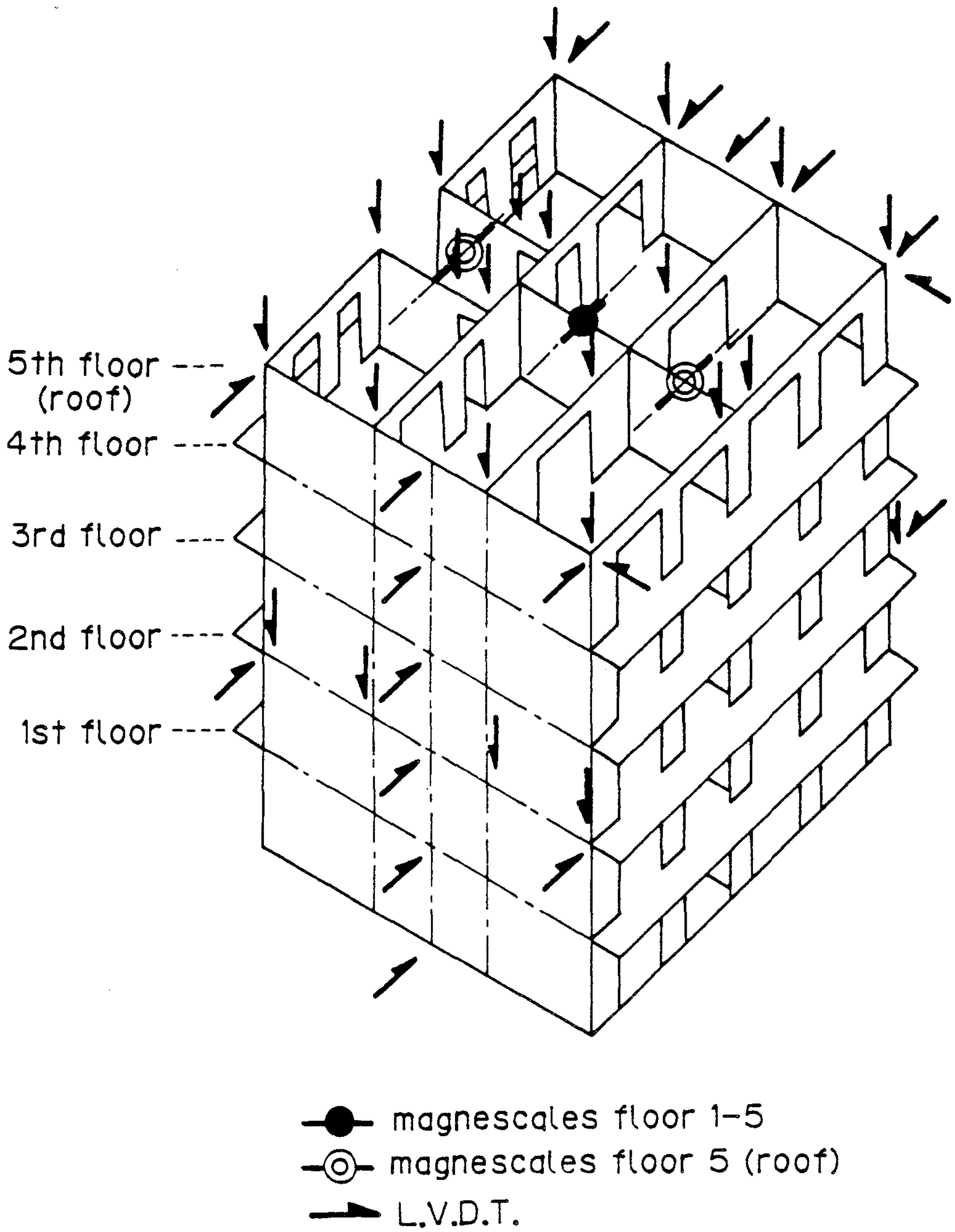


Fig. 8 Location of External Displacement Transducers

states of each individual lateral load resisting frame, as indicated in Fig.9. In order to capture these assumed deformation states, displacement transducers are arranged as shown in Fig.9, with instrumented components indicated by solid lines. Flexural (F), diagonal (D), and rotational (R) instrumentation in Fig.9 refers to LVDT's arranged in series along the extreme fibers of the flexural element, diagonally between the corner points of the element, and perpendicular to the member axis, respectively, as depicted in Fig.10. The LVDT's for the component deformation measurements have typically ranges of 10 to 100mm(0.4 to 4").

The strain state and the onset of yield mechanisms in individual components can be monitored by electrical resistance gages of 5mm (0.2") gage length glued to selected rebars at critical member sections. These strain gages are primarily located on the main flexural component reinforcement, namely the rebar located in the extreme cells of wall elements and at the top and bottom of beam elements. Spiral hoop strains in beams and first story walls are also selectively monitored. A total of 600 rebar strain gages were provided and examples of the arrangements for the critical first two

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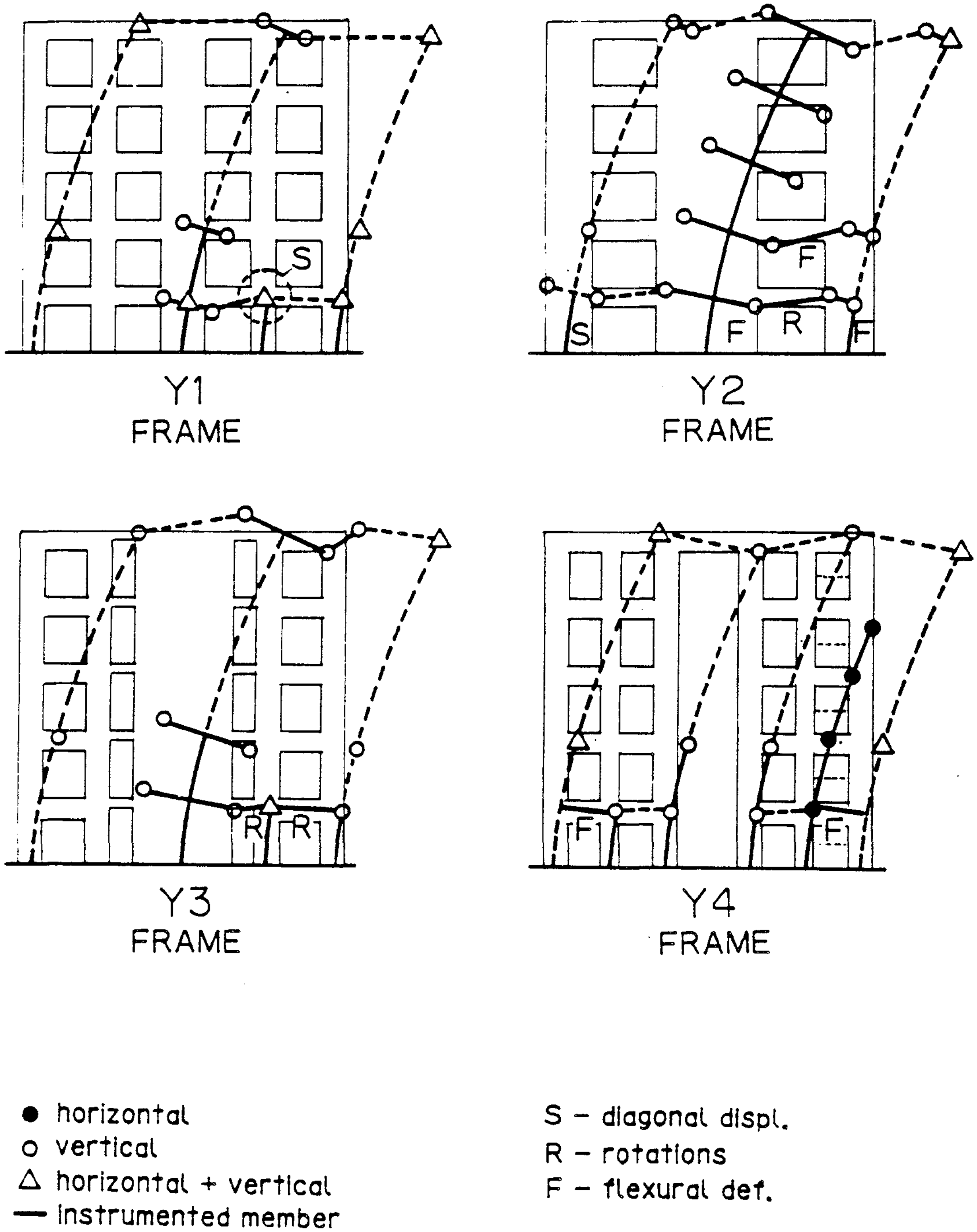

Fig. 9 Expected Deformation Patterns and Displacement Instrumentation

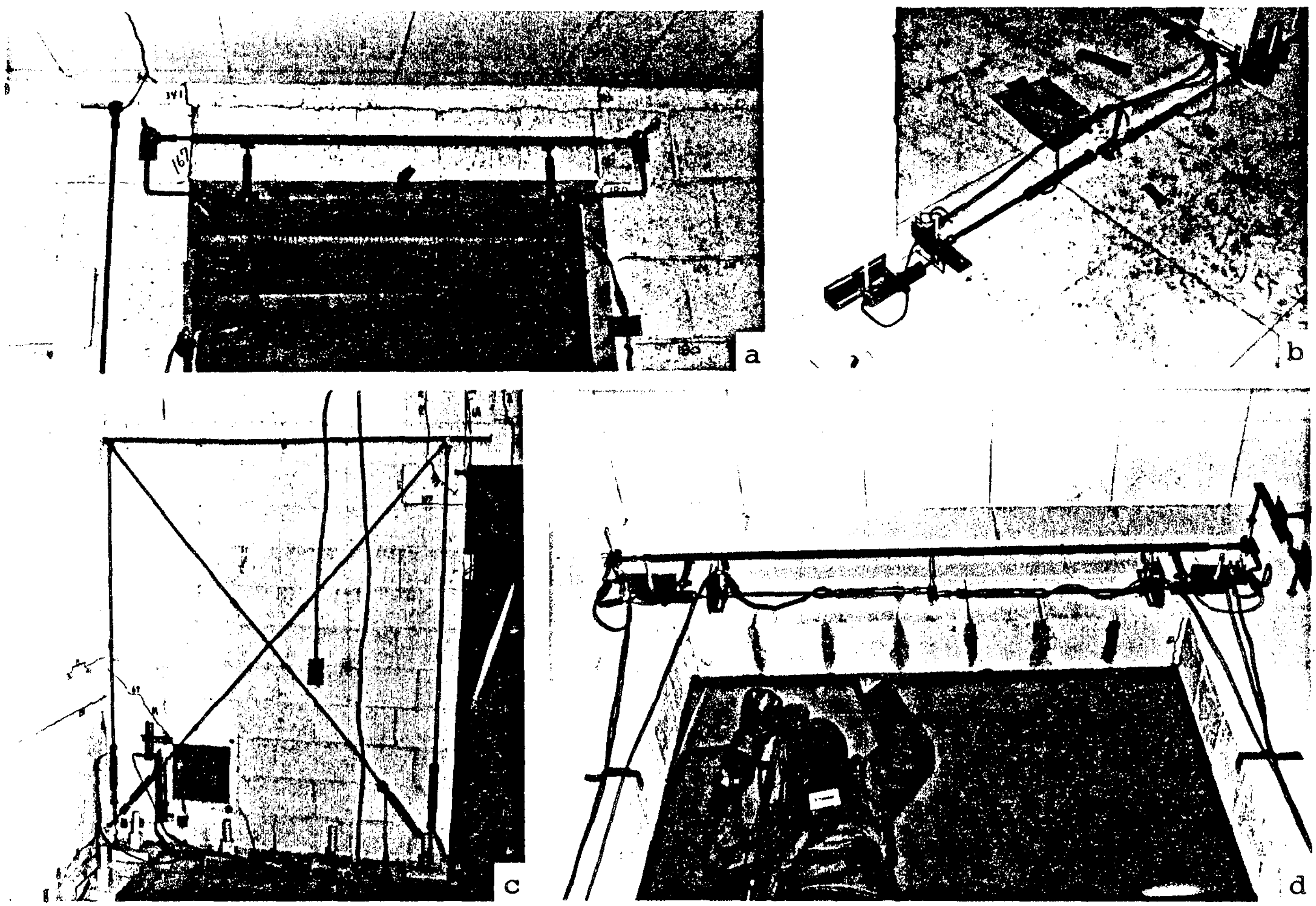

Fig. 10 Component Instrumentation a) Rotational b) Flexural (top) c) Diagonal d) Flexural (bottom)

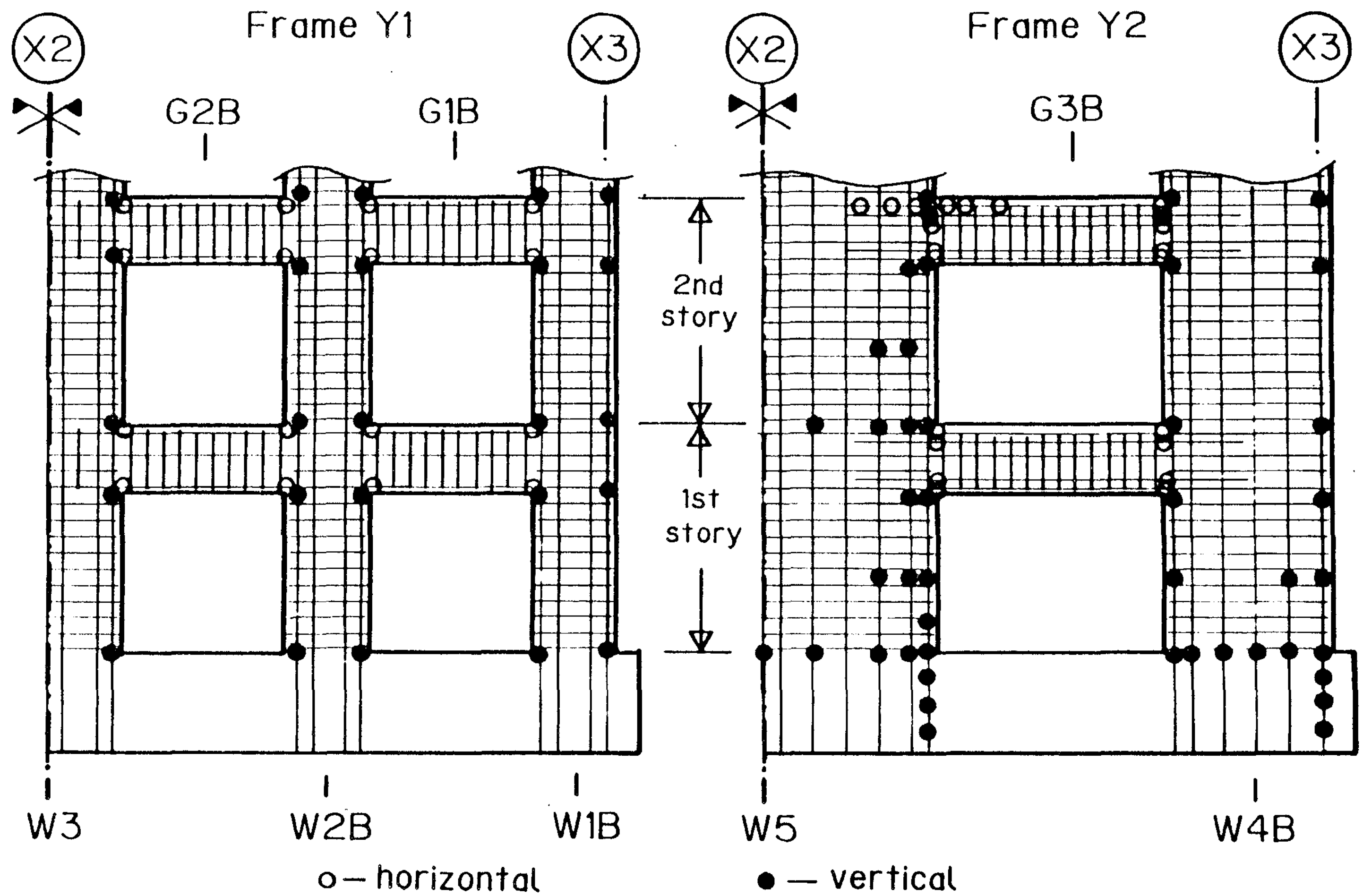

Fig. 11 Example of Rebar Strain Gage Locations in Frames Y1 and Y2

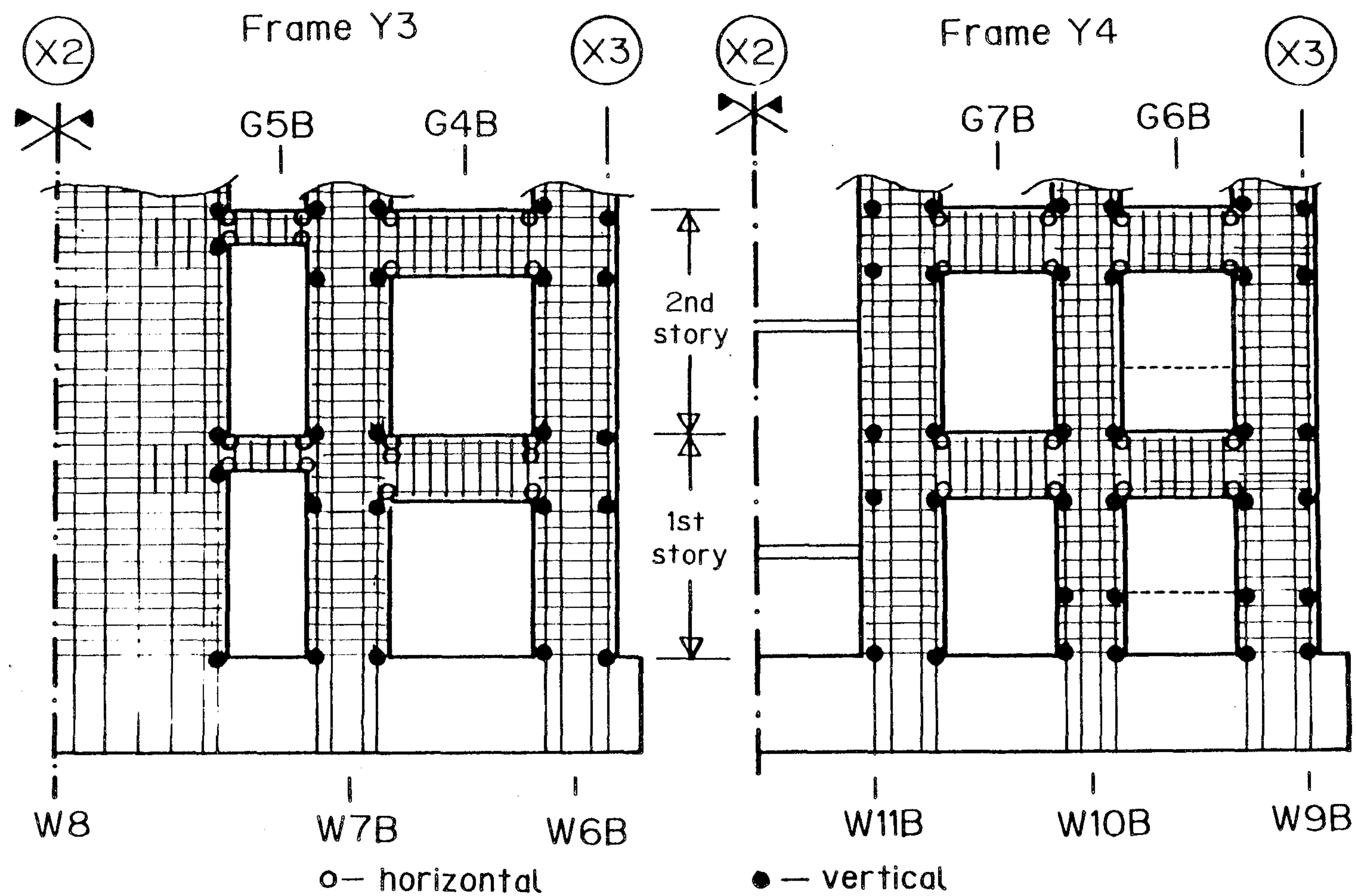

Fig. 12 Example of Rebar Strain Gage Locations in Frames Y3 and Y4

stories of each of the lateral load resisting frames are depicted in Figs.11 and 12.

Electrical resistance rebar gages were also provided in parts of the transverse walls close to wall-to-wall intersections and in the floor slabs close to floor-to-wall intersections in order to establish experimentally the contributing effective width of flanged elements.

CONCLUSIONS

The overall test plan for the Japanese 5-story full scale reinforced masonry research building was designed to provide relevant behavioral response data for the verification of design models for medium rise reinforced masonry structures which are incorporated in the draft of the new Japanese design guidelines. The loading history which is comprised of a series of increasing cyclic static load steps and forced vibration tests at important limit states, as well as a pseudo dynamic test during the inelastic structural response phase, establishes the experimental database for the analytical modeling of reinforced masonry structures under seismic loading.

The described, extensive external reference instrumentation is used to determine the global structural response and to correlate the test structure behavior with appropriate concentrated parameter models, while the internal or component instrumentation is used to determine the response state of selected structural components. This component data is important not only to establish sequential yield mechanism formation but also to correlate independent component tests with integrated component tests to see if component data can be assembled to predict the prototype structural response.

The response of the test structure to the described load history and analytical correlation studies, as well as the design and the construction of the test specimen are presented in separate papers.

ACKNOWLEDGEMENTS

The described research effort was only possible through the dedicated contributions of many individuals and organizations. The writers would like to express their gratitude to the Japanese Promotion Committee for Masonry Research (PROCMAR, chaired by Prof.H.Umemura), for providing the 5-story full scale masonry test building. Special thanks and appreciation goes to all the Japanese TCCMAR (Technical Coordinating Committee for Masonry Research) and BLDCMAR (Building Construction Committee for Masonry Research, chaired by Prof.K.Kamimura) members who contributed invaluably to the design and the testing of the full scale masonry research building. Principal funding for the described masonry research was provided by the Ministry of Construction of the Japanese Government as part of the UJNR cooperative research program. The contributions of the U.S.-TCCMAR members in numerous discussions and joint meetings are greatly appreciated. In particular the outstanding and dedicated leadership of the TCCMAR coordinators, Dr. James L. Noland on the U.S. side, and Dr. Shin Okamoto on the Japanese side ensured the success of this cooperative research project.

APPENDIX I

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APPENDIX VI

INSTITUTE OF INDUSTRIAL SCIENCE
UNIVERSITY OF TOKYO

Prof. Frieder Seible

貴殿が当所においてなされた下記の講演はまことに
有益でありました　ここに深く感謝の意を表します

I deeply appreciate your significant contributions
by your lecture entitled :

FULL SCALE INTERACTIVE STRUCTURAL TESTING IN THE U.S.
-EXPERIMENTAL+ANALYTICAL FACILITIES AND DEVELOPMENTS-

given here on November 24, 1987.

増子昇

Noboru Masuko
Director

東京大学生産技術研究所長印