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Abstract Journal in Earthquake Engineering

Volume 7: 1977 Literature
December 1978
Earthquake Engineering Research Center
University of California, Berkeley

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1 (a)

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Contents

Preface

The *Abstract Journal in Earthquake Engineering* is a comprehensive annual collection of abstracts and citations of current literature pertinent to the field of earthquake hazard mitigation. The present volume contains more than 1,350 abstracts of technical papers, research reports, books, codes, and conference proceedings. The abstracts are obtained from 82 technical journals, and from the publications of academic, professional, and governmental institutions in 23 countries. The staff of the Abstract Journal sincerely appreciates the efforts of those many individuals and organizations who have made valuable contributions to Volume 7.

National Information Service for Earthquake Engineering

The publication of the Abstract Journal is one of the principal activities of the National Information Service for Earthquake Engineering (NISEE). The information service was established in 1971 as a joint project of the University of California, Berkeley, and the California Institute of Technology. NISEE is sponsored by the National Science Foundation under a public service grant. The staff of the Earthquake Engineering Research Center at UC Berkeley is responsible for the publication of the Abstract Journal.

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We wish to thank those users who have commented on Volume 6. To assist us in further improving the journal, we continue to welcome such constructive criticism and suggestions.

R. C. DENTON, Editor

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Journals Surveyed

The journals listed below were surveyed for the purpose of collecting abstracts for this issue of the *Abstract Journal in Earthquake Engineering.* The Earthquake Engineering Research Center wishes to express its gratitude to the publishers of many of these journals for granting permission to reprint selected abstracts and summaries.

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*Applied Mathematical Modelling**
IPC Science and Technology Press Ltd.
IPC House
32 High Street
Guildford, Surrey GU1 3EW
England

Beton i zhelezobeton
Structural Literature
Pr. Vladimirova 4, K-12
Moscow 103012
Union of Soviet Socialist Republics

Bibliography of Seismology
International Seismological Centre
Newbury RG13 1LA, Berkshire
England

Bollettino di Geofisica
Osservatorio Geofisico Sperimentale
34123 Trieste
Italy

*Building and Environment**
Pergamon Press, Inc.
Maxwell House
Fairview Park
Elmsford, New York 10523

*Bulletin of the Association of Engineering Geologists**
2570 Oakwood Manor Drive
Florissant, Missouri 63031

Bulletin of the Disaster Prevention Research Institute
Kyoto University
Kyoto, Japan

Bulletin of the Indian Society of Earthquake Technology
Prabhat Press
Meerut, U.P., India

Bulletin of the Institution of Engineers (India)
8 Gokhale Road
Calcutta, 700 020, India

Bulletin of the International Institute of Seismology and Earthquake Engineering
3-28-8 Hyakunin-cho
Shinjuku-ku
Tokyo, Japan

Bulletin of the New Zealand National Society for Earthquake Engineering
P.O. Box 243
Wellington, New Zealand

*Bulletin of the Seismological Society of America**
P.O. Box 826
Berkeley, California 94701

California Geology
California Division of Mines and Geology
P.O. Box 2980
Sacramento, California 95812

*Canadian Geotechnical Journal**
National Research Council of Canada
Ottawa K1A 0R6
Canada

*Canadian Journal of Civil Engineering**
National Research Council of Canada
Ottawa K1A 0R6
Canada

*Canadian Journal of the Earth Sciences**
National Research Council of Canada
Ottawa K1A 0R6
Canada

*Civil Engineering**
American Society of Civil Engineers
345 East 47th Street
New York, New York 10017

*Civil Engineering**
Morgan-Grampian, Ltd.
30 Calderwood Street
Woolwich, London SE18 6QH
England

Civil Engineering Transactions
The Institution of Engineers (Australia)
Science House
Gloucester & Essex Streets
Sydney, N.S.W.
Australia

*Computers and Structures**
Pergamon Press, Inc.
Maxwell House
Fairview Park
Elmsford, New York 10523

Cuadernos del IIEA
Instituto de Investigaciones y Estudios Avanzados
Universidad de Guayaquil
Guayaquil, Ecuador

*Earthquake Engineering and Structural Dynamics**
John Wiley & Sons, Ltd.
Baffins Lane
Chichester, Sussex
England

Earthquake Notes
Eastern Section
Seismological Society of America
E34-454, Lincoln Laboratory
Massachusetts Institute of Technology
Cambridge, Massachusetts 02142

*Emergency Planning Digest**
Emergency Planning Canada
Ottawa K1A 0W6
Canada

*Engineering Geology**
Elsevier Scientific Publishing Co.
P.O. Box 211
Amsterdam, The Netherlands

*Engineering Journal, AISC**
American Institute of Steel Construction
1221 Avenue of the Americas
New York, New York 10020

*EOS Transactions of the American Geophysical Union**
American Geophysical Union
1909 K Street, N.W.
Washington, D.C. 20006

*Experimental Mechanics**
Society for Experimental Stress Analysis
21 Bridge Square
Westport, Connecticut 06880

*Geological Society of America Bulletin**
Geological Society of America, Inc.
3300 Penrose Place
Boulder, Colorado 80301

*Geoscience Canada**
Geological Association of Canada
Department of Earth Sciences
University of Waterloo
Waterloo N2L 3G1
Canada

*Géotechnique**
The Institution of Civil Engineers
Great George Street
London, S.W. 1 England

*Geothermics**
Istituto Internazionale per le Ricerche Geotermiche
Lungarno Pacinotti 55
56100 Pisa
Italy

Ingeniería Sísmica
Sociedad Mexicana de Ingeniería Sísmica, A. C.
Apartado Postal 70-227
Mexico, 20, D.F., Mexico

*International Journal of Engineering Science**
Pergamon Press, Inc.
Maxwell House
Fairview Park
Elmsford, New York 10523

*International Journal of Fracture**
Noordhoff International Publishing
P.O. Box 26
Leyden, The Netherlands

*International Journal of Mechanical Sciences**
Pergamon Press, Inc.
Maxwell House
Fairview Park
Elmsford, New York 10523

*International Journal of Non-Linear Mechanics**
Pergamon Press, Inc.
Maxwell House
Fairview Park
Elmsford, New York 10523

*International Journal for Numerical and Analytical Methods in Geomechanics**
John Wiley & Sons, Ltd.
Baffins Lane
Chichester, Sussex
England

*International Journal for Numerical Methods in Engineering**
John Wiley & Sons, Ltd.
Baffins Lane
Chichester, Sussex
England

*International Journal of Pressure Vessels and Piping**
Applied Science Publishers, Ltd.
Ripple Road
Barking, Essex
England

*International Journal of Solids and Structures**
Pergamon Press, Inc.
Maxwell House
Fairview Park
Elmsford, New York 10523

*Iranian Journal of Science and Technology**
School of Engineering
Pahlavi University
Shiraz, Iran

*Journal of the Acoustical Society of America**
American Institute of Physics
335 East 45th Street
New York, New York 10017

*Journal of the American Concrete Institute**
American Concrete Institute
P.O. Box 19150
Redford Station
Detroit, Michigan 48219

*Journal of the American Institute of Architects**
American Institute of Architects
1735 New York Avenue, N.W.
Washington, D.C. 20006

*Journal of Applied Mechanics**
American Society of Mechanical Engineers
345 East 47th Street
New York, New York 10017

*Journal of Dynamic Systems, Measurement, and Control**
American Society of Mechanical Engineers
345 East 47th Street
New York, New York 10017

Journal of the Earth and Space Physics
Institute of Geophysics
University of Tehran
Amirabad, Tehran 14
Iran

*Journal of the Engineering Mechanics Division**
American Society of Civil Engineers
345 East 47th Street
New York, New York 10017

*Journal of Geophysical Research**
American Geophysical Union
1909 K Street, N.W.
Washington, D.C. 20006

*Journal of Geophysics**
Springer-Verlag
Postfach 105 280
D-6900 Heidelberg 1
Germany

*Journal of the Geotechnical Engineering Division**
American Society of Civil Engineers
345 East 47th Street
New York, New York 10017

*Journal of the Hydraulics Division**
American Society of Civil Engineers
345 East 47th Street
New York, New York 10017

Journal of the Institution of Engineers (India)
Civil Engineering Division
8 Gokhale Road
Calcutta, 700 020, India

*Journal de Mécanique**
Gauthier-Villars
70, rue de Saint-Mande
93100 Montreuil, Paris
France

*Journal of Mechanical Engineering Science**
The Institution of Mechanical Engineers
Northgate Avenue, Bury St. Edmunds
Suffolk IP32 6BW
England

*Journal of Physics of the Earth**
University of Tokyo Press
c/o Center for Academic Publications
4-16, Yayoi 2-chome
Bunkyo-ku
Tokyo 113, Japan

*Journal of the Prestressed Concrete Institute**
Prestressed Concrete Institute
20 North Wacker Drive
Chicago, Illinois 60606

*Journal of Sound and Vibration**
Academic Press Limited
24-28 Oval Road
London NW1 7DX
England

*Journal of the Structural Division**
American Society of Civil Engineers
345 East 47th Street
New York, New York 10017

*Journal of Structural Mechanics**
Marcel Dekker, Inc.
270 Madison Avenue
New York, New York 10016

*Journal of Testing and Evaluation**
American Society for Testing and Materials
1916 Race Street
Philadelphia, Pennsylvania 19103

*Journal of the Waterway, Port, Coastal and Ocean Division**
American Society of Civil Engineers
345 East 47th Street
New York, New York 10017

Magazine of Concrete Research
Cement and Concrete Association
Wexham Springs
Slough SL3 6PL
England

*Matériaux et Constructions**
Secrétariat Général de la RILEM
12, rue Brancion
75737 Paris Cedex 15
France

New Zealand Engineering
New Zealand Institution of Engineering
P.O. Box 12-241
Wellington, New Zealand

New Zealand Journal of Geology and Geophysics
Department of Scientific and Industrial Research
P.O. Box 9741
Wellington, New Zealand

*Nuclear Engineering and Design**
North-Holland Publishing Co.
P.O. Box 211
Amsterdam, The Netherlands

Osnovania, fundamenty i mekhanika gruntov
Stroyizdat
Second Institutskaya Str., D.6
Moscow Zh-389
Union of Soviet Socialist Republics

*Physics of the Solid Earth**
American Geophysical Union
1909 K Street, N.W.
Washington, D.C. 20006

*Proceedings**
The Institution of Civil Engineers
Great George Street
London, S.W. 1
England

*Quarterly Reports**
Railway Technical Research Institute
Japanese National Railways
Kunitachi P.O. Box 9
Tokyo, Japan

*Science**
American Association for the Advancement of Science
1515 Massachusetts Avenue, N.W.
Washington, D.C. 20005

Soils and Foundations
Japanese Society of Soil Mechanics and Foundation Engineering
Toa Bekkan Building
13-5 1-chome
Nishi-Shinbashi
Minato-ku
Tokyo, Japan

Stroitel'naya mekhanika i raschet sooruzhenii
Stroyizdat
Second Institutskaya Str., D.6
Moscow Zh-389
Union of Soviet Socialist Republics

*The Structural Engineer**
The Institution of Structural Engineers
11 Upper Belgrave Street
London SW1X 8BH
England

*Surveying and Mapping**
American Congress on Surveying and Mapping
P.O. Box 601
Falls Church, Virginia 22046

*Technocrat**
Fuji Marketing Research Co., Ltd.
3F Kohri Building
6-11-17, Roppongi
Minato-ku
Tokyo 106, Japan

*Tectonophysics**
Elsevier Scientific Publishing Co.
P.O. Box 211
Amsterdam, The Netherlands

*Transactions of the Architectural Institute of Japan**
Architectural Institute of Japan
19-2, 3 Chome Ginza
Chuoku
Tokyo, Japan

Voprosy inzhenernoi seismologii
Institute of Earth Physics
U.S.S.R. Academy of Sciences
Nauk Publishing House
Podsosenskii Per., 21
Moscow, K-62
Union of Soviet Socialist Republics

Zisin, Journal of the Seismological Society of Japan
Seismological Society of Japan
Earthquake Research Institute
University of Tokyo
Yayoi, Bunkyo-ku
Tokyo, Japan

1. General Topics and Conference Proceedings

1.1 General

● 1.1-1 Sharpe, R. L., **The earthquake problem,** *Reinforced Concrete Structures in Seismic Zones, SP-53,* American Concrete Inst., Detroit, 1977, 25–46.

An overview of the earthquake problem is presented. How earthquakes occur, the resulting ground motions, effects of site conditions, response of structures, performance of concrete buildings and recent design improvements are discussed. Damaging earthquakes can occur in many areas of the world and therefore must be considered in the design of buildings of all types. Thousands of concrete buildings have been subjected to damaging earthquake ground motions and, in general, their performance has been reasonably good although some have collapsed. Careful quality assurance in design and construction and the use of improved design provisions, tempered with sound earthquake engineering judgment, can provide good-performing concrete buildings.

● 1.1-2 Scalzi, J. B., **The earthquake engineering program of the National Science Foundation,** *Wind and Seismic Effects,* VI-28–VI-36. (For a full bibliographic citation, see Abstract No. 1.2-4.)

The research program, supported by the National Science Foundation, is given in general terms. The required interaction between social, economic, and technological areas is described.

● 1.1-3 Torres, K. and Cochran, A., **A selected, partially annotated bibliography of recent (1975-1976) natural hazards publications,** Natural Hazards Research and Applications Information Center, Univ. of Colorado, Boulder, 1977, 80.

Presented is an annotated bibliography of publications concerned with loss reduction from natural hazards. Hazards include earthquakes, tsunamis, floods, hurricanes, cyclones, tornados, volcanos, and others.

● 1.1-4 **Reference monography,** Inst. of Earthquake Engineering and Engineering Seismology, Univ. Kiril and Metodij, Skopje, Yugoslavia, Apr. 1976, 45.

The main research activities of the Inst. of Earthquake Engineering and Engineering Seismology in Skopje, Yugoslavia, are presented. Staff training, post-graduate, and special study programs are described. Laboratory equipment and professional publications and reports are listed.

● 1.1-5 Habercom, Jr., G. E., ed., **Earthquake engineering: buildings, bridges, dams, and related structures. Vol. 2. 1974–September 1977; a bibliography with abstracts,** National Technical Information Service, Springfield, Virginia, Oct. 1977, 365. (NTIS Accession No. NTIS/PS-77/0862)

Seismic phenomena relative to buildings, bridges, dams, and other structures are investigated. Damage assessment is made and design inadequacies are revealed. Suggestions for structural improvements for dynamic response are presented. Abstracts on site selection and earthquake-proofing for atomic power plants are included. This updated bibliography contains 365 abstracts, 125 of which are new entries to the previous edition.

● 1.1-6 White, A. G., **Earthquakes and cities: a selected bibliography,** Vance, M., ed., *Exchange Bibliography 1109,* Council of Planning Librarians, Monticello, Illinois, Aug. 1976, 9.

This report contains 77 bibliographic references, covering the years 1906–1975.

● *See **Preface**, page v, for availability of publications marked with dot.*

● **1.1-7** Bath, M., **Seismological Institute Uppsala progress report 1971 - 1975,** *1-76,* Seismological Inst., Uppsala, Sweden, 1976, 41.

The Seismological Inst., Uppsala, operates a seismograph station network covering almost the entire area of Sweden. It issues continuously the corresponding seismological bulletins, and it conducts basic and applied research within the whole range of seismology. In addition, it offers teaching and research guidance, leading to a doctoral degree in the subject. The paper reviews all activities of the institute from 1971 to 1975, inclusive, and is a continuation of earlier reports.

● **1.1-8** **Learning from earthquakes: 1977 planning and field guides,** Earthquake Engineering Research Inst., Oakland, California, 1977, 200.

The contents include: I. Planning Guide; II. Preface to the Field Guides; III. Engineering Field Guide; IV. Geoscience Field Guide; and V. Social Science Field Guide.

● **1.1-9** Krishna, J. and Chandrasekaran, A. R., **Elements of earthquake engineering,** Sarita Prakashan, Nauchandi, India, 1976, 250.

The contents are as follows: 1: The earthquakes. 2: Vibrations of single degree of freedom system. 3: Vibrations of multiple degrees of freedom systems. 4: Continuous systems. 5: Earthquake motion and response. 6: Aseismic design of structures. Appendix: comprehensive intensity scale.

● **1.1-10** Bolt, B. A. *et al.,* **Geological hazards: earthquakes-tsunamis-volcanoes-avalanches-landslides-floods,** 2nd rev. ed., Springer-Verlag, New York, 1977, 330.

This edition provides a supplement for introductory and advanced courses in geology, engineering, geography, and environmental studies. In addition to chapters on each geological hazard mentioned in the title, a chapter is included on hazard mitigation and control. Also included are a subject index and the following appendixes: A. Notable World Earthquakes; B. Important Earthquakes of the United States, Canada and Mexico; C. Number of Active and Other Geologically Recent Volcanoes in Various Regions; D. Major Flood Disasters of the World 1963–1974; E. Metric–English Conversion Table; and F. Geologic Time Scale.

1.2 Proceedings of Conferences

● **1.2-1** **Proceedings of U.S.-Japan Seminar on Earthquake Engineering Research with Emphasis on Lifeline Systems,** Japan Society for the Promotion of Earthquake Engineering, Tokyo, Nov. 1976, 499.

The seminar was held in Tokyo, Nov. 8–12, 1976. The U.S. National Science Foundation and the Japan Society for the Promotion of Science sponsored the seminar. The 32 papers presented at the seminar provide a representative cross section of the lifeline earthquake engineering research being conducted in the United States and Japan. The paper titles and authors are listed below. All papers are abstracted in this volume of the *AJEE.*

Earthquake resistance of civil structures in Japan, Okamoto, S.—*Earthquake problems of networks and systems,* Jennings, P. C.—*Strain amplitude by body and surface waves in a near surface ground,* Toki, K.—*Observed earthquake ground displacements along a 2,500 meter line,* Tsuchida, H. and Kurata, E.—*Effects of traveling seismic waves on structure response: a research program,* Werner, S. D.—*Statistical analysis of strong-motion earthquake response spectra,* Iwasaki, T. and Katayama, T.—*Earthquake motions parameters affecting structural response statistics,* Kameda, H.—*Most important factors for aseismic characteristics of bridges,* Kubo, K.—*Aseismic displacement requirements at highway bridge girder supports,* Goto, H., Iemura, H. and Nakata, T.—*Vibrational characteristics of bridges with damping devices,* Kuranishi, S. and Takahashi, T.—*Seismic damage of embankment by quantification analysis,* Hoshiya, M.—*Study on pipeline failure due to earthquake,* Narita, K.—*An example of seismic design and earthquake response measurement of buried pipeline,* Miyajima, N. *et al.*—*Effect of ground conditions on seismic damage to buried pipelines,* Katayama, T.—*Dynamic frictional forces and efficiency of joint parts for aseismic strength of buried pipelines,* Takada, S. and Nagao, S.—*Some examples of earthquake resistant design in electric power supply system,* Egawa, K.—*Protecting communications equipment against earthquakes,* Foss, J. W.—*Response analysis of 500KV circuit breaker with nonlinear damping devices under seismic excitation,* Shimogo, T. and Fujimoto, S.—*The earthquake design and analysis of equipment isolation systems,* Iwan, W. D.—*Earthquake resistant design of long-span bridges,* Yamada, Y.—*Evaluating power system response to earthquakes with simulation,* Schiff, A. J., Feil, P. J. and Newsom, D. E.—*Dynamics of fixed-base liquid-storage tanks,* Veletsos, A. S. and Yang, J. Y.—*Aseismic design of liquid storages,* Sogabe, K., Shigeta, T. and Shibata, H.—*Design of cooling towers to resist earthquakes,* Schnobrich, W. C. and Gupta, A. K.—*Earthquake resistant design of intake-outlet towers,* Chopra, A. K. and Liaw, C. Y.—*Control of train operation on the new trunk lines on the occasion of earthquake,* Tamura, K., Ashida, Y. and Okamoto, S.—*Design of underground structures by considering ground displacement during earthquakes,* Tamura, C.—*Damage probability of line structures due to earthquake,* Hakuno, M.—*Active and passive instrumentation of hospitals,* Hart, G. C.—*A trend on earthquake engineering researches for lifeline systems,* Kuribayashi, E. and Tazaki, T.—*Organization and activities of the ASCE Technical Council on Lifeline Earthquake*

● *See **Preface,** page v, for availability of publications marked with dot.*

Engineering, Parmelee, R. A.—*Fundamental concept of aseismic design code of lifeline systems and industrial facilities,* Shibata, H.

● 1.2-2 Jaeger, T. A., comp., **International Seminar on Extreme Load Conditions and Limit Analysis Procedures for Structural Reactor Safeguards and Containment Structures,** Bundesanstalt fur Materialprufung (BAM), Berlin, July 1975, var. pp.

The seminar was sponsored by and held at the Bundesanstalt fur Materialprufung (BAM) in Berlin, Sept. 8–11, 1975. The International Assn. for Structural Mechanics in Reactor Technology organized the seminar, which was performed in connection with the 3rd International Conference on Structural Mechanics in Reactor Technology held in London, Sept. 1–5, 1975 (see abstract No. 1.2-11, Vol. 6, *AJEE*).

Papers of interest to earthquake engineers are contained in Part T: "Methods for Dynamic Structural Analysis" and Parts U I, II, and Supplement III: "Seismic Response Analysis of Nuclear Power Plant Systems." Following are the paper titles, authors' names, and paper numbers (in parentheses). All papers are abstracted in this volume of the *AJEE.* Those papers with asterisks following the titles have also been printed in *Nuclear Engineering and Design;* the abstracts for these papers are cited only from that publication.

Part T—Methods for Dynamic Structural Analysis: *Comparative evaluation of numerical methods for dynamic structural analysis,* Belytschko, T. (T 1/1)—*Matrix of transmission in structural dynamics,* Mukherjee, S. (T 1/2)—*Analysis of equations of motion with complex stiffness mode superposition method applied to systems with many degrees of freedom,* Tsushima, Y., Jido, J. and Mizuno, N. (T 1/3)—*On the incorporation of damping in large, general-purpose computer programs,* Nelson, F. C. and Greif, R. (T 1/4)—*Calculation of transient, non-linear phenomena in nuclear reactors using the PISCES computer codes,* Birnbaum, N. K. *et al.* (T 2/1)—*Transient dynamic and inelastic analysis of shells of revolution—a survey of programs,* Svalbonas, V. (T 2/3)—*Prediction of the nonlinear dynamic response of structural components using finite elements,* Donea, J., Giuliani, S. and Halleux, J. P. (T 2/4)—*Dynamic ultimate load analysis using a finite difference method,* Stangenberg, F. (T 3/3)—*Dynamic plastic buckling of cylindrical and spherical shells,* Jones, N. and Okawa, D. M. (T 3/4)—*Extremum principles in the dynamics of rigid-plastic bodies: a critical review of existing applications,* Wierzbicki, T. (T 3/5)—*Mathematical programming methods for displacement bounds in elasto-plastic dynamics,* Corradi, L. (T 3/6).

Part U—Seismic Response Analysis of Nuclear Power Plant Systems I: *Generation of artificial time-histories, rich in all frequencies, from given response spectra,** Levy, S. and Wilkinson, J. P. D. (U 1/3)—*Development and use of seismic instructure response spectra in nuclear plants,** Stoykovich, M. (U 1/5)—*Probabilistic frequency variations of structure-soil systems,** Hamilton, C. W. and Hadjian, A. H. (U 2/5)—*Effect of non-linear soil-structure interaction due to base slab uplift on the seismic response of a high-temperature gas-cooled reactor (HTGR),** Kennedy, R. P. *et al.* (U 2/6)—*Soil-structure interaction with separation of base mat from soil (lifting-off),** Wolf, J. P. (U 2/7).

Part U—Seismic Response Analysis of Nuclear Power Plant Systems II: *Non-linear dynamic response of reactor containment,** Takemori, T., Sotomura, K. and Yamada, M. (U 4/1)—*Axisymmetric finite element analyses of the KKP-II containment and reactor pressure vessel structures,* Kost, G., Tsui, E. Y. W. and Krutzik, N. (U 4/2)—*Bell-ring vibration response of nuclear containment vessel with attached masses under earthquake motion,** Shiraki, K. *et al.* (U 4/3)—*Aseismic design of turbine houses for nuclear power plants,** Danisch, R. and Labes, M. (U 4/4)—*Three-dimensional dynamic response modelling for floating nuclear power plants using finite element methods,** Johnson, H. W. *et al.* (U 4/5)—*Seismic analysis of a reactor coolant pump by the response spectrum method,** Villasor, Jr., A. P. (U 5/4)—*Scram and nonlinear reactor system seismic analysis for a liquid metal fast reactor,** Morrone, A., Nahavandi, A. N. and Brussalis, W. G. (U 5/7).

Part U—Supplement III: *Special topics on soil-structure interaction,** Hall, Jr., J. R. and Kissenpfennig, J. F. (U 2/2)—*State-of-the-art of seismic design of nuclear power plants—an assessment,** Howard, G. E., Ibanez, P. and Smith, C. B. (U 3/1)—*A discussion of coupling and resonance effects for integrated systems consisting of subsystems,* Lin, C.-W. and Liu, T. H. (U 3/3)—*Some comments on the seismic loading condition and the design criteria of nuclear vessels, pipings, and other equipment,* Shibata, H. (U 5/1)—*Comparative methods for analysis of piping systems subjected to seismic motion,** Hure, D. and Morysse, M. (U 5/2)—*A calculation model for a HTGR core seismic response—comparison with experimental results on the VESUVE shaking table,* Buland, P. *et al.* (U 5/5)—*Seismic model of the gas cooled fast breeder reactor core support structure,** Penzes, L. E. (U 5/6).

● 1.2-3 Hawkins, N. M. and Mitchell, D., eds., **Reinforced concrete structures in seismic zones,** *SP-53,* American Concrete Inst., Detroit, 1977, 485.

The Symposium on Reinforced Concrete Structures in Seismic Zones was held during the 1974 American Concrete Inst. (ACI) Annual Convention in San Francisco. ACI and the Structural Engineers Assn. of Northern California

● *See* ***Preface****, page v, for availability of publications marked with dot.*

sponsored the symposium. Seventeen papers were presented during the symposium. Following the symposium, the speakers and others whose papers could not be presented because of time limitations were invited to submit articles for this volume. That invitation generated sixteen of the papers in the volume. To that number were added two papers which had been published previously in the ACI journal and which had considerable significance for understanding the behavior of concrete structures in seismic zones. An introduction by the editors and a combined subject-author index are included.

The following is a list of the papers; all are abstracted in this volume of the *AJEE: The earthquake problem,* Sharpe, R. L.—*Seismic design of reinforced concrete buildings from a designer's viewpoint,* Englekirk, R. E.—*Predicting the earthquake response of reinforced concrete structures,* Clough, R. W.—*Resonance capacity criterion for evaluation of the aseismic capacity of reinforced concrete structures,* Yamada, M. and Kawamura, H.—*Inelastic responses of reinforced concrete structures to earthquake motions,* Gulkan, P. and Sozen, M. A.—*Ductile shear walls in earthquake resistant multistory buildings,* Fintel, M.—*Ductility of reinforced concrete shearwalls for seismic areas,* Paulay, T.—*Shear strength of low-rise walls with boundary elements,* Barda, F., Hanson, J. M. and Corley, W. G.—*Experimental investigation of seismic shear transfer across cracks in concrete nuclear containment vessels,* Laible, J. P., White, R. N. and Gergely, P.—*Displacement versus force-seeking effects for reinforced concrete frames,* Ernst, G. C. and Smith, G. M.—*Seismic behavior of ductile moment-resisting reinforced concrete frames,* Bertero, V. V. and Popov, E. P.—*Strength and ductility of cast-in-place beam-column joints,* Uzumeri, S. M.—*Reinforced concrete connection hysteresis loops,* Townsend, W. H. and Hanson, R. D.—*Steel fibrous, ductile concrete joint for seismic-resistant structures,* Henager, C. H.—*Anchorage of reinforcing bars for seismic forces,* Hassan, F. M. and Hawkins, N. M.—*Prediction of the seismic loading anchorage characteristics of reinforced bars,* Hassan, F. M. and Hawkins, N. M.—*Biaxial inelastic frame seismic behavior,* Selna, L. G. and Lawder, J. H.—*Earthquake—repair—earthquake,* Gallegos, H. and Rios, R.

● **1.2-4** Lew, H. S., ed., **Wind and seismic effects,** *NBS Special Publication 477,* Proceedings of the Eighth Joint Panel Conference of the U.S.-Japan Cooperative Program in Natural Resources, U.S. National Bureau of Standards, Washington, D.C., May 1977, 626.

The Eighth Joint Meeting of the U.S.-Japan Panel on Wind and Seismic Effects was held in Gaithersburg, Maryland, on May 18–21, 1976. The proceedings of the joint meeting include the program, the formal resolutions, and the technical papers. The subject matter covered in the papers includes wind effects on structures and design criteria; extreme winds for structural design; earthquake ground motions and instrumentation; seismicity and earthquake risk; seismic effects on structures and design criteria; lessons learned from recent natural disasters; and design of nuclear reactor facilities.

Twenty-six papers pertinent to earthquake engineering are abstracted in this volume of the *AJEE.* The following are the titles and authors' names: *Planning and design of strong motion instrument networks,* Matthiesen, R. B.—*Observation of earthquake response of ground with horizontal and vertical seismometer arrays,* Hayashi, S., Tsuchida, H. and Kurata, E.—*Building strong motion earthquake instrumentation,* Rojahn, C.—*Characteristics of underground seismic motions at four sites around Tokyo Bay,* Iwasaki, T., Wakabayashi, S. and Tatsuoka, F.—*Relationship between earthquake damage of existing wooden houses and seismic intensities,* Kuribayashi, E., Tazaku, T. and Hadate, T.—*A method for calculating nonlinear seismic response in two dimensions,* Joyner, W. B.—*A new scale representing the "quake sensitivity" at a certain region,* Terashima, T. and Santo, T.—*Seismic response of reinforced concrete highway bridges,* Penzien, J. *et al.*—*An evaluation method for the earthquake resistant capacity of reinforced concrete and steel reinforced concrete columns,* Ozaki, M. and Ishiyama, Y.—*The earthquake engineering program of the National Science Foundation,* Scalzi, J. B.—*Large-scale testing programs related to wind and seismic effects currently under way in Japan,* Inaba, S.—*Earthquake damages to earth structures,* Sawada, K.—*Dynamic test of a circuit breaker for transformer substation,* Inaba, S. and Kinoshita, S.—*Comprehensive seismic design provisions for buildings—A status report,* Culver, C.—*Retrofitting of vulnerability in earthquake disaster mitigation problems,* Ichihara, K., Kuribayashi, E. and Tazaki, T.—*Dynamic response characteristics of a model arch dam,* Norman, C. D., Crowson, R. D. and Balsara, J. P.—*The measurement of the dynamic k-value in site and its application to design,* Kunihiro, T., Yahagi, K. and Okahara, M.—*Laboratory investigation of undisturbed sampling and standard penetration tests on fine sands,* Marcuson, III, W. F., Cooper, S. S. and Bieganousky, M. A.—*Dynamic soil properties with emphasis on comparison of laboratory tests and field measurements,* Iwasaki, T. and Tatsuoka, F.—*Design earthquakes,* Krinitzsky, E. L. and Chang, F. K.—*Relation between seismic coefficient and ground acceleration for gravity quaywall,* Hayashi, S., Noda, S. and Uwabe, T.—*Investigation of earthquake resistance of structural (shear) wall buildings carried out at Portland Cement Association,* Fintel, M.—*A philosophy for structural integrity of large panel buildings,* Fintel, M. and Schultz, D. M.—*Wind and seismic design of U.S. nuclear power plants,* Shao, L. C., Stuart, R. J. and Hofmayer, C. H.—*Outline of basic philosophy and practices of aseismatic design for nuclear facilities in Japan,* Watabe, M. and Ohsaki, Y.—*Structural damage to bridges resulting from the Guatemala earthquake,* Cooper, J. D.

● *See **Preface**, page v, for availability of publications marked with dot.*

● 1.2-5 Jaeger, T. A. and Boley, B. A., eds., **Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology**, Commission of the European Communities, Brussels, 1977.

The conference was organized by the International Association for Structural Mechanics in Reactor Technology and the Commission of the European Communities, in cooperation with eight other organizations. The conference was held in San Francisco from Aug. 15-19, 1977. The papers are arranged into a set of volumes according to the technical divisions of the conference. Several volumes contain papers pertinent to earthquake engineering; however, because of publication deadlines, the papers from only two volumes—K(a) and (b) Seismic Response Analysis of Nuclear Power Plant Systems—are abstracted or cited in this volume of the *AJEE*. The following are the paper titles, authors' names, and paper numbers (in parentheses). For the titles with an asterisk following, only the abstract of the paper was published in the transactions; these abstracts have not been included in the Abstract Journal.

Vol. K(a) Seismic Response Analysis of Nuclear Power Plant Systems

Session K 1 I. Ground Motion and Design Criteria I: *The determination of seismic design criteria,** Jennings, P. C. (K1/1)—*Measures taken in the member countries of the European communities for anti-seismic design compared to actual U.S. practice,* Vinck, W. and Maurer, H. A. (K1/2)—*Seismic risk maps of Switzerland,* Sagesser, R., Rast, B. and Merz, H. (K1/3)—*Development of ground response spectra from site Mercalli intensities,** Werner, S. D. and Ts'ao, H. S. (K1/4)—*Significance of seismic response spectrum normalization in nuclear power plant design,** Dalal, J. S. and Perumalswami, P. R. (K1/5)—*Comments on earthquake response spectra,** Mohraz, B. (K1/6)—*Cyclic characteristics of earthquake time histories,* Hall, Jr., J. R., Shukla, D. K. and Kissenpfennig, J. F. (K1/7).

Session K 1 II. Ground Motion and Design Criteria II: *Simulated earthquake ground motions,* Vanmarcke, E. H. and Gasparini, D. A. (K1/9)—*Automated generation of spectrum compatible artificial time histories,** Kost, G. *et al.* (K1/10)—*Criteria for the generation of spectra consistent time histories,* Lin, C.-W. (K1/11)—*Application of method of characteristics in seismic analysis,* Huang, W., Gupta, D. C. and Agrawal, P. K. (K1/12)—*Strong ground motion spectra for layered media,* Askar, A., Cakmak, A. S. and Engin, H. (K1/13)—*An improved algorithm for non-linear soil amplification,* Kausel, E., Christian, J. T. and La Plante, F. J. (K1/14)—*Shear stress distribution due to shear and Rayleigh wave propagation at deep soil sites,* Hall, Jr., J. R., Shukla, D. K. and Kissenpfennig, J. F. (K1/15).

Session K 2 I. Soil Structure Interaction I: *Soil-structure interaction analysis by finite element methods—state-of-the-art,* Seed, H. B. and Lysmer, J. (K2/1)—*Comparative aseismic response study of different analytical models of nuclear power plant,* Takemori, T. *et al.* (K2/2)—*Finite element random vibration method for soil-structure interaction analysis,* Romo-Organista, M. P., Lysmer, J. and Seed, H. B. (K2/3)—*Numerical analysis of soil-structure systems of unbounded geometry,* Melosh, R. J. and Buyukozturk, O. (K2/4)—*Nonlinear seismic soil-structure interaction analysis of nuclear power plant structures,* Khanna, J. K., Setlur, A. V. and Pathak, D. V. (K2/5)—*Dynamic analysis of embedded structures,* Kausel, E. *et al.* (K2/6)—*Earthquake response of nuclear reactor building deeply embedded in soil,* Masao, T. *et al.* (K2/7)—*Earthquake analysis of structures including structure-soil interaction by a substructure method,* Chopra, A. K. and Gutierrez, J. A. (K2/8).

Session K 2 II. Soil Structure Interaction II: *Experimental and analytical investigations in nonlinear dynamic soil-structure interaction,** Chan, C. *et al.* (K2/10)—*Monodimensional schematization of soil for seismic response analysis* (in French), Costes, D. (K2/11)—*The effects of soil-structure interaction modeling techniques on in-structure response spectra,* Johnson, J. J., Wesley, D. A. and Almajan, I. T. (K2/12)—*Effective seismic input through rigid foundation filtering,** Ray, D. and Jhaveri, D. P. (K2/13)—*Assessment of seismic wave effects on soil-structure interaction,* Bernreuter, D. L. (K2/14)—*Base response arising from free-field motions,* Whitley, J. R. *et al.* (K2/15)—*Seismic response analysis of nuclear reactor buildings under consideration of soil-structure interaction with torsional behaviour,* Mizuno, N. *et al.* (K2/16)—*The multiple structure-soil interaction problem,* Parker, J. V., Ahmed, K. M. and Ranshi, A. S. (K2/17)—*A modified rigid base approach for the seismic analysis of nuclear power plants,** Chakraborty, S. K. and Mukherjee, A. N. (K2/18).

Session K 3 I. Dynamic Modeling of Structures, Systems and Components I: *Seismic analysis of structures by simulation,* Sundararajan, C. and Gangadharan, A. C. (K3/2)—*A probabilistic model for seismic analysis of nuclear plant structures,** Singh, A. K. and Singh, S. (K3/3)—*Seismic structural response analysis using consistent mass matrices having dynamic coupling,* Shaw, D. E. (K3/4)—*Response spectrum analysis of coupled structural response to a three component seismic disturbance,* Boulet, J. A. M. and Carley, T. G. (K3/5)—*A finite element method for seismic response analysis,* Yu, I.-W. (K3/6)—*Post-earthquake evaluation of nuclear piping systems,** Matzen, V. C., McNiven, H. D. and Mayes, R. L. (K3/7).

● *See* ***Preface,*** *page v, for availability of publications marked with dot.*

Session K 3 II. Dynamic Modeling of Structures, Systems and Components II: *New beam and plate bending elements in finite element analysis,* Kawai, T. (K3/8)—*Dynamic interaction of components, structure, and foundation of nuclear power facilities,* Pajuhesh, J. and Hadjian, A. H. (K3/9)—*A simplified procedure for evaluating modal damping factors in structures with widely varying damping capacities,* Brusa, L., Ciacci, R. and Restelli, F. (K3/10)—*Constructing mathematical models of cable tray and support systems to determine seismic response in nuclear plants,* Thulin, Jr., F. A. (K3/11)—*Modeling of slabs in seismic analysis of nuclear power plant buildings,** Perumalswami, P. R. and Dalal, J. S. (K3/12)—*The effect of rotatory inertia on the dynamic response of cantilever structures,* Lin, Y. J. and Hadjian, A. H. (K3/13)—*Model of cyclic inelastic flexural behavior of reinforced concrete members,* Popov, E. P., Bertero, V. V. and Ma, S. M. (K3/14).

Session K 4 I. Seismic Response of Nuclear Power Plant Structures I: *Inelastic design of nuclear reactor structures and its implications on design of critical equipment,* Newmark, N. M. (K4/1)—*An evaluation of seismic-resistant design methods for inelastic structures,** Mahin, S. A. and Bertero, V. V. (K4/2)—*Inelastic seismic response of turbine buildings,* Hsiu, F. J. and Hanson, R. D. (K4/3)—*Inelastic seismic studies of buildings,** Gauvain, J., Livolant, M. and Hoffmann, A. (K4/4)—*Effective duration of accelerogram to produce response maxima,** Wang, W. Y. L. (K4/5)—*Comparison between time-step-integration and probabilistic methods in seismic analysis of a linear structure,* Schneeberger, B. and Breuleux, R. (K4/6)—*Analytical procedure in aseismic design of eccentric structure using response spectrum,* Takemori, T. *et al.* (K4/7)—*Dynamic analysis of a reactor building on alluvial soil,* Arya, A. S. *et al.* (K4/8).

Session K 4 II. Seismic Response of Nuclear Power Plant Structures II: *An extensive study on a simple method estimating response spectrum based on a simulated spectrum,* Sato, H., Komazaki, M. and Ohori, M. (K4/9)—*Direct evaluation of floor response spectra from a given ground response spectrum,** Schmitz, D. and Peters, K. (K4/10)—*The problem of resonance in the evaluation of floor response spectra,** Peters, K., Schmitz, D. and Wagner, U. (K4/11)—*Floor response spectra from spectrum compatible motions,* Duff, C. G. and Biswas, J. K. (K4/12)—*Development of compatible secondary spectra without time histories,* Scanlan, R. H. and Sachs, K. (K4/13)—*Comparison of artificial and natural earthquake time history functions with regard to their floor response spectra,** Busch, K. A. and Wagner, U. (K4/14)—*Standardized seismic design spectra for nuclear plant equipment,** Tsai, N. C. and Tseng, W. S. (K4/15)—*A probabilistic seismic analysis of containment liner integrity,* Fardis, M. N., Cornell, C. A. and Meyer, J. E. (K4/16)—*Seismic analysis of floating nuclear plant,** Shulman, J. S. and Orr, R. S. (K4/17).

Vol. K(b) Seismic Response Analysis of Nuclear Power Plant Systems

Session K 5. Seismic Response of Storage Tanks: *Seismic response of flexible cylindrical tanks,* Clough, R. W. and Clough, D. P. (K5/1)—*Seismic response of flexible cylindrical liquid storage tanks,* Kana, D. D. (K5/2)—*Seismic response of flexible liquid containers,* Lev, O. E. and Jain, B. P. (K5/3)—*Approximate seismic response analysis of self-supported thin cylindrical liquid storage tanks,* Fujita, K. and Shiraki, K. (K5/4)—*Effect of hydroelastic coupling on the response of a nuclear reactor to ground acceleration,* Au-Yang, M. K. and Skinner, D. A. (K5/5).

Session K 6 I. Seismic Response of Nuclear Power Plant Piping and Equipment I: *Analysis of uncertainty in seismic response of secondary appendage system,* Suzuki, K. (K6/2)—*Buried pipes under earthquake excitation,** Sachs, K. (K6/4)—*Dynamic analysis of electric equipment for nuclear power stations under seismic loads,* Buck, K. E., von Bodisco, U. and Winkler, K. (K6/5)—*Seismic analysis of a dry spent fuel cask handling system,* Morris, N. F., Harstead, G. A. and Soot, O. (K6/6)—*Seismic analysis of spent fuel racks,** Longo, R. *et al.* (K6/7)—*Seismic analysis and design of electrical cable trays and support systems,** Shahin, R. M., Manuelyan, R. and Jan, C.-M. (K6/8)—*Seismic analysis of category I overhead cranes considering structural non-linearities,** Braccio, M. and Rigamonti, G. (K6/9).

Session K 6 II. Seismic Response of Nuclear Power Plant Piping and Equipment II: *Response of a reactor coolant pump to time history seismic displacements,** Villasor, Jr., A. P. (K6/11)—*A seismic analysis of nuclear power plant components subjected to multi-excitations of earthquakes,* Ichiki, T., Matsumoto, T. and Gunyasu, K. (K6/12)—*Modal dynamic analysis of linear elastic systems with specified displacement time histories,** Chan, A. W. (K6/13)—*Seismic response analysis of structural system to multiple support excitations,** Wu, R. W., Hussain, F. A. and Liu, L. K. (K6/14)—*Analysis of high frequency and multiple-support excitation problems with KWUROHR,** Leimbach, K. R. (K6/15)—*Seismic design for steam generators—a multiple system approach,* Duff, C. G. and Asmis, G. J. K. (K6/16)—*Preliminary seismic design of dynamically coupled structural systems,* Pal, N., Dalcher, A. W. and Gluck, R. (K6/17)—*Dynamic structural analysis of uncoupled subsystems,* Gerdes, L. D. (K6/18)—*Determination of support spacing tables used in the design of safety class nuclear plant piping,** Bergman, L. A. and Stevenson, J. D. (K6/19).

● *See **Preface**, page v, for availability of publications marked with dot.*

Session K 7. Seismic Response of Reactor Core Structures: *Seismic behavior of reactor internals,** Bohm, G. J. (K7/1)—*OSCIL and OSCVERT: computer codes to evaluate the non-linear seismic response of an HTGR core,* Lasker, L. *et al.* (K7/2)—*Effect of clearance and distribution of mass on the dynamic response of an HTGR core,* Reich, M. and Koplik, B. (K7/3)—*Subharmonic excitation in an HTGR core,* Bezler, P. and Curreri, J. R. (K7/4)—*Study of fuel block collision in HTGR core,* Tzung, F.-K. (K7/5)—*Seismic response of a stacked HTGR fuel column interacting with a control rod,* Lee, T. H. and Wesley, D. A. (K7/6)—*Measuring the seismic response of an HTGR core model,* Rakowski, J. E. and Olsen, B. E. (K7/7)—*Two-dimensional vibration test and simulation analysis for a vertical slice model of HTGR core,* Muto, K. *et al.* (K7/8)—*Basic study on seismic response of HTR core,* Ishizuka, H. *et al.* (K7/9)—*Seismic model test of GCFR core support structure,** Penzes, L. E., Buttemer, D. R. and Bedore, R. L. (K7/10).

Session K 8 I. Dynamic Tests and Qualifications of Structures, Equipment, Components, and Systems I: *Seismic qualification of systems, structures, equipment, and components,* Fischer, E. G. (K8/1)—*Dynamic behavior of a nuclear reactor building,* Hirasawa, M., Okajima, S. and Satoh, K. (K8/2)—*In-situ testing for seismic evaluation of Humboldt Bay nuclear power plant for Pacific Gas and Electric Company,* Ibanez, P. *et al.* (K8/3)—*In situ dynamic tests and seismic records on the RHR system building ENEL IV nuclear plant Caorso (Italy),** Castoldi, A., Casirati, M. and Scotto, F. L. (K8/4)—*Objectives of seismic tests in the HDR safeguards research program* (in German), Muller-Dietsche, W., Jehlicka, P. and Steinhilber, H. (K8/5)—*A comparison of vibration tests and analysis on nuclear power plant structures and piping,* Gundy, W. E. *et al.* (K8/6)—*In situ measurement of dynamic characteristics of atomic power plant equipment,* Arya, A. S., Gupta, S. P. and Shrivastava, S. K. (K8/7).

Session K 8 II. Dynamic Tests and Qualifications of Structures, Equipment, Components, and Systems II: *On proving test of earthquake resistant piping and active components,** Shibata, H. (K8/8)—*Effect of damping on the response of a non-linear system with multiple sine wave excitation,* Curreri, J. R. and Bezler, P. (K8/9)—*Seismic qualification of safety related electrical equipment—latest methods that meet IEEE-344-75,** Jarecki, S. J. and Coslow, B. J. (K8/10)—*Instantaneous response spectrum in seismic testing of nuclear power plant equipment,* Morrone, A. (K8/11)—*Experimental tests results and theoretical dynamic analysis of the PEC fast reactor shutdown system during a S.S.E.,** de Sogus, C. *et al.* (K8/12)—*Seismic instrumentation for NPP—an interpretative review of current practice and the related standard in Germany,** Bork, M. and Kaestle, H. J. (K8/13).

Session K 9. Design Considerations of Nuclear Power Plant Structures, Systems and Components: *The economic effect of increased seismic load on nuclear power plant design and construction costs,** Stevenson, J. D. (K9/1)—*Aseismic foundation system for nuclear power stations,* Jolivet, F. and Richli, M. (K9/2)—*Seismic risk analysis for Canadian nuclear power plants,* Aziz, T. S. and Charlwood, R. G. (K9/3)—*On design errors and system degradation in seismic safety,* Hsieh, T.-M. and Okrent, D. (K9/4)—*Dynamic response of nuclear power plant due to earthquake ground motion and aircraft impact,* Parker, J. V., Ahmed, K. M. and Ranshi, A. S. (K9/5)—*Gapped guide-type restraints used as piping seismic restraints,** Rich, S. E. and Rigamonti, G. (K9/6)—*A rational and economical seismic design of beam columns in steel frames,* Gupta, A. K., Fang, S.-J. and Chu, S. L. (K9/7).

● **1.2-6 Earthquake engineering research at Berkeley-1976,** *UCB/EERC-77/11,* Earthquake Engineering Research Center, Univ. of California, Berkeley, May 1977, 198. (NTIS Accession No. PB 273 507)

At the Sixth World Conference on Earthquake Engineering held in New Delhi, India, Jan. 10-14, 1977, twenty-five papers were presented by faculty participants and research personnel associated with the Earthquake Engineering Research Center, Univ. of California, Berkeley. The papers have been compiled in this report to illustrate some of the research in earthquake engineering being conducted at the university. The research described in the papers has been sponsored by several agencies including the following: National Science Foundation, National Bureau of Standards, and the Alyeska Pipeline Service Company.

Characteristics of three-dimensional ground motions along principal axes, San Fernando earthquake, Kubo, T. and Penzien, J.—*Earthquake losses as a function of construction types,* Steinbrugge, K. V., Lagorio, H. J. and Algermissen, S. T.—*Seismic risk analysis for a metropolitan area,* Oliveira, C. S.—*Seismic design regionalization maps for the United States,* Whitman, R. V. *et al.*—*Nonlinear response spectra for probabilistic seismic design of reinforced concrete structures,* Murakami, M. and Penzien, J.—*General purpose computer program for dynamic nonlinear analysis,* Mondkar, D. P. and Powell, G. H.—*Earthquake response of a class of torsionally coupled buildings,* Kan, C. L. and Chopra, A. K.—*Energy absorbing devices in structures under earthquake loading,* Kelly, J. M. and Tsztoo, D. F.—*Evaluation of methods for earthquake analysis of structure-soil interaction,* Gutierrez, J. A. and Chopra, A. K.—*Problems in prescribing reliable design earthquakes,* Bertero, V. V., Mahin, S. A. and Herrera, R. A.—*Seismic design implications of hysteretic behaviour of reinforced concrete structural walls,* Bertero, V. V. *et al.*—*Infilled frames in aseismic construction,* Klingner, R. E. and Bertero, V. V.—*On seismic design of R/C interior joints of frames,* Popov, E. P. *et al.*—*Seismic design and analysis provisions for the*

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United States, Newmark, N. M. *et al.—Emergency post earthquake inspection and evaluation of damage in buildings,* Bresler, B., Graham, L. and Sharpe, R.—*A nonlinear seismic design procedure for nuclear facilities,* Kamil, H. and Bertero, V. V.—*Dynamic behavior of an eleven story masonry building,* Stephen, R. M. and Bouwkamp, J. G.—*Mathematical modeling of a steel frame structure,* Tang, D. T. and Clough, R. W.—*Evaluation of contribution of floor system to dynamic characteristics of moment resisting space frames,* Edgar, L. and Bertero, V. V.—*Inelastic cyclic behavior of reinforced concrete flexural members,* Atalay, B. and Penzien, J.—*Cyclic shear tests on masonry piers,* Mayes, R. L., Omote, Y. and Clough, R. W.—*Effect of test technique on masonry shear strength,* Omote, Y. *et al.—Test of the model of joint between floorslab and shear walls of a precast multistory building made of prestressed concrete,* Petrovic, B. and Bouwkamp, J.—*Analysis of earthquake effects on pipelines,* Powell, G. H.—*Failure criteria,* Bertero, V. V.—*Dynamic tests on structures,* Penzien, J.

● **1.2-7 Eight papers published in Proceedings of the 6th World Conference on Earthquake Engineering,** *UILU-ENG-77-2001, Structural Research Series 436,* Dept. of Civil Engineering, Univ. of Illinois, Urbana, Jan. 1977, 49.

This report is a collection of papers delivered by members of the faculty of the Dept. of Civil Engineering at the Univ. of Illinois to the Sixth World Conference on Earthquake Engineering, held in New Delhi, Jan. 10–14, 1977. Here follow the paper titles and authors' names: *Risk-consistent earthquake response spectra,* Der-Kiureghian, A. and Ang, A. H.-S.—*Simulation of strong earthquake motions for inelastic structural response,* Kameda, H. and Ang, A. H.-S.—*Seismic design and analysis provisions for the United States,* Newmark, N. M. *et al.—Comparison of building response and free field motion in earthquakes,* Newmark, N. M., Hall, W. J. and Morgan, J. R.—*Safety of reinforced concrete buildings to earthquakes,* Portillo, M. and Ang, A. H.-S.—*Extended applications of response spectra curves in seismic design of structures,* Singh, M. P., Singh, S. and Ang, A. H.-S.—*Seismic force distributions for computation of shears and overturning in buildings,* Smilowitz, R. and Newmark, N. M.—*Computed behavior of coupled shear walls,* Takayanagi, T. and Schnobrich, W. C.

● **1.2-8 Seismic '77, UCLA contributions to the Sixth World Conference on Earthquake Engineering,** *UCLA-ENG-7683,* School of Engineering and Applied Science, Univ. of California, Los Angeles, Jan. 1977, 56.

This report is a collection of papers delivered by members of the faculty of the School of Engineering and Applied Science at UCLA to the Sixth World Conference on Earthquake Engineering, held in New Delhi, Jan. 10–14, 1977. Here follow the paper titles and authors' names: *Design earthquakes based on the statistics of source, path and site effects,* Campbell, K. W.—*Effects of site on ground motion in the San Fernando earthquake,* Duke, C. M. *et al.—Seismicity and site effects on earthquake risk,* Eguchi, R. T. and Campbell, K. W.—*The influence of source parameters on strong ground motion,* Levy, N. A. and Mal, A. K.—*Separation of body and surface waves in strong ground motion records,* Liang, G. C. and Duke, C. M.—*Transfer functions for surface waves,* Mal, A. K. and Duke, C. M.—*Short period surface waves in a layered medium,* Nemani, D. and Mal, A. K.—*Collapse analysis of multistory buildings,* Selna, L. G.—*Soil liquefaction in cyclic cubic test apparatus,* Wolfe, W. E., Annaki, M. and Lee, K. L.

● **1.2-9 Proceedings, Sixth World Conference on Earthquake Engineering,** Sarita Prakashan, Meerut, India, 1977, 3 vols., 3389.

The conference was held in New Delhi from Jan. 10–14, 1977, and was sponsored by the Indian Society of Earthquake Technology and eleven other organizations. Volume I lists the sponsoring organizations, the participants, and the contents of the three volumes. Volume I also contains an author index for all volumes. Each of the three volumes presents panel and technical papers, summaries, and discussions.

The following is a list of theme report and panel and technical paper titles and authors' names. None of the theme reports or panel papers is individually cited in this volume of the *AJEE.* An asterisk following a technical paper title indicates that only a summary was presented in the proceedings; these summaries are not individually cited in this volume of the Abstract Journal. All other technical papers are either cited or abstracted in this volume of the journal.

Volume I

Panel Papers and Discussions: Panel 1—Earthquakes: *Remarks of the chairman,** Duda, S. J.—*1.1 Earthquake mechanism: preparatory phase and rupture,* Wyss, M.—*1.3 Reliability of records,* Hudson, D. E.—*1.4 Microzoning: models and reality,* Esteva, L. Panel 2—Behaviour as Related to Design Criteria: *2.1 Behaviour as related to design: soils,* Whitman, R. V.—*2.2 Behaviour as related to design criteria: materials,* Sozen, M. A.—*2.3 Members: behaviour as related to design criteria,* Park, R.—*2.4 Behaviour of systems,* Wakabayashi, M. Panel 3—Design and Engineering Decisions: *3.1 Failure criteria (limit states),* Bertero, V. V. and Bresler, B.—*3.2 Consequences of failure (including human reaction),* Lind, N. C.—*3.3 Optimization: the only rational way or only a rationalistic way?,* Cornell, C. A.—*3.4 Codes,* Arya, A. S. Panel 4—Earthquake Parameters for Design of Major Projects: *4.1 Earthquake magnitude as characteristic of spectral features of the seismic source,* Duda, S. J.—*4.2 Forecasting the spectral amplitudes of strong earthquake ground motion,* Trifunac, M. D.—*4.3 Design spectra,* Chandrasekaran, A. R.—*4.4*

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Allowable stresses and earthquake performance, Blume, J. A.

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Topic 8—Earthquake Resistant Design of Nuclear Power Facilities: *Theme report,** Ramachandran, V.—*Probabilistic dynamic earthquake analysis of nuclear power plants,* Ermutlu, H. E.—*Approach to probability evaluation of design earthquakes for nuclear plants,* Patwardhan, A. S., Cluff, L. S. and Tocher, D.—*Dynamic structure-medium interaction of underground nuclear reactor containments,* Reddy, D. V., Moselhi, O. E. and Sheha, S. A. E.—*Effects of site conditions on floor response spectra,* Kunar, R. R. and Beresford, P. J.—*Artificial earthquake generation for nuclear power plant design,* King, A. C. Y. and Chen, C.—*Two dimensional vibration test for a vertical slice model of HTGR core,* Muto, K. and Kuroda, T.—*Overturning behavior of nuclear power plant structures during earthquakes,* Dalal, J. S. and Perumalswami, P. R.—*A nonlinear seismic design procedure for nuclear facilities,* Kamil, H. and Bertero, V. V.—*Elastic-plastic dynamic analysis of reactor buildings,* Umemura, H. and Tanaka, H.—*Seismic analysis of a complex turbo-generator building including torsion,* Arya, A. S., Chandra, B. and Gupta, S. P.—*Seismic evaluation of critical facilities at the Lawrence Livermore Laboratory,* Murray, R. C. and Tokarz, F. J.—*Seismic effects on modularized spent fuel storage facilities,* Dong, R. G. and Tokarz, F. J.—*Seismic analysis of reactor internals of Narora atomic power plant, India,* Arya, A. S. *et al.*—*Seismic response of floating nuclear power plant equipment (turbine-generator on steel foundation),* Aneja, I. K.—*Seismic analysis of reactor building of an atomic power plant on alluvial soil,* Arya, A. S. *et al.*—*Dynamic behaviour of embedded structures,** Chandrasekaran, A. R. and Paul, D. K.—*Earthquake safety of nuclear power plants-an interpretive review of current design practice and the related regulatory system in West Germany,** Bork, M. and Neu, K.—*Comparison of site dependent and regulatory agency earthquake input motion used in the design of nuclear power plant,** Fischer, J. A., Peir, J. C. and Lu, B. T. D.—*Design evaluation of reactor components for seismic disturbances,** Shrivastava, S. K., Kumar, S. N. and Patwardhan, V. M.

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The solution of seismic stability problems on small-scale models by optic methods, Belogorodsky, B. A., Malyshev, L. K. and Panteleev, A. A.—*Principles of physical modelling of structures resistant to earthquakes,* Monahenko, D. V.—*Behaviour of structures during earthquakes beyond the limits of the elastic stage,* Korchynsky, I. L. and Aliev, G. A.—*Approximate vibration analysis of a cable stayed bridge,* Arya, A. S., Thakkar, S. K. and Rani, P.—*The study of non-linear performance of structures in frameless earthquake-proof residential buildings by means of powerful vibrators,* Ashkinadze, G. N. and Simon, Y. A.—*The research into the earthquake-proof capacity of in-situ concrete frameless residential buildings by vibro-tests of the large-size reinforced concrete model,* Barkov, Y. V. and Glina, Y. V.—*The dependence of inertial force upon displacements in the non-linear area under oscillations (basing on the data resulted from the high-capacity vibro-tests of structures),* Shapiro, G. A. and Zakharov, V. F.—*On the non-linear deformations base of earthquake-proof buildings under oscillations,* Shapiro, G. A. and Ashkinadze, G. N.—*Acousto optico method of analysis of dynamic rock structure,* Zadgaonkar, A. S. and Tarnekar, M. G.—*Properties of confined concrete for design of earthquake resistant structures,* Kaar, P. H. and Corley, W. G.—*Influence of properties and stressed state of foundation soils on absorption and diffusion of energy of seismic vibrations,* Aliev, G. A.—*Regression analysis on resonance curves of forced vibration tests and its error estimation due to microtremors,** Hiromatsu, T.—*Experimental determination of lateral stiffness characteristics of space framed three dimensional steel structures,* Basak, A. K. and Gupta, Y. P.—*The influence of partition walls on the rigidity of frame structures,** Mirabal, Y. M.—*Tests on spatial frames subjected to horizontal rotating loads until the incremental collapse,** Belli, P.—*Assessment of a multistoreyed building by vibration tests,** Srinivasulu, P., Lakshmanan, N. and Vaidyanathan, C. V.—*Seismic analysis of multistory prefabricated framed structures,** Negoita, A., Missir, I. and Scharf, F.—*Utilization of ultrasonic modelling procedure for studies of seismic processes in non-uniform rock foundations of dams,** Lipovskaya, V. Ya. and Yaryshev, B. P.—*Tilting platform for measuring earthquake resistance of small buildings,** Munski, K. D. and Keightley, W. O.—*Tests on steel beam-column joints,** Shepherd, R., Spring, K. C. F. and Mead, F. H.—*Model investigations of arch dams response on seismic effect,** Vardanashvili, T. Z. and Gutidze, P. A.—*Model investigation of tunnels seismic resistance,** Kartzivadze, G. N. and Metreveli, Gh. B.—*Proposed standard method for dynamic testing of reinforced concrete members and for presentation of test results,** Razani, R.

Topic 10—Earthquake Instrumentation: *Theme report,** Hudson, D. E.—*An evaluation of the Mexican strong motion radio telemetry network after three years of operation,* Prince, J. and Rodriguez, F. H.—*Strong motion instrument network in Yugoslavia,* Mihailov, V.—*New automatic train stopping system during earthquake,* Fujiwara, T.—*Optimal sensor location for identification of building structures using response to earthquake ground motions,* Shah, P. C. and Udwadia, F. E.—*Statistical analysis of the computed response of structural response recorders (SRR) for accelerograms recorded in the United States of America,* Trifunac, M. D.—*Review of current standards and practice for earthquake instrumentation at nuclear plants,* Pauly, S. E.—*A simple instrument for determining earthquake response spectra,* Sattaripour, A.—*Performance of multiple structural response recorders in India,* Agrawal, P. N.—*U.S. highway bridge seismic instrumentation network,** Podolny, Jr., W. and Cooper, J. D.—*Earthquake instrumentation requirements for nuclear power plants in the United States,** Mehta, D. S. and Godfrey, D. A.

Topic 11—Dynamic Behaviour of Structural Elements: *Remarks of the chairman of theme11B,** Wakabayashi, M.—*Theme report,** Sozen, M. A.—*The influence of frequency on the shear strength and ductility of masonry walls in dynamic loading tests,* Tercelj, S., Sheppard, P. and Turnsek, V.—*The influence of horizontally-placed reinforcement on the shear strength and ductility of masonry walls,* Sheppard, P., Tercelj, S. and Turnsek, V.—*Evaluation of contribution of floor system to dynamic characteristics of moment-resisting space frames,* Edgar, L. and Bertero, V. V.—*Earthquake response and damage prediction of reinforced concrete masonry multistory buildings, Part I: program definition,* Hegemier, G. A. *et al.*—*Earthquake response and damage prediction of reinforced concrete masonry multistory buildings, Part II: selected results,* Hegemier, G. A. *et al.*—*Experimental study on reinforced concrete truss frames as earthquake resistance elements,* Shimazu, T. and Fukuhara, Y.

Study on low cycle fatigue of structural frames due to randomly varying load, Mizuhata, K., Gyoten, Y. and Kitamura, H.—*Elasto-plastic earthquake response of frames with shear wall,* Kobori, T., Inoue, Y. and Kawano, N.—*Building damping by Coulomb friction,* Keightley, W. O.—*Bond deterioration in reinforced concrete members under cyclic loads,* Gosain, N. K. and Jirsa, J. O.—*Brittle matter ductilization in structures,* Cismigiu, A. I. and Dogaru, L. C.—*Inelastic cyclic behavior of reinforced concrete flexural members,* Atalay, B. and Penzien, J.—*Computed behavior of coupled shear walls,* Takayanagi, T. and Schnobrich, W. C.—*Cyclic deformation behaviour of reinforced concrete shear walls,* Yamada, M. *et al.*—*Tests and analyses of SRC beam-column subassemblages,* Aoyama, H., Umemura, H. and Minamino, H.—*Influence of lateral beams on the behavior of beam-column joints,* de Aleman, M. A., Meinheit, D. F. and Jirsa, J. O.—*Performance of large reinforced concrete beam-column joint units under cyclic loading,* Blakeley, R. W. G. *et al.*—*Seismic resistance of reinforced concrete beam-and-column assemblages with emphasis on shear failure of column,* Minami, K. and Wakabayashi, M.—*An evaluation method for the earthquake resistant capacity of reinforced concrete and steel reinforced concrete columns,* Ozaki, M. and Ishiyama, Y.—*Experimental study on reinforced concrete columns with double spiral web reinforcements,* Umemura, H. *et al.*—*Confinement effects of web reinforcements on reinforced concrete columns,* Shimazu, T. and Hirai, M.—*Influences of loading excursions on restoring force characteristics and failure modes of reinforced concrete columns,* Higashi, Y., Ohkubo, M. and Ohtsuka, M.—*Nonelastic hysteretic behaviour of RC columns and infilling walls,* Paskaleva, I.—*Structural assemblage of shear wall high-rise buildings exposed to cyclic loading,* Anicic, D. and Zamolo, M.—*Aseismic characteristics of RC box and cylinder walls,* Umemura, H. *et al.*

Ductility of reinforced concrete bridge piers, Priestley, M. J. N. *et al.*—*A simulation of earthquake response of steel buildings,* Takanashi, K., Udagawa, K. and Tanaka, H.—*An experimental study on inelastic behavior of steel members subjected repeated loading,* Suzuki, T. and Ono, T.—*Inelastic behavior of high strength steel frames subjected to constant vertical and alternating horizontal loads,* Matsui, C. and Mitani, I.—*Influence of local buckling on cyclic behavior of steel beam-columns,* Mitani, I., Makino, M. and Matsui, C.—*Cyclic behavior of a restrained steel brace under axial loading,* Wakabayashi, M., Matsui, C. and Mitani, I.—*Hysteretic behavior of steel braces subjected to horizontal load due to earthquake,* Wakabayashi, M. *et al.*—*Cyclic shear tests on masonry piers,* Mayes, R. L., Omote, Y. and Clough, R. W.—*Experimental study of prestressed reinforced concrete elements of antiseismic buildings,* Kvitsaridze, O. I. *et al.*—*Effect of test technique on masonry shear strength,* Omote, Y. *et al.*—*Behaviour of one storey reinforced concrete frame infilled with brickwork under lateral loads,* Barua, H. K. and Mallick, S. K.—*Multi-story walls subjected to simulated earthquakes,* Sozen, M. A.,

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Aristizabal, D. and Lybas, J. M.—*Some recent research in New Zealand into aspects of the seismic resistance of prestressed concrete frames,* Park, R. and Thompson, K. J.—*Effect of amount and type of reinforcement on behaviour of reinforced concrete elements under seismic impulse excitation,* Zhunusov, T. Zh. and Bespayev, A. A.—*Free vibration tests of structural walls,* Oesterle, R. G., Fiorato, A. E. and Corley, W. G.—*Performance of model reinforced concrete cores for tall buildings cycled in torsion,* Irwin, A. W. and Andrew, N.—*Shock absorbers with viscous shear resistance and its experiment,** Ishiguro, Y. *et al.*—*A method to analyze the cyclic behavior of reinforced concrete slender shear walls,** Aktan, H. M. and Hanson, R. D.—*An approach to model reinforced concrete chimneys for calculating inelastic dynamic response,** Goel, S. C. and Rumman, W. S.

*Inelastic hysteresis behavior of axially loaded steel members with rotational end restraints,** Prathuangsit, D., Goel, S. C. and Hanson, R. D.—*Behavior of framed shear walls under shear load,** Imai, H., Yokoyama, M. and Sonobe, Y.—*Response of beams to correlated random base excitation,** Masri, S. F. and Udwadia, F.—*Cyclic loading on structural assemblage of RC high-rise building with core and pendulum columns,** Zamolo, M. and Anicic, D.—*Inelastic cyclic response of split K-braced frames,** Kaldjian, M. J.—*Behavior of RC beam-column connection with large size deformed bars under cyclic loads,** Ishibashi, K., Kamimura, T. and Sonobe, Y.—*Behaviour of joints in prefabricated shear walls for seismic zones,** Santhakumar, A. R., Swamidurai, A. and Lakshmipathy, M.—*Test of the model of joint between floor slab and shear walls of a precast multistory building made of prestressed concrete,** Petrovic, B. and Bouwkamp, J.—*Ductility of bent members of prestressed expanded clay concrete,** Negoita, A., Dumitras, M. and Negoita, I.—*Vibration characteristics of cable systems,** Krishna, P., Chandra, B. and Gupta, V. K.—*Effects of straining rate on deformation and fracture of reinforced concrete members,** Takeda, J., Tachikawa, H. and Fujimoto, K.—*Low cycle fatigue damage of reinforced concrete members,** Muller, F. P. and Henseleit, O.—*Studies on dynamic behaviour and earthquake resistance of bridge piers,** Ozaka, Y., Yanagida, T. and Kodera, J.—*Design of anchored bars for seismic loadings,** Jirsa, J. O., Burgieres, Jr., S. T. and Peyton, R. L.—*Inelastic analysis of connecting beams in coupled shear-walls under cyclic loading,** Agrawal, A. B., Mufti, A. A. and Jaeger, L. G.—*Shear strength of reinforced concrete columns,** Tanaka, Y. *et al.*—*Hysteretic behaviors of reinforced concrete members subjected to bi-axial bending moments,** Takiguchi, K. and Kokusho, S.—*Shear requirements for load reversals on reinforced concrete members,** Gosain, N. K., Brown, R. H. and Jirsa, J. O.—*A simple continuum model for dynamic analysis of complex plane frame structures,** Sun, C. T. *et al.*

Topic 12—Earthquake Resistant Design of Equipment and Service Facilities: *Theme report,** Vanmarcke, E. H.—*Provisions for seismic design of non-structural building components and systems,* Goldberg, A. and Sharpe, R. L.—*On a simple method estimating the appended system response spectrum from a statistically simulated spectrum,* Sato, H. *et al.*—*On the decoupling of secondary systems for seismic analysis,* Hadjian, A. H.—*Predicting the earthquake response of resiliently mounted equipment with motion limiting constraints,* Iwan, W. D.—*Computer simulation of lifeline response to earthquakes,* Schiff, A. J., Feil, P. J. and Newson, D. E.—*Earthquake protection of communications facilities,* Liu, S. C.—*Telecommunications equipment seismic effect study,* Sugimoto, Y. and Sato, Y.—*Earthquake resistant installation device of computers,* Muto, K., Nagata, M. and Fukuzawa, E.—*A simple procedure for predicting amplified response spectra and equipment response,* Vanmarcke, E. H.—*Aseismic tests on some electro-mechanical systems,* Guha, S. K., Wedpathak, A. V. and Desai, P. J.—*Elevator earthquake safety control,* Benuska, K. L., Aroni, S. and Schroll, W.—*Deformation of railway track and running stability of train in earthquake,* Sato, Y. and Miura, S.—*The way of setting the aseismic design code of oil refineries and petro-chemical industries,* Shibata, H.

Aseismic design of cylindrical and spherical storages for their sloshing phenomenon, Sogabe, K. and Shibata, H.—*Analysis of earthquake effects on pipelines,* Powell, G. H.—*Quantitative analysis of seismic damage to buried utility pipelines,* Katayama, T., Kubo, K. and Sato, N.—*Earthquake resistant design of underground pipelines,* Takada, S.—*Equipment systems in the seismic environment,** Merz, K. L.—*Forced vibration analysis of 230KV air blast circuit breakers: energy dissipation parameters calculated for different excitation levels,** Kircher, C. A.—*Effect of joint parts on aseismic strength of buried pipeline,** Goto, H., Takada, S. and Nagao, S.—*Separation of category I pipelines in granular soil,** Aggour, M. S. and Miller, R. P.—*Earthquake resistance of porcelain insulator electric power equipment,** Hashimoto, K. and Tsutsumi, H.—*Locating and designing facilities in seismically active areas,** Nair, K. and Kulkarni, R. B.—*An active device to protect nuclear particle accelerators from earthquakes,** Peyrot, A. H. and Plautz, K. A.—*Earthquake resistant design of steel towers, of electrical high voltage transmission lines subjected to horizontal or vertical ground motions,** Syrmakezis, C. A.

● **1.2-10 Intergovernmental conference on the assessment and mitigation of earthquake risk, Paris 10-19 February 1976,** *SC/MD/53,* United Nations Educational, Scientific and Cultural Organization (UNESCO), Paris, May 1976, 50.

The objective of the conference was to review developments in seismology, earthquake engineering, and related subjects. This publication lists participants and summarizes

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topics of discussion, including recent advances in research, current problems, objectives, assessment of earthquake risks, engineering measures for loss reduction, and implications of earthquake risk. A complete list of papers presented at and resolutions made by the conference is given.

● **1.2-11 Proceedings of the International Meeting on the Friuli earthquake,** *Bollettino di Geofisica,* XIX, 72, Dec. 1976, 2 parts, 1626.

The meeting was held in Udine December 4-5, 1976. The following is a list of titles pertinent to earthquake engineering. None of these papers is abstracted in this volume of the *AJEE.*

Section1—Seismology, Geophysics and Geology: *Geodynamic outline and seismicity of Friuli-Venetia Julia region,* Amato, A. *et al.—Engineering seismology aspects of the Gemona-Friuli earthquake,* Ambraseys, N. N.—*Crustal structure and seismicity of northern Italy,* Aric, K. *et al.—The relation between seismological and neotectonic characteristics of Friuli and neighbouring areas,* Arsovski, M. *et al.—Earthquake catalogue of Friuli-Venezia Giulia region (from Roman epoch up to 1976),* Bartole, R. *et al.—Strong motion records relative to the Friuli earthquake, accelerogram processing and analysis,* Basili, M. *et al.—Macroseismic aspects and seismological and statistical considerations,* Basili, M., Jaccarino, E. and Tenaglia, G.—*Note on the seismicity of northeastern Italy (Friuli area),* van Bemmelen, R. W.—*Seismicity of Friuli-Venezia Giulia region,* Bernardis, G. *et al.—Stress corrosion theory of crack propagation applied to the earthquake mechanism. A possible basis to explain the occurrence of two or more large seismic shocks in a geologically short time interval,* Bonafede, M. *et al.—Horizontal pendulum observations at Trieste,* Chiaruttini, C. and Zadro, M.—*Epicenter distribution and analysis of 1976 earthquakes and aftershocks of Friuli,* Colautti, D. *et al.—Preliminary spectral analysis of aftershocks of the 1976 Friuli earthquakes,* Finetti, I., Frinzi, U. and Scotti, A.—*An example of microseismic zoning in the Madonna di Buia area,* Giorgetti, F.—*Isoseismal map of the May 6, 1976 Friuli earthquake,* Giorgetti, F.—*Macroseismic investigations and maps for the Friuli earthquake May 6, 1976,* Graf, R. and Mayer-Rosa, D.—*Topographical effect on the distribution of earthquake damage,* Hakuno, M.—*Macroseismic effects of the Friuli earthquake of May 6, 1976 in Austria, Czechoslovakia, F.R.G., France, Poland and Yugoslavia,* Karnik, V. *et al.—Research on the relationships between surface geology and seismically active tectonic structures,* Magri, G. and Mittempergher, M.—*Response spectra analysis for different sites in Friuli,* Maistrello, B. *et al.—The Friuli earthquake, May 6, 1976: geology,* Martinis, B., ed.—*Geotechnical aspects on the recent Friuli earthquake,* Maugeri, M.—*Aftershock-statistics and fault-plane solutions of Friuli earthquakes 1975 and 1976,* Mayer-Rosa, D., Pavoni, N., and Graf, R.—*Strong-motion records of the May 6th and September 15th, 1976 Friuli earthquakes obtained by the Yugoslav strong-motion instrument network,* Mihailov, V.—*May 6, 1976 Friuli earthquake: field measurements,* Papastamatiou, D.—*Preliminary analysis of Friuli earthquake records,* Ritsema, A. R.—*Preliminary report on the recording of some aftershocks along the profile Gemona-Trieste,* Scarascia, S. *et al.—Liquefaction of sands in Friuli during May 6 and September 15, 1976 earthquakes,* Siro, L.—*Vp/Vs ratio and implications for hypocenter determinations of aftershocks of the Friuli earthquake, May 6th, 1976,* Schmedes, E., Miller, H. and Gebrande, H.—*Crustal deformation after the Friuli earthquake, 1976,* Tokuyama, A.—*The level of the macroseismic field for the territory of Friuli,* Vukashinovitch, M.—*The soil acceleration excited by the earthquake of May 6, 1976,* Vukashinovitch, M.—*Seismic shakeability and seismic risk for the territory of Udine (Italy) and vicinity,* Vukashinovitch, M.—*The estimation of the degree of building's damage according to new seismic MSK-64 scale,* Vukashinovitch, M. and Nedeljkovitch, S.—*Geological emergency interventions in the earthquake stricken zones of Friuli: the example of Osoppo town,* Bramati, A. *et al.*

Section 2—Seismic Engineering: *Lateral force analysis of a simple reinforced concrete structure damaged in May 6th, 1976 Friuli earthquake,* Bayulke, N.—*Ductility of r.c. buildings during Friuli earthquake,* Benedetti, D. and Vitiello, E.—*Implicit design loads of Friuli r.c. damaged buildings,* Benedetti, D. and Vitiello, E.—*Behaviour of some simple structures during the Friuli earthquake,* Bo, G. M. *et al.—Use of a modelling approach in the analysis of the effects of repairs to earthquake-damaged stone-masonry buildings,* Bostjancic, J. *et al.—Elastic-plastic degrading oscillators behaviour under earthquakes - preliminary results,* Braga, F. and Parducci, A.—*Dynamic investigation of the effects on non-structural elements on a reinforced concrete building in Friuli,* Brancaleoni, F. and Petrangeli, M. P.—*Damage to bridges and roads in the 1976 Friuli (Italy) earthquake,* Briseghella, L.—*Earthquake in Friuli (Italy)-1976. Damage to historical monuments and other buildings of artistic interest,* Briseghella, L. *et al.—Recently constructed masonry buildings,* Cartapati, E. and Cherubini, A.—*Report on damages and provisions attaining to earthquake concerning the electrical system,* Faoro, G. G. and Ferrazza, G.—*Collapse of a nonseismic resistant r.c. building during the Friuli earthquake: the "ASTRO" building in Maiano,* Giuffre, A., Menegotto, M. and Pinto, P. E.—*Assessment and interpretation of building damage,* Glauser, E. C., Heimgartner, E. and Sagesser, R.—*Behaviour of damaged buildings under strong earthquakes,* Heimgartner, E.—*Building damage in Friuli earthquake 1976 - buildings in Osoppo,* Izumi, M.—*On the inelastic behavior of structures during earthquakes as exemplified by the Friuli catastrophe 1976,* Olszak, W.—*On the behaviour*

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of a r.c. building with suspended floors during the Friuli earthquake on May 1976, Petrini, V. *et al.—Dynamic analysis of Friuli r.c. damaged buildings,* Petrini, V. and Vitiello, E.—*Notes on old masonry buildings structural behaviour - proposal of investigation,* Pistone, G. and Roccati, R.

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2. Selected Topics in Seismology

2.1 Seismic Geology

2.1-1 Kelleher, J. and McCann, W., **Buoyant zones, great earthquakes, and unstable boundaries of subduction,** *Journal of Geophysical Research,* 81, *26,* Sept. 10, 1976, 4885–4896.

The distribution of large shallow earthquakes along subduction boundaries does not agree with the distribution pattern that might be predicted from a simple model of plate tectonics; that is, along extensive sections of some island arcs, large shocks occurred infrequently or not at all during recorded history. Most of these zones of long-term quiescence are nearly coterminous with segments of the margin where zones of seamounts, aseismic ridges, or other bathymetric highs of the underthrust slab appear to be interacting with the subduction process. This spatial correlation suggests that at least some of the long-term absences of great shocks may result from a tectonic origin and not from temporary intervals of strain accumulation. The zones where rises interact with active trenches are also characterized in many instances by a near absence of low-angle thrust-type mechanisms, by gaps in intermediate depth hypocenters, and by gaps and offsets in the line of active volcanoes. Thus major departures from classic subduction activity may develop where significant bathymetric features interact with a convergent margin. To explain these observations, the authors favor the hypothesis that aseismic ridges or other uplifted regions may delineate zones of oceanic lithosphere which are relatively buoyant and resist subduction upon collision with an active trench. In place of "typical" oceanic lithosphere, therefore, there may exist a broad spectrum of average densities for oceanic lithosphere; and the relative buoyancy of both the underthrusting and the overthrusting slabs near the subducting margin may be a dominant influence in the development of subduction tectonics and in the locations and frequency of great earthquakes.

2.1-2 Prescott, W. H. and Savage, J. C., **Strain accumulation on the San Andreas fault near Palmdale, California,** *Journal of Geophysical Research,* 81, *26,* Sept. 10, 1976, 4901–4908.

Precise distance measurements of a 10 X 25 km 15-station trilateration network that spans the San Andreas fault west of Palmdale, California, have been repeated annually in the period 1971–1975. The network appears to be deforming under simple uniform tensor shear of about 0.21 ± 0.03 μstrain/yr with the direction of maximum right-lateral shear parallel to the local strike of the San Andreas fault. Comparison of trilateration with triangulation surveys of the same network shows that the rate of strain accumulation has been constant over the past 40 yr. The strain accumulation can be explained by conventional dislocation models (i.e., slip at depth beneath a locked section) of the San Andreas fault with 30- to 50-mm/yr slip. Leveling surveys along a 16-km line that crosses the fault at Palmdale indicate significant changes in tilt but with frequent reversals, so that no net tilt has accumulated in the overall period 1935–1975.

2.1-3 Chapman, M. E. and Solomon, S. C., **North American-Eurasian plate boundary in northeast Asia,** *Journal of Geophysical Research,* 81, *5,* Feb. 10, 1976, 921–930.

The intracontinental portion of the boundary between the North American and Eurasian plates can be identified on the basis of seismicity, recent tectonics, and earthquake focal mechanisms. The simplest plate geometry that can explain these data involves a North American-Eurasian boundary that extends from the Nansen ridge through a broad zone of deformation in northeast Asia to the Sea of Okhotsk and thence southward through Sakhalin and Hokkaido to a triple junction in the Kuril-Japan trench. Such a configuration can account quantitatively for the slip vectors derived from earthquake mechanisms in Sakhalin and Hokkaido. On the basis of new slip vector data, the North

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American-Eurasian angular velocity vector is revised only slightly from previous determinations. The intracontinental plate boundary is diffuse and may be controlled by ancient plate sutures. Deformation within about 10° of the rotation pole, which lies very near the boundary, cannot be modeled by rigid plate tectonics. These characteristics of intracontinental plate boundaries are related to the greater thickness and heterogeneity of continental lithosphere and to the influence of continents on the plate tectonic driving forces.

● **2.1-4** Fife, D. L. *et al.*, **Geologic hazards in southwestern San Bernardino County, California,** *Special Report 113,* California Div. of Mines and Geology, Sacramento, 1976, 40.

The study area encompasses 550 sq mi (1400 km^2) of the Upper Santa Ana Valley in southwestern San Bernardino County and has a population of 600,000. Twelve maps of the area are included with this report. The active Cucamonga, San Jacinto, San Andreas, and Elsinore faults are within the region. Based on well data and magnetic measurements, a principal trace of the Elsinore fault is believed to extend northwesterly, east of the Puente Hills. The San Jacinto fault, which was not considered during early land development, presents the greatest hazard from ground rupture in the urbanized San Bernardino area. The San Andreas fault, until this year (1974), has been similarly disregarded. Ground shaking, resulting from fault movement, is a serious seismic hazard in areas of widespread alluvial sediments. The thickness of alluvial sediments averages about 800 ft (240 m) in the valley area with maximum thicknesses of about 1300 ft (396 m) near Ontario and San Bernardino. The surface effects of shaking will probably be greatest in areas of ground-water depletion and subsidence near San Bernardino and Chino. Depletion of ground water and draining of swamps during the first half of this century have largely eliminated the liquefaction hazard; however, urban irrigation maintains perched water tables, which may present local liquefaction hazards.

The region experienced major seismic events in 1769, 1812, and 1857; and five strong earthquakes near magnitude 6 or greater were experienced in the region between 1890 and 1923. Landslide-prone areas within the National Forest along the mountain front of the Transverse Ranges present hazards to dams, reservoirs, roads, railroads, and utilities. The Puente Hills, which are underlain by Tertiary marine sedimentary rocks, are considered the most hazardous landslide-prone area.

● **2.1-5** Slemmons, D. B. and McKinney, R., **Definition of "active fault,"** *Misc. Paper S-77-8,* Soils and Pavements Lab., U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, May 1977, 22.

The term "active fault" has had a long, well-established usage among engineering geologists and seismologists. Despite many shades of meaning in the definitions, the term signifies offsets during geologically recent time, association with earthquakes, and probability or potential for recurrence of these events. The term would benefit from a commonly accepted definition. "Capable fault" has a more restricted meaning as defined by the Nuclear Regulatory Commission.

2.1-6 Stauder, W. and Mualchin, L., **Fault motion in the larger earthquakes of the Kurile-Kamchatka arc and of the Kurile-Hokkaido corner,** *Journal of Geophysical Research,* **81,** *2,* Jan. 10, 1976, 297–308.

The focal mechanisms of 120 large earthquakes (m_b>5.7) of the Kurile-Kamchatka arc for the period 1962–1973 illustrate particular features of the subduction of the Pacific plate beneath this island arc system. The faulting characteristic of normal-depth earthquakes is consistent with convergence of the oceanic plate in the direction N60°W with respect to the continental plate. Shallower foci under the trench are tensional; those seaward of the trench and related to the Hokkaido rise are compressional. Intermediate-depth foci separate into those which represent either axial compression with respect to the plate or axial tension. The occurrence of the two types in close spatial proximity may possibly correspond to differences in the orientation of principal stresses due to volume change across the olivine-spinel phase transition boundary. In the northern Kuriles, deep-focus earthquakes are compressional with respect to the axis of the descending plate. In the vicinity of the Hokkaido corner, east-west striking normal faults and northward dipping tension axes in intermediate- and deep-focus earthquakes imply hinge faulting and contortion of the Pacific plate between the more moderately dipping Japanese segment of the plate and the steeper dipping Kurile Islands segment. Recurrent Tokachi-oki earthquakes seaward of the Hokkaido corner indicate that the direction of convergence of the Pacific plate is unaffected by the presence of the junction of the two island arc systems.

● **2.1-7** Prescott, W. H. and Lisowski, M., **Deformation at Middleton Island, Alaska, during the decade after the Alaska earthquake of 1964,** *Bulletin of the Seismological Society of America,* **67,** *3,* June 1977, 579–586.

Comparison of precise leveling done on Middleton Island in the Gulf of Alaska in 1966, 1974, and 1975 indicates that between 1974 and 1975 the island was tilting down to the northwest at a rate of 5.0 ± 0.8 μrad/yr. The direction of tilt is normal to the axis of the Aleutian trench 60 km to the southeast. The data are inadequate to determine variations in the rate since the earthquake, but suggest that it has been higher. A simple model for slip on

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the near-surface projection of the fault surface that ruptured in 1964 suggests that on the order of 2 1/2 m/yr of slip would be required to account for the observed tilt.

● **2.1-8** Keaton, J. R. and Keaton, R. T., **Manix fault zone, San Bernardino County, California,** *California Geology,* 30, *8,* Aug. 1977, 177–186.

An earthquake of magnitude 6.2, centered about 25 miles east of Barstow, California, occurred on Apr. 10, 1947. An aftershock measuring magnitude 5.2 followed five minutes later. The earthquake was accompanied by two inches of left-lateral displacement along a two-mile stretch of fault which is part of a series of faults known as the Manix fault zone. The fault zone is located in a small area in the east-central part of the Mojave Desert along the northern boundary of the Cady Mountains. Geologic characteristics of the area are discussed with primary emphasis placed on faulting.

● **2.1-9** Vyskocil, P., **Global recent crustal movements as determined by geodetic measurements,** *Tectonophysics,* 38, *1-2,* Mar. 2, 1977, 49–59.

The main problems of geodetic determination of recent crustal movement and their characteristics are presented. Horizontal movements are discussed in light of plate tectonics; vertical crustal movements are discussed in connection with heat flow and the deep structure of the earth's crust.

● **2.1-10** Sacks, I. S., **Interrelationships between volcanism, seismicity, and anelasticity in western South America,** *Tectonophysics,* 37, *1-3,* Jan. 5, 1977, 131–139.

Anelasticity studies of the upper mantle beneath South America show that, while the thickness of the continental lithosphere is generally in excess of 300 km, there are differences in the interaction between this lithosphere and the downgoing oceanic plate underthrusting the west coast. In the Chile–southern Peru region, there is asthenospheric material between the continental and oceanic lithospheres. The seismicity is mainly confined to the subducting slab, and volcanoes occur in the same relative position to this seismicity as in other subduction regions such as Japan. In central Peru, however, the asthenosphere seems to be absent, the seismicity is dispersed, and there are no volcanoes. This suggests that the contact of hot and weak material (asthenospheric) with the subducting plate is necessary for the release of magma from it.

● **2.1-11** Hedervari, P. and Papp, Z., **Seismicity maps of the New Guinea-Solomon Islands region,** *Tectonophysics,* 42, *2-4,* Oct. 20, 1977, 261–281.

Using the data of all tectonic earthquakes with $M \geq 7.0$, which occurred between Jan. 1, 1900, and Dec. 31, 1972, between 130° and 163°E and 0° and 12°S, maps for the distribution of tectonic flux and epicenters were constructed. These were compared with the tectonic map of the area investigated (Weber deep, New Guinea, Bismarck Sea, New Ireland, New Britain, Solomon Sea, Bougainville Island, Solomon Islands). It was found that the so-called active centers of the tectonic flux are arranged either exactly on, or very near, the plate boundaries established by other authors. The map of epicentral distribution shows a less clear correlation of this kind. The pattern of distribution of shallow (h = 0–99 km) shocks is rather diffuse on New Guinea itself, but less diffuse over oceanic areas where many of the epicenters are to be found near or just on the suggested boundaries of lithospheric plates. The intermediate and deep shocks ($h \geq 100$ km) form three, well-separated regions. The first is related to the Weber deep; the second, to the westernmost edge of the south Bismarck plate and to the Solomon plate; the third, to the easternmost corners of the two Bismarck plates and to the northeastern edge of the Solomon plate.

The data of 152 shocks were applied; this number appeared to be small for the construction of Benioff zones. Because of the research carried out by other authors, it is known, however, that Benioff zones do exist in some parts of the area investigated. The deepest shock in the magnitude range of 7.0–8.6 originated at 430 km beneath the surface. The distribution of shocks down to this depth by and large corresponds to the pattern which is often experienced at and underneath typical island arcs, with an outstanding maximum between 0 and 99 km. This statement is valid for the number and summarized energy of the earthquakes.

The distribution of volcanoes is unusual since many of them are situated either exactly on or very near the plate boundaries. In general, for the less-complicated island arcs, such as the Indonesian arc, the volcanoes are usually at a distance of about 280–300 km or more from the axis of the oceanic trench which is the nearest plate boundary measured from them. The area investigated in this paper is one of the most complicated regions of the world.

● **2.1-12** Smith, R. B., **Intraplate tectonics of the western North American plate,** *Tectonophysics,* 37, *4,* Feb. 2, 1977, 323–336.

Fault-plane solutions, Cenozoic geology, and in-situ stress measurements are used to infer contemporary extension between subplates of the western North American plate. Intraplate northwest extension is accommodated by strike-slip and oblique normal faulting along present-day seismic zones. Cenozoic volcanism, facilitated by regional extension above a subducting plate, may have spread radially outward from the northern Great Basin to form a

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continental triple junction with the principal arms: the central Great Basin, the Snake River Plain, and a southeast-trending zone of rhyolite domes in the southern Columbia Plateau.

● 2.1-13 Gupta, R. P., **Delineation of active faulting and some tectonic interpretations in the Munich-Milan section of the eastern Alps–use of LANDSAT-1 and 2 imagery,** *Tectonophysics,* **38,** *3-4,* Mar. 23, 1977, 297–315.

Based on studies of images obtained from LANDSAT-1 and 2, several seemingly active movement zones have been delineated in a section of the eastern Alps and are being reported in the present paper for the first time. These zones, trending W-E to NW-SE, cut across all earlier Alpine boundaries and contacts and on either side along their length are marked with drag effects, indicating their post-Alpine neotectonic nature. Their relation with the present-day central European stress field, as determined from fault-plane studies and in-situ stress measurements, has been sought. In conjunction with the evidence from neighboring areas, a dextral shear tendency of the present-day Mediterranean is indicated. Further, a number of extensive lineaments have been observed in the Alpine section. Statistically, there are three major lineation sets trending N45°, N15°, N345°. They appear to have developed cogenetically as a result of shear and tensile failures due to a stress field with maximum principal stress oriented averagely at N15°. This direction of the maximum principal stress, deduced from the above lineation analysis of the eastern Alps, is in striking conformity with the one believed to have been in existence for the development of the Rhine graben (N20°). It appears that the Rhine graben and the Alpide belt have evolved cogenetically and concurrently under the same dominant stress field and hence the two geotectonic features are really not antagonistic and mutually incompatible as usually believed on the grounds that one involves tension (taphrogenesis—Rhine graben) and the other compression (orogenesis—Alpide belt) but are different manifestations of the same stress field. Also, some additional light has been thrown on the possible controls of development of the Giudicaria line and the cause of predominance of NE-SW trending sinistral faults.

● **2.1-14** Berg, E., Sutton, G. H. and Walker, D. A., **Dynamic interaction of seismic activity along rising and sinking edges of plate boundaries,** *Tectonophysics,* **39,** *4,* May 11, 1977, 559–578.

Cumulative and differential cumulative seismic strain release of shallow earthquakes during 1964 through 1972 show significant time-delayed correlation for many of the rises and/or sinks of tectonic plate boundaries for the Atlantic, Indian, and Pacific oceans. For the Pacific, time delays are almost exclusively from areas of higher to areas of lower strain release, and could imply viscous stress relaxation along or near the lithosphere-asthenosphere boundary. Seismically determined slip rates for many of the subduction zones during 1964–1972 are lower than those of the preceding part of the century by a factor of 5. The significant correlation between many rises and sinks suggests worldwide interaction and coupling of plate motion.

● **2.1-15** Abe, K., **Tectonic implications of the large Shioya-oki earthquakes of 1938,** *Tectonophysics,* **41,** *4,* Aug. 31, 1977, 269–289.

The tectonic processes taking place along the southern part of the Japan trench are discussed on the basis of the focal mechanism of the 1938 Shioya-oki event, which consists of the five large earthquakes of M_s = 7.4, 7.7, 7.8, 7.7, and 7.1. Detailed analyses of seismic waves and tsunamis are made for each of these earthquakes, and the dislocation parameters are obtained. The total seismic moment amounts to 2.3 x 10^{28} dyne cm. The five earthquakes are grouped into either a low-angle thrust type or a nearly vertical normal-fault type. These mechanisms are common with other great earthquakes of the northwestern Pacific belt, and can be explained in terms of the interaction between the oceanic and continental plates. The vertical displacement inferred from the seismic results is in approximate agreement with the precise level data over the period from 1897 and 1939. This agreement suggests that the rate of the strain accumulation at the preseismic time is very small in the epicentral area. Repeated levelings at the postseismic time reveal a large-scale recovery of the coseismic subsidence. The postseismic deformation is one-third to one-half of the coseismic displacement. The time constant of the recovery is estimated to be 5 years or less. This type of deformation may be a manifestation of viscoelasticity of a weak zone underlying the continent. The amount of dislocation, together with the long-term seismicity, suggests a seismic slip rate of about 0.4 cm/yr, which is one order of magnitude smaller than that for the adjacent regions. This suggests that a large part of the plate motion is taking place aseismically in this region. The tectonic process now taking place in the southern Japan trench can be considered to represent a stage just prior to a complete detachment of the sinking portion of the oceanic plate.

● **2.1-16** Bennett, J. H., Taylor, G. C. and Toppozada, T. R., **Crustal movement in the northern Sierra Nevada,** *California Geology,* **30,** *3,* Mar. 1977, 51–57.

Relative elevation changes having a variable sense of movement are closely associated with the Melones fault zone and other known faults or structural contacts in the northern Sierra Nevada. Whereas elevation changes near the eastern front of the Sierra Nevada are probably related to the occurrence of frequent local earthquakes, similar changes within the Sierra block are not clearly related to seismic activity. These movements imply strain accumulation and stain release. When the imposed stresses are not released by aseismic deformation and when they exceed the

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strength of the crust, the weakest zones will rupture by faulting to relieve the accumulated strain. Therefore, wherever current crustal movement is observed to be associated with known faults, those faults should be regarded as potentially active even though seismic activity or surficial geologic evidence may be lacking. Whether these changes within the Sierra Nevada have occurred as distinct episodic strain events or whether they reflect more gradual variations in the regional strain field is uncertain. Whatever the exact nature of the movement and whatever the causative forces may be, crustal movements of significant magnitude may be normal in tectonically active areas and may not necessarily precede earthquakes.

The knowledge that aseismic crustal movements occur in California bears upon current efforts to develop effective earthquake predictions. The "anomalous" precursory crustal movements that have been observed before some earthquakes may have been "normal" crustal movements similar to those observed in the Sierra Nevada. The mere detection of crustal movements by various geophysical or geodetic measurements may not, therefore, reliably portend an impending earthquake but may only suffice to define areas where more intensive geophysical surveillance will be required to detect the possible onset of crustal failure which produces the earthquake. Greater attention should be given to understanding those geophysical anomalies which do not produce earthquakes and are written off as inexplicable or are assumed to have resulted from some instrumental malfunction.

The recently reported uplift (Palmdale bulge) in southern California, most of which appears to have occurred during relatively brief episodes, affords an opportunity to assess the ultimate consequences of a very significant crustal upheaval. To date, this uplift has produced no unusual seismic activity other than, possibly, the magnitude 6.4 San Fernando earthquake of 1971 which relieved accumulated strain over only a small fraction of the entire uplift region. Should the nature of the movements exhibited in the northern Sierra Nevada be characteristic of crustal movements in southern California, the Palmdale uplift may one day subside as unobtrusively as it appeared.

● **2.1-17** Williams, J. W. and Bedrossian, T. L., **Coastal zone geology near Gualala, California,** *California Geology,* **30**, *2*, Feb. 1977, 27-34.

The California Div. of Mines and Geology studied geologic hazards and mineral resources between Schooner Gulch and the Gualala River, Mendocino County, California. Potential geologic hazards such as landslides, settlement and/or liquefaction, susceptibility to earthquake shaking, tsunami inundation, and fault rupture are explored. The area sustained considerable damage during the 1906 San Francisco earthquake.

2.1-18 Nersesov, I. L. *et al.*, **On crustal deformations in the Surkhob fault zone,** *Physics of the Solid Earth,* **12**, *12*, 1977, 756-762.

The crustal deformation measured at the Sari-Pul station, which is located on one of the active sections of the Surkhob deep fault, is considered. The movement was recorded continuously by repeated geometric leveling, and with the aid of quartz deformographs and hydrostatic tiltmeters. The values obtained for the vertical and horizontal strain rates are two orders of magnitude greater than the rates on undisturbed sections of the crust and amount to 10^{-3} in the case of the vertical component and to more than 5×10^{-5} for the horizontal component. The distributions of the vertical and horizontal strains along the observation profile are of the same nature. The rock compression noted in the Surkhob fault zone corresponds to tectonic concepts regarding the compression of the Garm region and agrees with geodetic data concerning the convergence of the walls of the Surkhob fault, and also with data obtained by a network of deformograph stations in the Garm region which have been recording rock compression for a number of years.

2.1-19 Carr, M. J. and Stoiber, R. E., **Geologic setting of some destructive earthquakes in Central America,** *Geological Society of America Bulletin,* **88**, *1*, Jan. 1977, 151-156.

Most destructive earthquakes in Central America are either shallow earthquakes of moderate size ($4 \leq M \leq 7$) that occur in the volcanic belt or large shallow earthquakes ($M > 7$) that occur along the inclined seismic zone. Each of these classes of earthquakes, as well as the locations where population centers have developed, appears to be spatially controlled or influenced by Quaternary structures. This study examines the relationships between destructive earthquakes and Quaternary tectonics, and uses this information to develop qualitative estimates of seismic hazard. Quaternary structures in the volcanic belt of Central America are linked to the zone of plate convergence by transverse breaks that segment the overriding slab and the underthrust slab. The surface expressions of these breaks are transverse structural depressions. Longitudinal structural depressions coincide with lines of active volcanos.

Moderate-sized shallow earthquakes in the volcanic belt occur within the grid of transverse and longitudinal structural depressions. Many of these shallow earthquakes occur near the lines of active volcanos or are associated with volcanic eruptions. The grid of structural depressions that localizes these earthquakes provides a topographically favorable urban site. Many cities have therefore been built in especially hazardous areas.

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Large shallow earthquakes that occur in the inclined seismic zone can be divided into two types: (1) major earthquakes that have relatively small focal areas and short recurrence times and which cluster around transverse breaks in the arc, and (2) great earthquakes that have large focal areas, cause great damage, are infrequent, rupture one or more segments of the arc, and have focal areas that end at transverse breaks. Most great Central American earthquakes occurred in two brief space-time progressions. During 1847-1851, great earthquakes ruptured the El Salvador, eastern Nicaragua, and two Costa Rican segments of the arc. During 1898-1902, the western Nicaragua segment, the three Guatemalan segments, and a segment in southeast Mexico were ruptured. The most probable areas for the next great earthquake are the El Salvador and central Costa Rican segments of the arc.

● **2.1-20** Ansari, A. R., Chugh, R. S. and Sinvhal, H., **Geodetic determination of earth strains in Koyna Dam area,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 905-906.

● **2.1-21** Yeats, R. S., **High rates of vertical crustal movement near Ventura, California,** *Science,* **196,** *4287,* Apr. 15, 1977, 295-298.

Fission track, radiometric, and paleomagnetic age determinations in marine sedimentary rocks of the Ventura basin make it possible to estimate the verticle components of displacement rates for the last two million years. The basin subsided at rates up to 9.5 ± 2.5 mm per year until about 0.6 million years ago, when subsidence virtually ceased. Since then, the northern margin of the basin has been rising at an average rate of 10 ± 2 mm per year, about the same rate as that based on the geodetic record north and west of Ventura since 1960 but considerably lower than the rate along the San Andreas fault at Palmdale since 1960.

2.1-22 Tapponnier, P. and Molnar, P., **Active faulting and tectonics in China,** *Journal of Geophysical Research,* *82, 20,* July 10, 1977, 2905-2930.

A study is presented of the active tectonics of China based on an interpretation of Landsat (satellite) imagery and supplemented with seismic data. Several important fault systems, most of which are located in regions of high historical seismicity are identified using photographs and maps. The type and sense of faulting are deduced from adjacent features seen on the photos, from fault plane solutions of earthquakes, and from existing field reports. In central China, the three major east-west trending fault systems are left-lateral, strike-slip faults. Movement on these faults appears to displace much of China eastward, out of the way of the converging Indian and Eurasian subcontinents. These large eastward displacements are a key to the understanding of the tectonics of southern China, where normal faulting and right-lateral strike-slip faulting occur in Yunnan in contrast with thrust faulting in Szechwan, and of northeastern China, where extensional tectonics and basaltic volcanism dominate. The authors relate all the recent tectonics of China to the convergence of India and Eurasia during the Cenozoic at a rate of about 5 cm/yr. The continental lithosphere of Asia appears to behave like a rigid plastic medium indented by India. In this context, the large strike-slip faults are analogous to slip lines in the indented plastic material.

2.1-23 Savage, J. C. *et al.,* **Geodetic measurements of deformation associated with the Oroville, California, earthquake,** *Journal of Geophysical Research,* **82,** *11,* Apr. 10, 1977, 1667-1671.

Comparison of preearthquake vertical geodetic surveys with postearthquake surveys in the region affected by the magnitude 5.7 Oroville earthquake of Aug. 1, 1975, indicates appreciable elevation changes. The data are consistent with 0.36 m of normal slip on a 12 x 10 km^2 rectangular fault dipping 60° to the west. Comparison of level surveys run 1 and 6 months after the earthquake suggests an additional 0.08 m of postseismic slip on the fault. Horizontal geodetic data do not show any significant changes, but the horizontal data are noisier than the vertical data and allow as much as 0.7 m of coseismic left lateral slip within a 95% confidence limit. There is no evidence for postearthquake dilatancy recovery, but the data are not adequate to exclude small amounts (10 mm) of relaxation, or even larger amounts if the dilatant region were very broad (80 km in diam.).

2.1-24 King, C.-Y., Nason, R. D. and Burford, R. O., **Coseismic steps recorded on creep meters along the San Andreas fault,** *Journal of Geophysical Research,* **82,** *11,* Apr. 10, 1977, 1655-1662.

Creep meters spanning an active trace of the San Andreas fault in central California recorded many apparent fault-slip steps at times of moderate local earthquakes of magnitudes 4-5. These steps are small (less than 1 mm) but are recorded over long fault segments (tens of kilometers). The corresponding strain steps, ranging from 10^{-5} to 10^{-4}, are about 2 orders of magnitude larger than those calculated from the seismically determined source parameters on the assumption of uniform crustal materials. These anomalously large coseismic steps may be the result of low effective rigidity of the fault gouge zone or of seismically triggered secondary surface breaks that occurred along the fault.

2.1-25 Johnston, M. J. S., Jones, A. C. and Daul, W., **Continuous strain measurements during and preceding episodic creep on the San Andreas fault,** *Journal of Geophysical Research,* **82,** *36,* Dec. 10, 1977, 5683-5691.

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Continuous strain measurements from 3 three-component invar wire strainmeters installed 1200, 1500, and 1700 m from the San Andreas fault indicate no observable strain change at the instrument resolution during 10 episodic creep events on the fault. These strain observations indicate that the slip area responsible for the creep observations is near surface and of limited extent. The episodic creep character probably results from the failure properties of near-surface materials rather than general fault behavior, which is better indicated perhaps by averaged creep. Deeper slower slip apparently loads the surface material. Longer-term strain changes do occur, but the form of the signal is not what would be expected from simple models, nor is it consistent, for successive events. The amplitude does not increase with creep event amplitude, and similar changes occur without creep events. Deeper slip on the San Andreas fault apparently is smoother than would be inferred from the duration of episodic creep observations. Unfortunately, signal discrimination capability gets worse at longer periods and needs improvements if slow deformation waves are to be detected at strain levels below 10^{-7}.

2.1-26 Brown, L. D. *et al.*, **Postseismic crustal uplift near Anchorage, Alaska,** *Journal of Geophysical Research,* **82,** *23,* Aug. 10, 1977, 3369–3378.

Results of four leveling surveys carried out by the National Geodetic Survey between Anchorage and Whittier, Alaska, combined with an analysis of sea level measurements at Anchorage, indicate as much as 0.55 m of land uplift in the decade following the 1964 Prince William Sound earthquake. The pattern of uplift is parabolic in shape, convex upward, and reaches a maximum approximately halfway between Anchorage and Whittier, or about 300 km northwest of the Aleutian trench axis. The data suggest that the position of maximum uplift is migrating away from Anchorage, i.e., toward the Aleutian trench. The observed uplift occurs in a region which subsided as much as 1.9 m during the earthquake. The rate of uplift has decreased exponentially since the time of the 1964 earthquake. These movements appear to represent postseismic deformation associated with the 1964 Alaska earthquake. The observations are most easily explained by creep along the downdip extension of the fault which ruptured during the 1964 earthquake, although viscoelastic rebound and long-term elastic strain accumulation mechanisms may play a part. There is no evidence supporting magma intrusion or dilatancy mechanisms. These results provide new constraints for models of tectonic processes at convergent plate margins.

2.1-27 Huggett, G. R., Slater, L. E. and Langbein, J., **Fault slip episodes near Hollister, California: initial results using a multiwave length distance-measuring instrument,** *Journal of Geophysical Research,* **82,** *23,* Aug. 10, 1977, 3361–3368.

The most precise measurements of strain on long base lines (1-11 km) are currently being made in central California near Hollister. Base lines radiating from Hollister have been measured daily, except when measurements are limited by poor visibility, since Sept. 1975. Strain episodes have been observed that are attributed to slip at depth on the major faults in the area. There is a suggestion that this slip begins several km down and propagates upward to the surface. Many of these strain episodes have coincided with magnitude 2.5-3.5 earthquakes in the region. Another strain episode indicates that the eastern end of the Sargent fault may continue past the Calaveras fault for approximately 2 km.

● 2.1-28 Kennedy, M. P. *et al.*, **Character and recency of faulting, San Diego metropolitan area, California,** *Special Report 123,* California Div. of Mines and Geology, Sacramento, 1975, 33.

The San Diego greater metropolitan area is complexly dissected by Quaternary faults. These faults lie within three major northwesterly striking zones: Rose Canyon, Point Loma, and La Macion. Together these fault zones lie within a 20 km wide belt that extends from La Jolla on the north to the U.S.-Mexican boundary and beyond on the south. Individual faults within these zones are as much as 8 km in length, although the average length is less than 1 km.

A minimum and/or maximum age is assigned to the displacements associated with each fault where the age of strata faulted or not faulted is known. The youngest strata known to be faulted belong to the upper Pleistocene Bay Point formation. Because these strata are not commonly overlain by Holocene sediments, it has not been possible to determine if overlying Holocene sediments are faulted. The late Pleistocene age assigned such faults represents the maximum age for their most recent movement. Along the Rose Canyon fault zone, an average rate of 1-2 m/10^3 years (meters per thousand years) strike slip and 10-15 cm/10^3 years (centimeters per thousand years) dip-slip separation has occurred since late Pliocene time. The amount of separation along the Rose Canyon fault zone associated with any one earthquake is unknown as is the total amount of possible aseismic creep.

2.1-29 Fujii, Y., **Creep dislocation propagation along the east-off-Izu tectonic line, Japan, around the year of 1930 and 1976** (in Japanese), *Zisin, Journal of the Seismological Society of Japan,* **30,** *4,* Dec. 1977, 389–400.

Crustal upheaval of 10 cm for two years in the Izu peninsula, Japan, was detected by the Geographical Survey Inst. in Mar. 1976. This upheaval developed afterward laterally to a southwestern direction from Ito to Shimofunabara through Hiyekawa pass. The upheaval was associated with small and microearthquake activities, the epicenters of which are restricted within the lifted region. In order to

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interpret this event, the author assumes slow fault movement at depth along the downward extension of thrust faulting on the east-off-Izu tectonic line. The propagating fault movement at a different depth and horizontal position can provide a model for migrating crustal movement.

Preceding the 1976 crustal uplift, uplift of 1 cm in 1971 at Oiso and 2 cm in 1972 at Manazuru occurred. The observed crustal upheaval associated with the 1974 Izu-hanto-oki earthquake cannot be interpreted by the calculated crustal movement, assuming the right-lateral strike slip faulting in Minami-izu; instead creep dislocation at depth is suggested. Therefore, a sequence of propagating crustal upheaval is presumed.

On the other hand, the 1923 Kanto earthquake was related mainly to the right-lateral faulting along the Sagami fault and also to the faulting along the northern part of the east-off-Izu tectonic line. After the 1923 event, there occurred the Ito earthquake swarm and the Kita-izu earthquake in 1930. The sequence of the Ito earthquake swarm and the crustal activities in 1930 are similar to the 1976 crustal upheaval and the microearthquake activities. Large or moderate earthquakes were associated with both crustal activities during the course of rapid accelerating upheaval, and small earthquakes followed during subsequent crustal subsidence. Based on these facts, the author concludes that creep dislocation propagation recurred around 1930 and 1976. This aseismic slip at depth along the east-off-Izu tectonic line will release some of the crustal strains that are stored on the northern boundary of the Philippine Sea plate.

2.1-30 Reilinger, R. E., Citron, G. P. and Brown, L. D., **Recent vertical crustal movements from precise leveling data in southwestern Montana, western Yellowstone National Park, and the Snake River Plain,** *Journal of Geophysical Research,* 82, *33,* Nov. 10, 1977, 5349-5359.

Repeated levelings in southwestern Montana, the western portion of Yellowstone National Park, and the Snake River Plain provide information on the pattern of relative vertical crustal movement throughout this region. Except for the coseismic deformation associated with the 1959 Hebgen Lake earthquake, the most outstanding and best-defined feature of the data is contemporary doming at a rate of 3-5 mm/yr involving approximately 8000 km^2 including the epicentral area and aftershock zone of the 1959 Hebgen Lake earthquake. On the basis of observations over different time intervals, doming appears to have continued throughout the time the movements were monitored, beginning at least 25 yr prior to the 1959 earthquake and continuing at least 1 yr after the earthquake. The character of the coseismic deformation associated with the 1959 earthquake and the high regional elevation are consistent with the observed doming. It is suggested that doming preceded the earthquake for a considerable time (of the order of hundreds to thousands of years, perhaps longer), giving rise to tensional stresses in the upper crust. When these stresses, combined with the regional tectonic stresses, exceeded some critical value, faulting and collapse in response to gravity occurred, resulting in the 1959 earthquake. The voluminous Tertiary and younger volcanics in the vicinity of the doming region suggest that magma intrusion into the crust is the most likely cause of the observed uplift. The proximity of the doming region to the thermally active Yellowstone area supports this suggestion. Secondary features of the data include (1) a spatial correlation between tilting and historic seismic activity; (2) uplift within the Norris Mammoth corridor in Yellowstone National Park relative to nearby bench marks to the north and south; (3) regional subsidence of the eastern Snake River Plain relative to points north and west of this physiographic province, including subsidence of the Pleistocene Island Park caldera floor relative to its rim fractures; and (4) rapid tilting in the vicinity of the Continental fault east of Butte, the intersection of the Gardiner, Mammoth, and Reese faults just north of Yellowstone National Park, and the Madison Range fault in eastern Idaho.

2.1-31 Klein, F. W., Einarsson, P. and Wyss, M., **The Reykjanes Peninsula, Iceland, earthquake swarm of September 1972 and its tectonic significance,** *Journal of Geophysical Research,* 82, *5,* Feb. 10, 1977, 865-888.

A temporary network of up to 23 short-period seismic stations operated on the Reykjanes Peninsula during the summer of 1972 and recorded an earthquake swarm consisting of more than 17,000 events. The area studied is the immediate landward extension of the Mid-Atlantic Ridge, and seismic processes are probably similar to those on submarine parts of the ridge crest. The Reykjanes Peninsula forms the transition between the Reykjanes Ridge and the south Iceland transform fault. The peninsula is essentially a leaky transform fault, and focal mechanism solutions are of normal and strike-slip types. The 12 km-long seismic zone, active during the 8-day swarm, is defined by 2514 hypocenters, which are mostly between 2 and 5 km deep. Individual structures within the seismic zone include several seismic lineations or faults trending obliquely to the zone. Also visible are two aseismic zones, which may be either rigid blocks or low-rigidity regions, such as magma chambers. Thus the observed seismic zone is not a single fault; instead it appears to be the shallow expression of a deeper-seated and aseismic deformation zone. In time, the seismic events cluster together into subswarms lasting a few hours and migrate laterally at speeds of 1-2 km/d. The energy released is equivalent to that of a single event of Icelandic local magnitude 4.9, and the total fault area is close to that expected for a single earthquake of this size. Although the high microearthquake and swarm seismicity can be generally related to the presence of an adjacent geothermal area, the observed earthquakes do not locate in or outline the hydrothermal upwelling zone visible at the surface. This

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contribution is the first study to reveal the details of faulting during an ocean ridge earthquake swarm, both in space and in time.

2.2 Wave Propagation

● 2.2-1 Yacoub, N. K. and Mitchell, B. J., **Attenuation of Rayleigh-wave amplitudes across Eurasia,** *Bulletin of the Seismological Society of America,* **67,** *3,* June 1977, 751-769.

Surface waves generated by six earthquakes and two nuclear explosions are used to study the attenuation coefficients of the fundamental Rayleigh mode across Eurasia. Rayleigh-wave amplitude data yield average attenuation coefficients at periods between 4 and 50 sec. The data exhibit relatively large standard deviations, and in some cases the average attenuation coefficients take on negative values which may be due to regional variations of the attenuative properties of the crust, lateral refraction, multipathing, and scattering.

A method has been developed to investigate the regional variation in the attenuative properties of the Eurasian crust and its effect on surface-wave amplitude data, employing the evaluated average attenuation coefficients for the fundamental Rayleigh mode. For this investigation, Eurasia is divided into two regions, one considered to be relatively stable, and the other considered to be tectonic in nature. This regionalization shows that the tectonic regions exhibit higher attenuation than the stable regions in the period range below about 20 sec, whereas in the period range above about 20 sec, no clear difference can be observed for the two regions. Although the effects of lateral refraction and multipathing may still significantly affect the observations, the regionalization lowers the standard deviations considerably and eliminates the negative values which were obtained in the unregionalized determinations.

● 2.2-2 Kobori, T., Takeuchi, Y. and Kawano, M., **On seismic wave propagation in a continuous random medium,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 605-610.

The effect of the irregular geologic properties of a transmission medium on seismic wave propagation from an earthquake focus to an observation point is statistically investigated. The amplitude and phase characteristics of the average surface displacements to a point source buried in a random, semi-infinite medium are evaluated for the case when the density of a medium is considered only as a random variable. As a result, it is pointed out that the scattering effect on the seismic wave propagation is mainly governed by the relationship between the size of the random inhomogeneity and the length of the traveling wave motion.

● 2.2-3 Mulay, J. M. and Ramesh, C. K., **Analysis of propagation of shock-waves by finite-element method,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 617-623.

A finite element method is proposed to study the propagation of waves generated by artificial blasts in underground media. An infinite medium having a buried source with six types of forcing functions, and a semi-infinite medium, with or without layers, having the source either on the surface or buried, are evaluated.

● 2.2-4 Nemani, D. and Mal, A. K., **Short period surface waves in a layered medium,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 611-616.

Properties of surface waves at relatively short periods and small epicentral distances are investigated by means of existing seismological techniques. The authors assume a single-layered model of the earth and calculate the Love-wave displacement spectra that would be produced by shallow earthquake sources located within the layer. The interference between the higher modes is shown to be a significant factor in producing complex ground motions. The amplification of the waves between the interface and the free surface is shown to have a highly anomalous behavior due to the presence of the higher modes.

● 2.2-5 Shima, E., **On the base rock of Tokyo metropolis,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 496-501.

From observations of seismic waves generated from four large explosions, the thickness of subsurface layers down to the base rock of Tokyo was found to be 2.3 km. The base rock of Tokyo having Vp=5.6 km/sec and Vs=3.0 km/sec corresponds to the uppermost layer of the earth's crust. Dispersion characteristics for both Love and Rayleigh waves were computed assuming the derived underground structure. Seismic waves having the period of 8 sec in the case of Love and 4.5 sec in the case of Rayleigh waves may predominate in Tokyo during large earthquakes.

2.2-6 Alekseyev, A. S. and Mikhaylenko, B. G., **Solution of Lamb's problem for a vertically inhomogenous elastic halfspace,** *Physics of the Solid Earth,* **12,** *12,* 1977, 748-755.

A solution of Lamb's problem is given on the basis of an extension of the method of partial separation of the variables using the finite-difference techniques of computational mathematics. Some dynamic peculiarities of the wave propagation in inhomogeneous media are examined.

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● 2.2-7 Dey, S. and Addy, S. K., **Reflection of plane waves under initial stresses at a free surface,** *International Journal of Non-Linear Mechanics,* **12,** *6,* 1977, 371–382.

This paper deals with the phenomenon of reflection of plane elastic waves in a free surface when the medium is initially stressed. It has been shown analytically that the reflected *P* and *SV* waves depend on initial stresses present in the medium. The numerical values of reflection coefficients for different initial stresses and the angle of incidence have been calculated using a computer and the results are given in the form of graphs. Many results not seen in an initially stress-free medium are discussed.

2.2-8 Singh, B. M. and Dhaliwal, R. S., **Diffraction of *SH* waves by Griffith cracks or rigid strips lying in a laterally and vertically non-homogeneous infinite medium,** *International Journal of Engineering Science,* **15,** *5,* 1977, 295–304.

The problem of diffraction of obliquely incident *SH* waves by one Griffith crack in an infinite nonhomogeneous elastic medium is solved. The problems of diffraction of normally incident *SH* waves by two coplanar cracks or by two coplanar rigid strips in an infinite nonhomogeneous elastic medium are also solved. Approximate expressions are derived for the displacement component and the stress intensity factors. The modulus of rigidity and the density of the material are assumed to vary in the horizontal and vertical directions, and the ratio is assumed to be constant. The numerical values of the stress-intensity factors are displayed graphically to show the effect of nonhomogeneity of the material. The results for the corresponding problems for an isotropic, homogeneous, elastic medium are derived as particular cases.

2.3 Source Mechanisms

2.3-1 Patton, H., **A note on the source mechanism of the southeastern Missouri earthquake of October 21, 1965,** *Journal of Geophysical Research,* **81,** *8,* Mar. 10, 1976, 1483–1486.

In studies of the source mechanism of small intraplate earthquakes, the analysis of surface wave spectra is extremely valuable because reliable *P* wave solutions of these earthquakes are difficult to obtain. To determine the source mechanism using surface waves alone, phase and amplitude spectra must be analyzed. In this study of the southeastern Missouri earthquake of Oct. 21, 1965, the authors examine the phase spectra of Rayleigh waves over periods of 20–50 s to follow up Mitchell's (1973) analysis of spectral amplitudes. It is shown that, using known regionalized dispersion curves to remove propagation effects, the authors can isolate focal phase spectra and determine the mechanism of the earthquake. The results provide conclusive evidence for normal faulting with a NNW–SSE axis of tension. Thus, the analysis of phase spectra gives results which are complementary as well as supportive to those obtained by Mitchell.

2.4 Seismicity, Seismic Regionalization, Earthquake Risk, Statistics and Probability Analysis

2.4-1 Borisov, B. A. and Reysner, G. I., **Seismo-tectonic prognosis of the maximum magnitude of earthquakes in the Carpathian region,** *Physics of the Solid Earth,* **12,** *5,* 1976, 300–306.

A variant of the prediction of the maximum magnitudes of expected earthquakes in the Carpathian region is presented. M_{max} is evaluated by using the solution of a trial problem regarding the Caucasus, the solution of which is based on one-dimensional nonlinear transformations. From this solution the functional dependence between the M_{max} values and the group of geological-geophysical parameters is found. The basic data concerning the Carpathian region consist of a set of maps from which nine formalized maps for every geological indicator used for the analysis are compiled. In the final map of scale 1:2500 000, the M_{max} values, computed from the formalized maps, are referred to the elementary area-cells (10 ft x 15 ft). Such a resolution is fully justified as is borne out by a comparison of the map of the predicted and the observed M_{max} values in the Carpathian region.

2.4-2 Dzhibladze, E. A., **A comparison of various methods of estimating the magnitude of the maximum possible earthquake in the Caucasus,** *Physics of the Solid Earth,* **12,** *7,* 1976, 425–429.

In order to compare the quantitative and qualitative methods of determining K_{max}, the long-term average recurrence frequency, B(I) is calculated for various values of K_{max} with the other initial data remaining unchanged and is compared with the observed recurrence frequency.

2.4-3 Dzhibladze, Z. A., Bolkvadze, I. N. and Dzhidzheyshvili, P. O., **The refinement of maps of the maximum possible earthquakes and shakeability of the Caucasus,** *Physics of the Solid Earth,* **12,** *8,* 1976, 499–504.

In the present paper the value K_{max} for the Caucasus is determined and mapped from gravimetric data (the horizontal gradients of the isostatic gravity anomalies) and composite data: seismological (seismic activity), geophysical (gravimetry), and geodetic (simultaneous vertical movements).

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Maps of the average period of expected intensities for Georgia and its adjacent territories with an intensity of level 7 to 9 are constructed on the basis of the seismic activity, the value of the maximum possible earthquakes K_{max}, and the attenuation of the macroseismic intensity I with focal distance. The results are compared with the observed long-term average recurrence frequency.

● 2.4-4 Kagan, Y. and Knopoff, L., **Earthquake risk prediction as a stochastic process,** *Physics of the Earth and Planetary Interiors,* **14,** 1977, 97–108.

The extrapolation in time of an earthquake sequence considered as a multidimensional stochastic point process is discussed. Estimates of seismic risk for both long- and short-term predictions are considered and an algorithm for the calculations is proposed. Several examples of short-term extrapolations are carried out by means of Monte Carlo simulations of the process. An assessment of the predictability of the seismic process shows that the catalog of strong earthquakes ($M \geq 7.0$) contains about 0.4 bits of information per earthquake for the particular model of the process applied here.

● 2.4-5 Smith, W. D., **Statistical estimates of the likelihood of earthquake shaking throughout New Zealand,** *Bulletin of the New Zealand National Society for Earthquake Engineering,* 9, *4,* Dec. 1976, 213–221.

A historical record of earthquakes can be used to estimate the probability that any particular locality will be shaken by earthquakes in the future. This approach assumes that the past record is representative of what will happen in the future. While this assumption may not be valid for long-term estimates, return periods for the same length of time or less as the historical record may be estimated using the approach. Intensity is chosen as the parameter which best describes the totality of earthquake shaking. Formulas relating intensity to the magnitude and location of the earthquake and to the location of the observer are developed. The complex structure of New Zealand demands three attenuation patterns, each characteristic of a distinct source region. The historical record of earthquake occurrence is then examined. Models are fitted to the statistical population of intensities likely to have been observed at each of a grid of sites throughout the country. The model parameters are mean return periods for intensities equalling or exceeding MM VI, VII, VIII and IX.

● 2.4-6 Tabban, A. and Gencoglu, S., **Earthquakes and their parameters** (Deprem ve parametreleri, in Turkish), *Deprem Arastirma Enstitusu bulteni,* 3, *11,* Oct. 1975, 7–83.

In this study, the causes responsible for the occurrence of earthquakes are briefly reviewed. Then the elastic waves generated by earthquakes are discussed at some length. The remaining part of the work is devoted to studying various earthquake parameters, such as intensity and magnitude.

2.4-7 Zhalkovskii, N. D. and Muchnaya, V. I., **Results of macroseismic investigations of strong earthquakes in the Altay-Sayan region** (Nekotorye rezultaty makroseismicheskikh issledovanii silnykh zemletryasenii Altae-Sayanskoi oblasti, in Russian), *Seismichnost Altae-Sayanskoi oblasti,* NAUKA, Novosibirsk, 1975, 28–42.

Macroseismic data from 1898 to 1972 are analyzed. Isoseismal maps for 11 destructive earthquakes are presented. The effects of the size of the focal region and other factors on intensity attenuation with increasing hypocentral distance are considered.

2.4-8 Talandier, J. and Kuster, G. T., **Seismicity and submarine volcanic activity in French Polynesia,** *Journal of Geophysical Research,* 81, *5,* Feb. 10, 1976, 936–948.

The implementation of 10 short-period seismic stations in French Polynesia since 1963 allowed the detection of previously unnoticed seismic activity in the central South Pacific (5°-30°S, 125°-160°W). The epicenter location, the depth, and the magnitude of all major events which occurred in this region are tabulated. The Tahiti-Mehetia region, which is the zone of activity closest to the stations, is studied in detail. Swarms of earthquakes are observed at two well-defined sites. Their similarity to typical preeruptive swarms, their location near two seamounts, and the general volcanic context are considered indicative of two active submarine volcanoes. The existence of a "hot spot" at the southeastern tip of the Society Island chain is suggested.

● 2.4-9 Krinitzsky, E. L. and Chang, F. K., **Design earthquakes,** *Wind and Seismic Effects,* VI-179–VI-191. (For a full bibliographic citation, see Abstract No. 1.2-4.)

Relationships between earthquake intensity and epicentral distance are presented. Peak motion results, from 187 field records, are given for examination. Also presented are data on displacements, velocities, and intensities.

● 2.4-10 Taleb-Agha, G. and Whitman, R. V., **Seismic risk analysis of discrete system of sites,** *Earthquake Engineering and Structural Dynamics,* 5, *3,* July–Sept. 1977, 293–304.

Two efficient schemes have been developed for the analysis of discrete systems of sites. Both schemes have the same objective of finding the probability of simultaneous failure of any number of sites belonging to a given system of sites subject to threats from a given set of earthquake sources with known seismic history. In the first scheme, systems with deterministic site resistances can effectively be analyzed using a nonlinear transformation of variables. In the second scheme, systems with random site resistances

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can be analyzed. To overcome the computational difficulties involved in the analysis, a new set of simple recursive formulas has been developed and used effectively. Based on these two schemes, two efficient computer programs were prepared and used to perform a parametric study on a system of nine actual or contemplated nuclear power plants in New England. The results have shown that the problem is very sensitive to the coefficient of variation of the resistances and not so sensitive to the mean resistances.

● **2.4-11** McCue, K. F., **Seismicity and seismic risk in South Australia,** *ADP 137,* Dept. of Physics, Univ. of Adelaide, Australia, July 1975, 92.

One aspect of the general engineering design and construction of normal buildings that has been all but neglected in Australia until very recently is the consideration of earthquake-resistant design. Following the Meckering Western Australia earthquake and the more recent publication of the draft Australian Building Code, the National Committee on Earthquake Engineering has urged the completion of a seismic zoning map for the whole of Australia. The author was commissioned by the Univ. of Adelaide and the National Committee on Earthquake Engineering to establish for South Australia contour maps of seismic risk, defined as the annual probability of exceeding a certain intensity of shaking as measured by the peak ground velocity. This report follows on from a similar study of the *Seismicity of Western Australia,* based on identical analytical methods and marks the completion of the preliminary analysis for the two most important seismic zones in Australia.

Historical information was used, calibrated by the instrumental data of recent years, to extend the sample interval from fifteen to about one hundred and thirty years. A Gumbel extreme-value analysis identical to that used to draw up the draft Australian wind code was employed to compute seismic risk figures for Adelaide and Redcliff and risk contours for the remainder of the state.

● **2.4-12** Sanford, A. R., **Seismicity of the Los Alamos region based on seismological data,** *LA-6416-MS,* Los Alamos Scientific Lab., Univ. of California, New Mexico, July 1976, 9.

Estimates of seismic risk in the Los Alamos area based on only seismological evidence are presented. The data used in the study and tabulated in this paper were restricted to shocks occurring within 111 km of Los Alamos and included: (1) noninstrumental reports of earthquakes prior to 1962, and (2) instrumental studies of shocks from 1962 to 1972.

The strongest earthquake to occur within the region of study during the 100-yr period, 1872–1972, had a probable magnitude of 5.5. Estimates of the strongest shock to occur in a 100-yr period, based on extrapolation of the earthquake frequency-magnitude relation, range from 3.9 to 5.4 depending on the data set used.

The report recommends that seismic risk in the Los Alamos area be based on the recurrence of a magnitude 5.5 earthquake once every 100 yr somewhere within the Rio Grande depression from Albuquerque to Questa. The seismicity of this part of the rift zone is: (1) less than the Albuquerque-to-Socorro segment of the same structure, and (2) substantially less than equivalent areas in southern California.

● **2.4-13** Aziz, T. S. and Charlwood, R. G., **Seismic risk analysis for Canadian nuclear power plants,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(b), Paper K 9/3, 11. (For a full bibliographic citation, see Abstract No. 1.2-5.)

A general formulation for seismic risk analysis of nuclear power plants sited in Canada is presented. The study identifies the important parameters by sensitivity analyses and draws certain conclusions regarding their importance for risk. The formulation allows the use of various forms of probabilistic description of the resistance. Analyses are presented for lognormal, uniform natural logarithms, and uniform probability distribution for the resistance, and it is shown that the total risk is insensitive to the actual distribution in the ranges of interest.

Two measures of seismic risk are discussed, the total seismic risk and the risk due to the design-basis earthquake only. Typical values for each are computed and comparisons show that the former is relatively insensitive to design decisions and may be as high as 10^{-5} while the latter is very sensitive and may be as low as 10^{-12}. The safety index β is used as the measure of the design basis. A Bayesian approach is proposed for treatment of uncertainty in the parameters. The effects of other loads on seismic risk are studied, and it is shown that inconsistencies exist in the overall factor of safety and that systems designed primarily for seismic loads have the least probability of survival.

● **2.4-14** Sagesser, R., Rast, B. and Merz, H., **Seismic risk maps of Switzerland,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(a), Paper K 1/3, 12. (For a full bibliographic citation, see Abstract No. 1.2-5.)

Seismic risk maps of Switzerland have been developed under the auspices of the Swiss Federal Div. on Nuclear Safety. They are primarily destined for the use of owners of future nuclear power plants. The results will be mandatory for these future sites. The results will be shown as contour maps of equal intensities for average return periods of 500, 1000, 10,000, etc., years. This general form will not restrict the use of the results to nuclear power plants only, but it

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allows their applicability to any site or installation of public interest (such as waste deposits, hydropower plants, etc.). This follows the recommendations of the UNESCO World Conference (Paris, Feb. 1976). In the study, MSK 64 intensity was chosen. The detailed scale allowed a precise handling of historical data and separates the results from continuously changing state-of-the-art correlations to acceleration and other input motion parameters. The method used is the probabilistic theory developed by C. A. Cornell and others at MIT in the late 1960s with the program in the version of the U.S. Geological Survey by R. McGuire. In the study, the program was extended for the use of the continuous attenuation law by Sponheuer, azimuth-dependency in the attenuation relation, a quadratic intensity-frequency relation, larger number of gross sources, and output modifications with respect to the mapping program used.

To determine the basic parameters, more than 3000 independent events in an area of approximately 240,000 km^2—Switzerland with its neighboring parts of Italy, Austria, Germany, and France—were systematically classified (and relocated where necessary). Important decisions in the handling of these data are discussed. The parameters were evaluated in the following time periods: years 0-1975, 1750-1975, 1856-1975, 1880-1975, and 1913-1975. In the risk analysis, numerical values based on the period 1750-1975 were finally taken, the longest for intensities equal to VI. In the low intensity range, the period 1880-1975 was taken as representative. Interarrival times were checked over a range of time steps of 5 to 150 years. Upper bounds were handled with a cutoff intensity since—even with this set of data—an obvious yield to an upper bound in the intensity-frequency relation was not visible. A quadratic law was found for all sources; the fit was better than by a linear or a bilinear curve. Seismotectonic boundaries, based on epicenter distribution, background rates, and the tectonics in the area were found to be laid in north-south lines crossing the Alps. The Rhine graben was considered as a province on its own. The maps were prepared to set seismic design provisions under a common philosophy, avoiding the elaborate work of individual analyses and the chance of large discrepancies in interpretation. Special topics on the parameters and preliminary results are presented. The value of seismic risk analyses, especially for the extensive sensitivity studies performed in conjunction with such analyses, is weighed against the limitation of such analyses.

● **2.4-15** Milne, W. G., **Earthquake: studies of the seismic risk in British Columbia,** *Emergency Planning Digest,* **4,** *2,* Mar.-Apr. 1977, 2-5.

Earthquakes, which are frequently and sometimes severely felt in British Columbia, have generated considerable public interest and concern. The Seismological Service of the Energy, Mines and Resources, Earth Physics Branch has responded to this concern with research programs to study earthquakes in the region and to supply public information on earthquakes. The number of regional seismograph stations has been increased from one installation in 1898 to a network of stations, capable of recording data from any earthquake felt in British Columbia, and, since 1975, capable of recording data from several of the regional seismographs at a central installation in Victoria. A scientific staff has been assembled at the Victoria Geophysical Observatory to study the data that are collected. Results of the research programs are provided to organizations responsible for the design of structures, the planning of regional development projects, and the planning for emergency situations. It is expected that a proper application of the knowledge gained from these studies can minimize the loss of life and property damage during the next severe earthquake.

● **2.4-16** Simons, R. S., **Seismicity of San Diego, 1934-1974,** *Bulletin of the Seismological Society of America,* **67,** 3, June 1977, 809-826.

Twelve quarry explosions in the city of San Diego have been used to determine a crustal velocity model for the region around it. A computer program employing this model has been used to recalculate the epicenters of all events previously located in the San Diego area, utilizing data from the California Inst. of Technology seismic network as well as recent new stations within the city. Tests on the accuracy of the location process indicate that over 50% of the solutions can be expected to be within 2 km of the true epicenters and that 90% will be within 4 km. A total of 37 earthquakes can now be identified with some confidence as having occurred within the study area (32.5°-33.0°N, 116.75°-117.5°W) from 1934 through 1974. Some events previously thought to be earthquakes are now found to have been quarry blasts. The great majority of the earthquakes lie either offshore or less than 10 km inland in regions of known faulting paralleling the Coronado escarpment and the Rose Canyon fault zone. Three earthquakes are located within 2 km of the La Nacion fault. Nine of the 11 events since 1963 have taken place within or around the south end of San Diego Bay. Depths are poorly controlled but seem to be generally less than 8 km. Magnitudes range from 2.3 to 3.7.

● **2.4-17** McGuire, R. K., **Effects of uncertainty in seismicity on estimates of seismic hazard for the east coast of the United States,** *Bulletin of the Seismological Society of America,* 67, 3, June 1977, 827-848.

The lack of knowledge of the cause of earthquakes in the eastern United States and the short length of seismic history lead to decisional and statistical uncertainties concerning the seismicity of the east. The effect of these uncertainties on the calculated seismic hazard for the east coast is assessed by deriving probability distributions of the

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important parameters. The available earthquake catalog for the eastern United States is adequate to establish activity rates for seismic sources but not maximum possible sizes of events. Thus, it is not justifiable on a statistical basis to assume that events larger than those observed historically in areas of the east will not occur in the future. The modified Mercalli intensities associated with an annual probability of 10^{-4} of being equaled or exceeded range from VIII to IX for selected sites from Florida to Maine; these intensities are generally insensitive to the manner in which seismic sources are drawn to represent seismicity, except for those sites which lie within an important seismic source under one hypothesis and outside all seismic sources under a different hypothesis. Several disadvantages of using a strictly deterministic procedure to determine design intensities are disclosed by this study. Specifically, design intensities established deterministically are sensitive to seismic source geometry and to the largest event observed historically in each source. The risks associated with these design intensities vary by more than a factor of ten for the sites on the east coast examined here.

● 2.4-18 Douglas, B. M. and Ryall, A., **Seismic risk in linear source regions, with application to the San Andreas fault,** *Bulletin of the Seismological Society of America,* **67,** *1,* Feb. 1977, 233–241.

A method is described for determining recurrence times as a function of distance to the causative fault and magnitude for earthquakes distributed along a linear source zone. The method takes into account rupture length, which is scaled to magnitude, and permits direct calculation of approximate return periods for peak ground motion parameters for large earthquakes when the appropriate attenuation functions are known. Several examples are presented using instrumentally determined seismicity along the San Andreas fault zone. Results illustrate the necessity of incorporating rupture length in calculations related to seismic risk; for large earthquakes, it is also necessary to use a source region large enough to contain the rupture zones of all such events. For a site in the San Andreas fault zone, it is found that the recurrence time is within 10 km of the causative fault of an earthquake with $M \geq 8$ is 200 to 300 years, depending on the choice of maximum magnitude (8.6 or 8.4). For a site on the fault in the Hollister region, it is found that recurrence times which are within 10 km of the rupture due to events of $M \geq 5$, 6 and 7 are, respectively, 12, 45 and 105 years.

● 2.4-19 Rogers, A. M., Perkins, D. M. and McKeown, F. A., **A preliminary assessment of the seismic hazard of the Nevada test site region,** *Bulletin of the Seismological Society of America,* 67, *6,* Dec. 1977, 1587–1606.

A deterministic analysis employing a map of active faults and using Schnabel and Seed attenuation curves suggests that mean expected peak accelerations range from 0.2 to 0.7 g at various locations at the Nevada Test Site for $M = 7$ maximum magnitude. Geological evidence suggests that recurrence intervals for these magnitudes average several thousand years. Hazard analyses for upper and lower bounds of expected test site seismicity indicate that accelerations of the size given by the deterministic analysis have return periods of 1500 to 15,000 years. If the variability in peak accelerations for a given distance and magnitude are taken into account, the accelerations for return periods of 1500 to 15,000 years would be larger by a factor of 3 than the deterministic accelerations.

● 2.4-20 Adams, R. D., **Principal earthquakes during the year 1976,** *Bulletin of the New Zealand National Society for Earthquake Engineering,* **10,** *1,* Mar. 1977, p. 52.

This report describes and enumerates the locations of principal earthquakes in New Zealand during 1976.

● 2.4-21 Chandra, U., **Earthquakes of peninsular India—a seismotectonic study,** *Bulletin of the Seismological Society of America,* **67,** *5,* Oct. 1977, 1387–1413.

Seismicity of peninsular India is studied from a detailed consideration of the historical as well as recent earthquake data, and a catalog of earthquakes from the earliest time through 1974 occurring within the region, 5°N-28°N-67.5°E-90°E, is prepared. Separate epicenter maps showing instrumentally located earthquakes and earthquakes located by macroseismic observations are presented. A number of seismic zones, based largely on the distribution of earthquake epicenters but, also, partly on local geology and fault-plane solutions, are tentatively identified. Some of the zones of weakness may be related to the plume-generated triple- or four-armed rifts formed at different times in the geological past. Recent seismicity is related to the rejuvenation of activity along some of these zones of weakness. Fault-plane solutions for six earthquakes, widely separated geographically and related to different seismic zones, obtained by using teleseismic observations from the records of worldwide standardized stations network, show two consistent features. One of these features is that the axis of pressure trends nearly in the north-south direction, and the other is that one of the nodal planes, in at least one possible solution for each event, strikes in a north-northeast to east direction and shows a left lateral sense of motion. The former observation indicates that the high stresses generated by the continental collision, caused by the northward movement of India, have affected the entire Indian peninsula. The latter observation suggests that the whole region may be under a state of left lateral shear. This shear is believed to be a consequence of the geometric disposition of the Indian and Eurasian continents at the time of collision and the subsequent plate kinematics. The latter observation also suggests that the orientation of the zones of weakness with respect to the ambient stress field may be an important factor in

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determining the faults along which earthquakes are likely to occur.

● **2.4-22** Ryall, A., **Earthquake hazard in the Nevada region,** *Bulletin of the Seismological Society of America,* 67, *2,* Apr. 1977, 517-532.

The seismic cycle in Nevada, corresponding to the rerupture time for major faults, is of the order of thousands of years long. Following the occurrence of a large ($M > 7$) earthquake, aftershock activity in the rupture zone continues for about a century. From earthquakes reported in the meizoseismal zones of the 1915 Pleasant Valley and 1954 Dixie Valley earthquakes, it appears that great Nevada earthquakes are also preceded by a moderate increase in seismicity, lasting several decades. Faults considered to be "active" on geologic grounds are distributed fairly evenly over most of the Nevada region so that fault mapping alone is not enough for determination of seismic potential. In addition, because of the length of the seismic cycle, the distribution of large historic earthquakes is by itself inadequate for seismic regionalization. A better method for this region may be to spatially correlate epicenter lineups for microearthquakes and small earthquakes with mapped geologic faults or lineaments. A correlation of this type based on earthquakes recorded for the period 1970-1974 suggests that a number of northwest-trending zones in an area of west-central Nevada may have high potential for large earthquakes in the future. On the other hand, high seismicity, complex faulting, and scattered epicenter distribution in the seismic "gap" between the 1932 Cedar Mountains and 1872 Owens Valley earthquakes suggest that tectonic stress in this zone may be relieved by a continuous series of small-to-moderate earthquakes.

● **2.4-23** Herrmann, R. B., **Recurrence relations,** *Earthquake Notes,* 48, *1-2,* Jan.-June 1977, 47-49.

Estimates of seismic risk require a recurrence relation between the expected number of earthquakes per year and a measure of earthquake size such as magnitude or intensity. Two such relationships are commonly seen in the literature: the cumulative number of earthquakes per year, and the incremental number of earthquakes per year. The purpose of this presentation is to clarify the relationship between the cumulative magnitude recurrence relation and the corresponding incremental magnitude recurrence relation. The derivation given here can also be used for intensity recurrence relations.

● **2.4-24** Verma, R. K., Mukhopadhyay, M. and Roy, B. N., **Seismotectonics of the Himalaya, and the continental plate convergence,** *Tectonophysics,* 42, *2-4,* Oct. 20, 1977, 319-335.

The relationship between seismicity and tectonics of the northwestern, central, and part of the eastern Himalayas lying between longitudes 74-88°E and latitudes 26-35°N has been studied. A seismicity map of the area for the period 1890-1970 is presented. A strain-energy release map of the Himalayas was prepared following the method discussed by Allen et al. On the basis of this map, it is suggested that the Himalayas do not behave as a single unit so far as seismic activity is concerned. The Himalayas could be divided appropriately into three different units: (1) the Panjab Himalaya, (2) the Kumaon Himalaya, and (3) the Nepal Himalaya. It is suggested that the extension of Aravalli structures into the Himalayan regions has played a role in the tectonics of the Kumaon Himalaya, and probably is the cause of the complex nature of the seismicity of the region.

In the Kumaon Himalaya, the maximum strain-energy release is related to the main central thrust, whereas in the Panjab, as well as in Nepal Himalaya, it is related to the activity of the main boundary fault. The strain-energy release characteristics of the two thrusts show that their mechanisms of storage and release of energy are different. Strain-energy release through the main central thrust seems to be more uniform, as compared to the main boundary fault, along which the release of energy has been mostly abrupt through large-magnitude earthquakes.

Focal-mechanism solutions of Himalayan earthquakes located north, as well as south of the Indus suture zone, indicate that the Indian plate is underthrusting the Tibetan plate towards the north, whereas the latter is underthrusting the Indian plate towards the south. P-axes laying to the north, as well as south of the Indus suture line, are shallow dipping. This confirms the view that the present-day seismicity of the area is a result of continental-continental collision and not of lithospheric subduction.

● **2.4-25** Hernandez, A., **Orientation of Managua's faults,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 304-307.

In this article, areas in the Managua fault zone that need further study, such as analyses of seismic risk, are pointed out.

● **2.4-26** Lou, Y. S. and Chu, B. P. H., **Statistical seismicity of Taiwan,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 806-810.

Statistical analyses are performed on earthquakes in existing catalogs (1720-1974) which occurred in Taiwan. A method of evaluating seismicity of sites in Taiwan is presented. The method uses an attenuation law based on

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the spatial distribution of isoacceleration areas in conjunction with the past record of seismic events. The probability, number of occurrences, and return period of various levels of ground acceleration (velocity and displacement) at a given site are presented. The method is then applied at a number of sites in Taiwan and the results are examined and compared with other statistical methods. The parameters affecting the results are also discussed.

● 2.4-27 Rao, P. V., **Seismic probability diagrams for risk evaluation in the Indian subcontinent,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 927–928.

● 2.4-28 Shukla, D. K., Johnson, W. J. and Kissenpfennig, J. F., **Attenuation of modified Mercalli intensity for earthquakes in the Iberian Peninsula,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 909–910.

● 2.4-29 Whitman, R. V. *et al.*, **Seismic design regionalization maps for the United States,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 742–749.

Two maps give the geographical variation of effective peak acceleration (EPA) and effective peak velocity (EPV). The map for EPA is based primarily upon a seismic risk study involving selection of source zones, seismicity parameters, and attenuation laws. The map for EPV is a modification of the map for EPA. The probability that EPA and EPV will not be exceeded at any location during a 50-year period is estimated as 80% to 95%. EPA and EPV are used as input to an equation for design base shear.

● 2.4-30 Wallace, R. E., **Time-history analysis of fault scarps and fault traces-a longer view of seismicity,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 766–769.

Time-history analysis of young faults by geologic means can add important data to the limited historic record of seismicity. In central Nevada, for example, large earthquakes are estimated to have occurred at the rate of approximately 3×10^{-5} per year per 1,000 km^2 during the last 12,000 years. Average recurrence intervals on individual fault zones are approximately 10,000 years and some active faults may remain dormant for several times that long. Long-term characteristics of subprovinces can be interpreted. For example, the central Nevada seismic belt has been characterized by an upper limit of M7-8 for earthquakes for the past few hundred thousand years, whereas eastern Nevada and western Utah have been characterized by an upper limit of M6-7 for earthquakes during that period.

● 2.4-31 Cummings, D. and Leeds, D. J., **Seismotectonic zoning using theoretical mechanics,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 836–842.

Theoretical mechanics, based on theories of plasticity and elasticity, is a powerful tool for defining tectonic zones and style of deformation. The mathematical arguments of theoretical mechanics describe stress distributions which can be used to define fault patterns and areas of relative seismicity. One applicable model of theoretical mechanics is the Prandtl cell, based on the theory of plasticity. Its mathematical arguments were applied to several geologic areas in the world. These areas have a distinctly different tectonic fabric and lower level of seismicity than the surrounding areas. The theory is consistent with observed tectonics and seismicity and may be used for prediction.

● 2.4-32 Neghabat, F. and Liu, S. C., **Earthquake regionalization of Iran,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 859–865.

A comprehensive earthquake microzonation analysis of Iran is carried out. Four overlapped seismic regions, including the Zagros folded belt, the Rezaiye-Esfandazheh orogenic belt, and the central southeast Persian and Alborz ranges are analyzed independently. The statistics describing earthquake occurrence and local effects are obtained from the seismological, geological, and tectonic data of the region. Regional isoseismic contour maps, in terms of intensity with periods from 20 to 1000 yr, are constructed and synthesized into an earthquake hazard map of Iran. The map can be used to establish the earthquake design criteria of structures.

● 2.4-33 Srivastava, V. K., Chouhan, R. K. S. and Singh, J., **Seismotectonic study of northwest Kashmir,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 903–904.

In this paper, based upon the data of past earthquakes, probabilistic methods have been applied to determine the seismic activity of the northwest Kashmir region. Focal mechanism solutions of the Sept. 3, 1972 event ($M = 6.2$) and its aftershock sequence have been studied, and a possible correlation with geologic features of the region has been attempted.

● 2.4-34 Shakal, A. F. and Toksoz, M. N., **Earthquake hazard in New England,** *Science,* **195,** *4274,* Jan. 14, 1977, 171–173.

The earthquake return periods (mean interoccurrence time) and the probability of earthquake occurrences for southern New England are calculated from the available seismic data for the period 1725 through 1974. For this

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region, the occurrence of larger earthquakes varied with time and the seismic activity was higher in the period 1725 through 1824 than in the next 100 years (1825 through 1924). This variation introduces large uncertainties into calculations of the earthquake hazard. The estimated return period, based on data covering the time period 1725 through 1974, for earthquakes in the southern New England area of intensity VI or greater is 25 years and for intensity VIII or greater is 130 years.

2.4-35 Grin, V. P., Medzhitova, Z. A. and Serebryanskaya, T. Ya., **Spatial distribution of the focal parameters of weak earthquakes in the territory of the Chu basin and their relationship with strong earthquakes,** *Physics of the Solid Earth,* **13,** *3,* 1977, 166–173.

The spatial distributions of regions characterized by the maximum probable variation of the dynamic parameters of earthquake foci and the zones of increased and decreased ratios of the travel times of transverse and longitudinal waves are examined. The connection between these distributions and the epicenters of fairly strong earthquakes is found. On the basis of a comparison of the results obtained with geological data of ancient seismic dislocations, it is surmised that the regularities which have been detected may characterize long-term seismicity.

● **2.4-36** Shenoi, R. S., **Seismotectonic features as guide to design of a high dam across Narmada River in Gujarat, India,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 943–944.

● **2.4-37** Kiremidjian, A. S. *et al.,* **Seismic hazard mapping for Guatemala,** *26,* John A. Blume Earthquake Engineering Research Center, Stanford Univ., Stanford, California, 1977, 227.

● **2.4-38** Carmona, J. S. and Girardi, J. P., **Seismic risks variation according to sequences of seismic events,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 955–956.

● **2.4-39** Srivastava, A. K. and Jalote, P. M., **Seismicity and tectonic setup of the area around Delhi,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 791–798.

An evaluation of earthquake data in relation to the tectonic arrangement of the area around Delhi was carried out. A statistical analysis of the instrumental data was performed, and strain rebound increments and tectonic flux maps were prepared. These studies reveal that the NE-SW trending minima zone, interpreted on the tectonic flux map, may possibly indicate a zone of fundamental fracture in the basement having the pattern of regional tectonic lineaments and possibly having a genetic relationship with the majority of earthquakes recorded in this area.

2.4-40 Borissoff, B. A., Reisner, G. I. and Sholpo, V. N., **A geotectonic method of predicting the maximum magnitudes of expected earthquakes as applied to the northern Italy area,** *Bollettino di Geofisica,* **XX,** *73-74,* Mar.-June 1977, 19–26.

Relationships previously set in the Caucasus between the tectonic features of a region and the maximum magnitude of expected earthquakes therein are applied to northern Italy. A prediction map for the 10′ by 15′ grid is presented which corresponds with the data and indicates the distribution of seismic areas. The map is compiled based on the data of geologic structure and development of the area.

2.4-41 Vered, M., **Relations between isoseismic area and magnitude for Italian earthquakes,** *Bollettino di Geofisica,* **XIX,** *75-76,* Sept.-Dec. 1977, 290–293.

Relationships between earthquake magnitudes of Italian events and isoseismic areas, for MM intensities V, VI, VII and VIII, are presented. It is shown that these relations differ significantly between southern and northern Italy. It is concluded that, for the same earthquake magnitudes, the extent of the damaged area will be greater in southern Italy than in California, while the seismic areas will be similar.

2.4-42 Okada, M., **Clustering in time and periodicity of strong earthquakes in Tokyo** (in Japanese), *Zisin, Journal of the Seismological Society of Japan,* **30,** *2,* Aug. 1977, 187–199.

The clustering in time and the periodicity of earthquake occurrence are investigated statistically by the use of historical data of strong earthquakes in Tokyo. In order to discuss the periodicity of main shocks, the author examines the homogeneity of historical materials and the distributions of the intervals between any two earthquakes and between two successive earthquakes. It is found that one-third to one-fourth of events can be regarded as foreshocks and aftershocks. Using the Monte Carlo method, it is tested statistically whether the occurrence of main shocks is represented by a stationary random process or a periodic process. As a result, the 69-year periodicity, which is the highest peak in the periodgram calculated from Kawasumi's table, is not statistically significant at a 95% confidence level, but becomes significant if the confidence level is lowered to 80%. From the table of Usami and Hisamoto, the 36-year periodicity, which is the most predominant in their table, is found to be insignificant even at the 80% confidence level.

● **2.4-43** Slemmons, D. B., **State-of-the-art for assessing earthquake hazards in the United States; Report 6, faults**

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and earthquake magnitude, *Misc. Paper S-73-1,* Mackay School of Mines, Univ. of Nevada, Reno, May 1977, 166.

The recency of seismologic and historic records, in combination with the long recurrence intervals between potentially damaging earthquakes, creates a need for developing geologic methods as alternatives or supplements to seismologic methods of establishing design earthquakes. The goal of this report is to review geologic methods of determining the maximum probable earthquakes for active faults based on empirical relationships between magnitude, length of surface faulting, maximum fault displacement, and combinations of fault length and maximum displacement. Empirical studies have developed and refined the formulations and graphs relating earthquake magnitude to surface faulting. The present study updates the most recent studies of the type on the basis of data that have become available over the past six years.

The study required the careful determination of fault activity or nonactivity, rate of activity, and an integrated program of historic, seismologic, geologic, geomorphic, and geophysical studies. The criteria for activity are summarized and methods of resolving questions of activity are provided. The geomorphic evidence for activity and type of fault is summarized in terms of complex, branching, conjugate, and distributed fracture patterns that require a regional and integrated geological and geophysical approach. The relationship of fracture patterns is presented in terms of plate tectonic environments and conjugate deformation by faulting, folding, and warping. The complex pattern analysis required for distributed fracture systems is considered for each fault type—strike-slip, reverse-slip, and oblique-slip—with examples and source literature provided for each type. Case histories are given for the selection of design values.

2.4-44 Basham, P. W., Forsyth, D. A. and Wetmiller, R. J., **The seismicity of northern Canada,** *Canadian Journal of Earth Sciences,* **14,** *7,* July 1977, 1646–1667.

The addition of over 1000 earthquakes to the northern Canadian data file during the past 3 years provides sufficient data to delineate distinctive patterns of seismicity, although the short history of low-level earthquake monitoring and the temporal and spatial clustering of earthquakes suggest that not all potentially active areas may yet have been identified. The data indicate areas of activity near the larger earthquakes located teleseismically prior to the post-1960 northern expansion of the Canadian Seismograph Network and additional clusters and trends that were not previously apparent. Correlations to seismicity with major deformational trends in the Yukon-Mackenzie Valley, the northern continental margin, the Arctic archipelago, and encircling much of the Baffin Island–Foxe Basin area show that structures formed or reactivated by Paleozoic and later orogenic phases are continuing activity in response to the contemporary stress field. Possible zones of Cenozoic movement show pockets of high seismic activity but important gaps in the trends remain off Banks Island, along Nares Strait, and in Davis Strait. Tectonic forces characteristic of plate margins do not appear to be acting in the Canadian Arctic, and contemporary movement of Greenland with respect to Baffin Island does not need to be invoked to explain the seismicity in the Baffin region.

Epicenter clusters in the Beaufort Sea and offshore of Ellef Ringnes Island are distributed mainly over the seaward gradient of elliptically shaped free air anomalies, indicating seismic adjustment in basement structures to uncompensated wedges of recent tertiary sediments. Seismicity around much of the Baffin Island–Foxe Basin block shows a significant correlation with the interval of isostatic equilibrium between broad areas of current postglacial uplift. If northeastern Baffin Island is a hinge zone in the rebound process, and the zone from Hudson Strait to northeastern Keewatin a line of inflection in the rate of uplift contours, the reactivation of structures is occurring along zones of high differential stress.

2.4-45 Sbar, M. L. and Sykes, L. B., **Seismicity and lithospheric stress in New York and adjacent areas,** *Journal of Geophysical Research,* 82, *36,* Dec. 10, 1977, 5771–5786.

The earthquake distribution obtained by a relatively dense network of short-period high-gain stations in New York and adjacent areas from 1971 to mid-1976 shows nearly all the features of seismicity maps from standard listings from 1928 to 1975. In addition, the network data show a region of significant activity in the Adirondack Mountains of northern New York and several sizeable earthquakes in western Lake Ontario. In both areas, few or no earthquakes were previously reported. Both epicentral maps show a region of low seismicity in the Vermont section of the controversial Boston-Ottawa seismic zone. Well-located earthquakes in the northeastern United States and southeastern Canada range in depth from near surface to about 25 km. Deeper earthquakes (>10 km) have not been found throughout the region but may be localized in those areas where large earthquakes have occurred. They may also be indicative of deep fault zones which have a greater potential for the occurrence of future large earthquakes. Single events, main shock-aftershock sequences, and swarms are found in this region. It is not yet understood why particular types of earthquake sequences occur in specific areas. In many cases, aftershock sequences or earthquake swarms can be related to mapped faults or inferred extensions of faults. Recent stress indicators in New York and adjacent areas support the presence of an ENE-trending maximum compressive stress in the lithosphere west of the Appalachian Mountains in New York, Ohio, and southern Ontario. The eastern boundary of this stress domain is suggested by limited data in northern New

England, southeastern New York, and New Jersey. It is postulated that variations in the stress field, which may exist near the eastern boundary of this stress domain, and the general lack of suitably oriented faults in Vermont may explain the region of low seismicity in Vermont along the proposed Boston-Ottawa seismic zone.

● 2.4-46 O'Rourke, M. J. and Solla, E., **Seismic risk analysis of Latham Water District, Albany, New York,** *SVBDUPS Project Report No. 3,* Dept. of Civil Engineering, Rensselaer Polytechnic Inst., Troy, New York, 1977, 59.

As a first step in developing a systematic way of assessing the adequacy of the Latham Water Distribution System to withstand seismic excitation, this report determines the seismic risk of the Albany, New York, area. First, Richter's relationship and the average occurrence rate for the Albany area are established from available data of earthquakes in the northeast. The coefficients for the attenuation relationship are estimated by a search of current literature. A standard deviation of the error term in the attenuation relationship is varied and the resulting return periods and annual risk are compared with results obtained using a deterministic attenuation relation. Finally, using these parameters, recommended return periods and seismic risk for the Albany area are presented.

2.4-47 Calcagnile, G. *et al.*, **Seismic risk of Basilicata in its geostructural frame,** *Bollettino di Geofisica,* **XIX,** *75-76,* Sept.-Dec. 1977, 117–139.

The activity of a well-defined seismotectonic region is analyzed, taking into account its geostructural frame. Several statistical parameters are obtained, indicating a severe seismic risk for the western side of the investigated area. A geostructural map is given; this combined with the composite isoseismal map is the basic information for correct microzoning planning.

2.4-48 Fletcher, J. B. and Sykes, L. R., **Earthquakes related to hydraulic mining and natural seismic activity in western New York State,** *Journal of Geophysical Research,* 82, *26,* Sept. 10, 1977, 3767–3780.

The seismic activity of western New York has been monitored since 1970, by eight permanent seismograph stations and several portable stations. The investigation centered on the Attica-Dale area where several damaging earthquakes have occurred in this century. Although the background level of seismicity was found to be extremely low, less than one event per month in the first 12 months of the study, a sharp increase in seismicity took place in 1971 at the site near Dale following the initiation of fluid injection under high pressure (120 bars tophole) at a hydraulic mining operation nearby. This facility, which mines salt from the Vernon formation of Silurian age, is centered near the Clarendon-Linden fault, a major north-south trending system of high-angle thrust faults that extends for over 100 km from Lake Ontario to Allegheny County, New York. Although the seismic events were small (none were recorded at our station 30 km to the northwest), as many as 80 occurred per day, and many were felt locally. The marked increase in seismic activity after attaining high pressures, the closeness of these events to the bottom of the injection well, and the near cessation of activity within 48 hours of the shutdown of injection strongly suggest that this activity was caused by the triggering of tectonic strain on or near the Clarendon-Linden fault by the high fluid pressures of the mining operation. The minimum pressure (41-48 bars) at which seismic activity occurred is consistent with predictions made by applying the Hubbert-Rubey theory of effective stress to hydrofracturing stress measurements from Alma, New York. Since 1972, five other wells at Dale have been hydrofractured at pressures of about 110 bars, but none had abrupt changes in seismic activity associated with them during the high-pressure phases of pumping. The well used in 1971 (0.43 km deep) is the closest (about 50 m) of the six to the Clarendon-Linden fault. It was hydrofractured near the base of the salt layer, whereas the others were hydrofractured well within the salt layer. Thus fluids under high pressure appear to have reached the Clarendon-Linden fault in 1971 but not in the other five cases. A focal mechanism involving thrust faulting on a plane nearly parallel to the Clarendon-Linden fault was obtained from events that continue to occur but at a much reduced level in the brine field in 1974 and 1975. An earthquake of magnitude 2.7, which occurred about 7 km to the west of the brine field in 1973, is apparently unrelated to the injection operation and is of natural origin. This and other nearby natural events appear to be associated with the western and southwestern branches of the Clarendon-Linden fault. Hydraulic diffusivity values calculated from the space-time relationship of earthquakes triggered by fluid injection at Denver and Rangely, Colorado, Matsushiro, Japan, and Dale are about 10^4 and 10^5 cm^2/s and are similar to those obtained from precursory anomalies of earthquakes. The similarity of the diffusivity values suggests that the precursory changes do involve the movement of water. In each of these four places, the wells used for fluid injection bottomed into or very close to major fault zones.

2.4-49 Knopoff, L. and Kagan, Y., **Analysis of the theory of extremes as applied to earthquake problems,** *Journal of Geophysical Research,* 82, *36,* Dec. 10, 1977, 5647–5657.

Procedures of the theory of extremes give unacceptably large probable errors in determinations of return times, b values, and maximum magnitudes of large- and intermediate-magnitude earthquakes using limited data runs. On all accounts, methods which utilize all available data give superior estimates of the seismicity parameters

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than do extreme value methods and provide for a procedure for handling inhomogeneous catalogs as well.

● **2.4-50** Meyers, H., **A historical summary of earthquake epicenters in and near Alaska,** *NOAA Technical Memorandum EDS NGSDC-1,* National Geophysical and Solar-Terrestrial Data Center, U.S. National Oceanic and Atmospheric Admin., Boulder, Colorado, Apr. 1976, 76.

This publication summarizes the Alaska earthquake data file as developed by the National Geophysical and Solar-Terrestrial Data Center. Additional data are being added for current and historical earthquakes as they are prepared for machine processing. The publication describes data formats, sources used in developing the data file, and data limitations. Included are several tables which summarize the data in usable forms. The publication also is designed to accompany magnetic tapes, microfilm, and printouts produced from the data file.

● **2.4-51** Friedman, M. E. *et al.*, **Seismicity of the southern California region: 1 January 1972 to 31 December 1974,** Seismological Lab., California Inst. of Technology, Pasadena, 1976, 91.

This listing of earthquakes and epicenter maps is an update to the southern California local earthquake catalog published in *Seismicity of the Southern California Region 1 January 1932 to 31 December 1972* by Hileman et al. Earthquakes in 1972 are relisted here because of numerous additions that resulted from an intensive study of seismicity in the Los Angeles basin. The basic seismological data was obtained from the southern California network of the Seismological Laboratory. Since 1974, earthquakes have been routinely located using the computer program HYP071 written by Lee and Lahr. A four-layered velocity model for southern California determined by Kanamori and Hadley has been used for the majority of epicenter solutions. Several special studies with epicenter locations included in this listing use a velocity model modified for the specific area considered. Prior to 1974, most of the locations were computed as described in Hileman et al.

The Seismological Laboratory attempts to identify and locate all earthquakes of magnitude 3.0 and greater that occur within the area shown on an included map, with the exception of shocks south of the international border within Mexico which are systematically located only when they exceed magnitude 4.0. This coverage is felt to be nearly complete. Many shocks of less than magnitude 3.0 are given in this listing, but no claim is made for the statistical homogeneity of these smaller shocks. Every effort has been made to exclude from the listing nonseismic events such as quarry and mine blasts.

2.4-52 Seno, T., **Recurrence times of great earthquakes in the seismotectonic areas along the Philippine Sea side coast of southwest Japan and south Kanto district** (in Japanese), *Zisin, Journal of the Seismological Society of Japan,* 30, *1,* Apr. 1977, 25-42.

The seismic crustal movement in the south Kanto district and on the Philippine Sea coast of southwest Japan is characterized by chronic tilting (during an interseismic period) and acute tilting (during an earthquake). The accumulation of this tilting in a geological time scale explains the topographic features on the coast such as tilted terraces. The value *d*, which is defined as the ratio of the subsidence in the interseismic period to the coseismic upheaval, was expressed by two independent linear functions of the recurrence interval *T* of a series of great earthquakes. The coefficients of these functions are determined by the data of precise levelings and the heights of coastal terraces in the late Quaternary period. Then, the *d* and *T* can be determined on the cross point of the two lines which represent the above two functions. This scheme was applied to the seismotectonic areas of southern Kanto and the outer arc of southwest Japan. The results of *T* are almost in concordance with the intervals of great earthquakes inferred from historical documents and the rates of convergence of the Philippine Sea plate under southwest Japan. It was also shown that this scheme can be used as a tool for earthquake prediction with the Tokai region as an example.

2.4-53 Seno, T., **Recurrence time intervals of great earthquakes in the Sagami trough region, Japan, as deduced from vertical crustal movement** (in Japanese), *Zisin, Journal of the Seismological Society of Japan,* 30, *3,* Dec. 1977, 253-264.

In a previous paper, the method of calculation of the recurrence time interval of great earthquakes in the Nankai and the Sagami trough regions was improved and refined. The recurrence time interval *T* and the value *d* (so-called recovery ratio) were estimated at several sites in the seismic crustal movement areas along these troughs. In the present paper, the vertical crustal movement observed by precise levelings and the deformation of the Holocene terraces in the south Kanto district are reexamined, because the relationship of the deformation of the coastal terraces to the seismic crustal deformations in this region is complicated. In the Shonan district, the deformation of the Holocene terraces is attributed to the crustal deformation associated with the 1923 Taisho-type earthquake; the recurrence time of *T* of 180-400 years and the value *d* of 0.25-0.50 are obtained. In the southern Boso peninsula the deformation of the Holocene terraces is divided into the crustal deformation associated with the 1923-type earthquakes, and that caused by the fault off the Boso peninsula which moved at the time of the 1703 Genroku earthquake. The recurrence time *T* and the value *d* for this fault off the

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Boso peninsula are estimated to be 950–2500 years and 0.20–0.54, respectively.

2.4-54 Crescenti, U. *et al.*, **Ancona: seismotectonic considerations** (Ancona: considerazioni sismo-tettoniche, in Italian), *Bollettino di Geofisica*, **XX**, *73-74*, Mar.-June 1977, 33–48.

In this paper are presented the results of the first stage of studies of the seismicity of the Ancona region. The final result will be a microzonation map of the Ancona area, based on recorded events.

2.5 Studies of Specific Earthquakes

2.5-1 Shteinberg, V. V. *et al.*, **Ground motion in the strong earthquakes of 1971 on the Kamchatka peninsula** (Kolebaniya grunta pri silnykh zemletryaseniyakh na Kamchatke v 1971 g, in Russian), *Silnie Kamchatskie temletryaseniya 1971 g*, n.p., Vladivostok, 1975, 7–14.

Records of ground motion parameters for the two strong earthquakes of 1971 (Petropavlovsk-Kamchatskii, Nov. 24 and Ust-Kamchatsk, Dec. 15) are presented. The instrumentation used is described and the accuracy of the recordings is analyzed.

● 2.5-2 Scott, R. F., **The Essex earthquake of 1884**, *Earthquake Engineering and Structural Dynamics*, **5**, *2*, Apr.-June 1977, 145–155.

The largest British earthquake to occur in several hundred years took place in Essex, in southeast England, in 1884. From contemporary accounts as well as a recent visit to the area affected by the earthquake, the intensity of the earthquake was assessed. It is estimated that the peak accelerations generated may have been as much as 0.1 g. This would be of consequence to some modern structures designed without earthquake provisions.

● 2.5-3 Vered, M. and Striem, H. L., **A macroseismic study and the implications of structural damage of two recent major earthquakes in the Jordan rift**, *Bulletin of the Seismological Society of America*, **67**, *6*, Dec. 1977, 1607–1613.

A detailed macroseismic study of the July 11, 1927, earthquake was carried out. A quantitative analysis of damage data provided a correlation for estimating intensities (MM): I = 6.4 + 1.2 log (percentage of damaged houses). Using axis lengths and areas bounded by the ensuing isoseismal lines, the depth (16 to 28 km) of the event was estimated, and its probable epicenter located near Damiya Bridge on the Jordan River. A comparison with the equivalent parameters, inferred from instrumental records, shows agreement between both sets of results and thus confirms the validity of the approach used in the macroseismic study.

The same procedure of studying macroseismic data was applied to an earlier (Jan. 1, 1837) destructive earthquake. It was found that this latter event originated in the upper crust, eastward of Safed, with a 6.25 to 6.5 magnitude.

The isoseismals of both these major earthquakes are elongated in a north-south direction along the major structural trend in the area. The southern coastal plain of Israel seems generally less vulnerable to Jordan rift valley earthquakes than inland regions of similar epicentral distances, though local pockets of anomalous intensities are observed for both earthquakes.

● 2.5-4 Herrmann, R. B., Fischer, G. W. and Zollweg, J. E., **The June 13, 1975 earthquake and its relationship to the New Madrid seismic zone**, *Bulletin of the Seismological Society of America*, **67**, *1*, Feb. 1977, 209–218.

The June 13, 1975, earthquake in the New Madrid seismic zone produced the first recorded strong-motion accelerograms for an event in the region, as well as the largest recorded accelerations to date for any event in eastern North America. The peak strong-motion values obtained from an analysis of the accelerograms are presented. Source parameter estimation using long-period surface waves, *Lg* spectra, *P*-wave first motions, and the integrated accelerograms leads to a consistent solution. The seismic moment is estimated to be 4E21 dyne-cm and the corner period 0.6 sec. The corner period-seismic moment pair for this event agrees with the regional scaling of these parameters observed by Street et al.

● 2.5-5 Einarsson, P., Klein, F. W. and Bjornsson, S., **The Borgarfjordur earthquakes of 1974 in west Iceland**, *Bulletin of the Seismological Society of America*, **67**, *1*, Feb. 1977, 187–208.

The earthquakes of 1974 in Borgarfjordur occurred west of the mid-Atlantic plate boundary in Iceland. The earthquake sequence lasted more than 2 months and was a combination of swarm activity and a foreshock-mainshock-aftershock sequence. The largest earthquake of the sequence occurred on June 12, 1974, and was of m_b = 5.5. No change in the *b*-value with time could be detected. An array of nine short-period seismometers was operated in the epicentral area for about 3 weeks. The main epicentral zone, as defined by 563 accurately determined hypocenters, is about 4 km wide, 25 km long and trends nearly east-west. The depth of most hypocenters is between 0 and 8 km. No earthquake was located below the crust-mantle boundary.

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The mainshock on June 12 probably occurred near the center of the epicentral zone where it is intersected by a secondary zone which has a northeast trend. A focal mechanism solution of the largest shock indicates normal faulting. One nodal plane of this solution strikes NE, which matches the secondary epicentral trend. Focal mechanisms in the western end of the epicentral zone also indicate normal faulting. Nodal planes of these focal solutions, the trend of epicenters, and observed surface fault breaks all have an E-W trend in the western part of the seismic zone. The Borgarfjordur area thus seems to be undergoing horizontal extension. The minimum compressive stress rotates from a horizontal northwesterly orientation in the center of the epicentral zone to a horizontal north-south direction in the western branch. This extension may be caused by a mantle plume under Iceland or by thermal contraction of the lithospheric plates as they move away from the accreting plate boundary.

● 2.5-6 Bolt, B. A., Stifler, J. and Uhrhammer, R., **The Briones Hills earthquake swarm of January 8, 1977, Contra Costa County, California,** *Bulletin of the Seismological Society of America,* **67**, *6,* Dec. 1977, 1555–1564.

Locations and magnitudes of a swarm of earthquakes in the hills east of San Francisco Bay have been estimated. The largest (M_L = 4.3 ± 0.13 S.E.M.) event had an origin time of 09h38m07.5s ± 0.11s, Jan. 8, 1977 UTC, a latitude of 37°54.31′ ± 0.69′N and longitude 122°10.97′ ± 0.50′W and a focal depth of 9.5 ± 0.8 km. The *P* fault-plane solution for earthquakes in the swarm was consistent with right-lateral slip on a plane with strike N28°W and dip 65°SW. No surface rupture was observed, but the mechanism is consistent with present regional deformation. Displacement seismograms recorded at Berkeley show a close similarity of wave form but the *P/S* amplitude ratios for the foreshocks are systematically greater than for the main shock and subsequent events. The calculated relative seismic energy suggests a very limited region of successive slip surfaces.

● 2.5-7 Espinosa, A. F., **Particle-velocity attenuation relations: San Fernando earthquake of February 9, 1971,** *Bulletin of the Seismological Society of America,* **67**, *4,* Aug. 1977, 1195–1214.

Accelerograms recorded at 61 stations during the San Fernando earthquake were integrated and used to determine the resultant horizontal and the vertical particle velocity as a function of distance. The attenuation of the average value of particle velocity, in the period range from 0.28 to 1.4 sec, is described by the relationship: $\log_{10}v$ = 3.19 – 1.35 $\log_{10}\Delta$. The attenuation of modified Mercalli intensity with distance and the horizontal particle velocity are $\log_{10}\Delta$ = 3.05 – 0.25 I and $\log_{10}v$ = –0.93 + 0.29 I. Attenuation relations obtained in the frequency domain for the horizontal and vertical pseudo-spectral velocities of 3-sec-period waves are $\log_{10}v^H = 3.68 - 1.25 \log_{10}\Delta$ and $\log_{10}v^V = 2.91 - 1.03 \log_{10}\Delta$, respectively. Other relations for different spectral components are also presented.

The ratios of horizontal to vertical pseudo-spectral velocities (v^H/v^V) are 3.5 for a 3-sec-period wave at Δ = 10 km, and 3.2 for a 1-sec period wave in the distance range from 10 to 100 km. Comparison between the particle-velocity attenuation curve found in this study and that determined for a design earthquake for the central U.S. having m_b = 7.2 shows a difference of one order of magnitude in the level of motion at T = 3.0 sec, and the attenuation for m_b = 6.2 and T = 3.0 sec agrees fairly well with the observed particle-velocity mean attenuation curve derived from the integrated accelerograms of the San Fernando earthquake, m_b = 6.2 (NEIS). The absorption coefficient for a T = 0.1-sec wave is σ = 0.0225/km. The horizontal pseudo-spectral velocity mean curves show a similar rate of attenuation for periods between 0.1 to 3.0 sec, suggesting a decay rate for the displacement-source spectrum of ω^{-1} in this frequency range.

● 2.5-8 Chaudhury, H. M. and Srivastava, H. N., **The Kinnaur earthquake of January 19, 1975 and its aftershocks,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 242–248.

Among the recent destructive earthquakes in India, the Kinnaur earthquake of Jan. 19, 1975, is of special significance. In this paper, this earthquake and its aftershocks, recorded to the end of Mar., 1975, are discussed. Based on well-determined epicentral parameters of the aftershocks of magnitudes greater than 4.5, a two-layered model is derived to determine the crustal structure of the region. The frequency-magnitude relationship of the aftershocks is considered. The focal mechanism of the main earthquake is studied and discussed in relation to the spatial distribution of the aftershocks and the trend of the isoseismals.

● 2.5-9 Nielsen, N. N. *et al.,* **Honomu, Hawaii earthquake of April 26, 1973,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 227–232.

The Richter magnitude 6.2 earthquake was deep seated (50 km). A strong-motion accelerograph was located about 50 km from the epicenter. The maximum recorded acceleration was 0.17 g, and the duration of the strong motion was short (7 sec). Sensitive aftershock equipment was installed on two different types of soil, hard lava rock and soft volcanic ash. The aftershock records were analyzed and response spectra were calculated. The response, for most frequencies, was considerably higher for the volcanic ash. Total amount of damage was $6,000,000. A significant portion of the damage was to roads and bridges.

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● **2.5-10** Gosavi, P. D., Bapat, A. V. and Guha, S. K., **Macroseismic studies of four recent Indian earthquakes,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 233-241.

From Oct. 17, 1973, to July 8, 1975, four medium-sized earthquakes occurred in India. These earthquakes were extensively studied through field damage surveys and the responses to questionnaires. A brief description of these earthquakes with reference to various seismological and geotectonic conditions is presented.

2.5-11 Hill, R. L. and Beeby, D. J., **Surface faulting associated with the 5.2 magnitude Galway Lake earthquake of May 31, 1975: Mojave Desert, San Bernardino County, California,** *Geological Society of America Bulletin,* 88, *10,* Oct. 1977, 1378-1384.

An earthquake of magnitude 5.2 occurred on May 31, 1975, near Galway Lake in the Mojave Desert, San Bernardino County, California. Preliminary information from the Seismological Lab. at the California Inst. of Technology placed the hypocenter at a depth of 5.8 km with the epicenter at lat. 34°31.4'N and long. 116°29.3'W. This event was accompanied by surface rupture in a zone 6.8 km long and as much as 100 m wide, trending N25°W to due north along an existing but unmapped fault. Field evidence and aftershock data by others indicate that the fault is vertical. Left-stepping in echelon fractures indicate predominant right-lateral displacement. Individual fractures are vertical and have right slip, generally from 0.2 to 0.5 cm up to a maximum of 1.5 cm. Shallow pits dug across surface faults in Quaternary alluvium revealed plant roots concentrated in fractures indicating that surface rupture occurred along these fractures during at least one previous event. These and other features, combined with topographic evidence, confirm previous Quaternary displacement and may indicate at least one previous event accompanied by surface rupture in Holocene time. This rupture zone has been named the Galway Lake fault zone.

● **2.5-12** Khattri, K. N., Saxena, A. K. and Sinvhal, H., **Determination of seismic source parameters for the 1967 earthquake in Koyna Dam region, India, using body wave spectra,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 655-664.

The spectra of teleseismic body wave signals are found to show characteristic features predicted by the Brune earthquake model and attest to the tectonic origin of the Koyna Dam earthquake. On the basis of these spectra, the seismic moment is found, and the source dimensions, stress drop, and seismic energy are estimated.

● **2.5-13** Tilling, R. I. *et al.,* **Earthquake and related catastrophic events, Island of Hawaii, November 29, 1975: a preliminary report,** *Circular 740,* U.S. Geological Survey, Arlington, Virginia, 1976, 33.

The largest earthquake in over a century—magnitude 7.2 on the Richter Scale—struck Hawaii the morning of Nov. 29, 1975, at 0448. It was centered about 5 km beneath the Kalapana area on the southeastern coast of the island (lat. 19°20.1' N, long. 155°01.4' W). The earthquake was preceded by numerous foreshocks, the largest of which was a 5.7-magnitude jolt at 0336 the same morning, and was accompanied, or closely followed, by a tsunami, massive ground movements, hundreds of aftershocks, and a volcanic eruption.

The tsunami reached a height of 12.2 to 14.6 m above sea level on the southeastern coast about 25 km west of the earthquake center. Elsewhere it was generally 8 m or less. The south flank of Kilauea volcano, which forms the southeastern part of the island, was deformed by dislocations along old and new faults along a 25-km long zone. Downward and seaward fault displacements resulted in widespread subsidence, locally as much as 3.5 m, leaving coconut palms standing in the sea and nearly submerging a small, near-shore island. A brief, small-volume volcanic eruption, triggered by the earthquake and associated ground movements, occurred at Kilauea's summit about three-quarters of an hour later. The earthquake, together with the tsunami it generated, caused severe property damage in the southeastern part of the island; the tsunami also caused two deaths. Damage from the earthquake and related catastrophic events is estimated by the Hawaii Civil Defense Agency at about $4.1 million.

The 1975 Kalapana earthquake and accompanying events represent the latest events in a recurring pattern of behavior for Kilauea. A large earthquake of about the same magnitude, tsunami, subsidence, and eruption occurred at Kilauea in 1868, and a less powerful earthquake and similar related processes are believed to have occurred in 1823. Indeed, the geologic evidence suggests that such events have been repeated many times in Kilauea's past and will continue. The 1975 events serve as a reminder of the dynamic nature of the volcano and point up the need for careful land-use planning and adequate building codes to minimize damage and loss of life from similar events in the future. Detailed scientific study of the cause and effects of the Nov. 29, 1975, event will take many months. This report summarizes information available in Feb. 1976.

2.5-14 Tsukuda, T., Nakamura, K. and Kishimoto, Y., **The earthquake on September 21, 1973 in the vicinity of the Yamasaki fault and its aftershock activity** (in Japanese), *Zisin, Journal of the Seismological Society of Japan,* **30**, *2,* Aug. 1977, 151-162.

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On Sept. 21, 1973, an earthquake of magnitude 5.1 occurred in the vicinity of the Yamasaki fault in the western Kinki district. In this region, it was the largest event since the occurrence of the earthquake swarm of 1961. Using data obtained at three temporary observation stations and routine stations of the Tottori Microearthquake Observatory, hypocenters of the main shock-aftershock sequence were determined. Three large shocks ($M>3$), including the main shock, have their foci concentrated in a small volume of 1-2 km in diameter at a depth of 11 km, where many aftershocks are also located. Another concentrated region of aftershocks is located 2 km south of the first one. Consequently, the size of the aftershock area is about 2 km x 3 km, which is roughly consistent with Utsu-Seki's formula. The focal mechanism of the main shock is of a thrust type, and its maximum pressure axis lies nearly horizontal and in a N64°E direction. Such a mechanism has not been found in the Yamasaki fault area where events of strike-slip are large in number. Major aftershocks are estimated to have different types of mechanisms as compared to the main shock. This implies some complexity of focal process or faulting for this earthquake.

2.5–15 Everingham, I. B., Gaull, B. and Dent, V., **Effects of a major earthquake near Bougainville, 20 July 1975,** *BMR Journal of Australian Geology & Geophysics*, 2, 1977, 305–310.

On July 20, 1975, a major earthquake (MS 7.9) shook the northern islands of the Solomon Islands chain. Damage amounting to at least $300,000 (Australian) occurred in the southern Bougainville/Shortland Islands region, where earthquake intensities were estimated to be MMVII-VIII. A tsunami with maximum amplitude of about two meters followed the earthquake and caused further damage. The earthquake caused landsliding, liquefaction, subsidence, slumping of roads and wharfs, and damage to villages, small government and mission buildings, and to the mining installations at Panguna. Aftershock epicenters were in a roughly elliptical area of 12,500 sq km off the southwestern coast of Bougainville. Focal depths were in the range of 30-70 km. A fault-plane solution and the pattern of aftershocks indicate that the principal earthquake was associated with underthrusting of the Solomon Sea crust beneath Bougainville, in a northeasterly direction and with a dip of about 37°. The faulting associated with the earthquake appears to be the extension of faulting associated with a 1974 earthquake series. An aseismic zone, centered at 6°S,154°E, exists immediately northwest of the 1975 earthquake fault zone, between zones where major earthquakes have occurred since 1970. It is considered to be a likely place for a major earthquake in the future.

2.6 Seismic Water Waves

● **2.6–1** Garcia, A. W., **Effect of source orientation and location in the Peru-Chile trench on tsunami amplitude along the Pacific Coast of the continental United States,** *Research Report H-76-2*, Hydraulics Lab., U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, Sept. 1976, 50.

An idealized axis of the Peru-Chile trench was divided into 12 segments of equal length. A hypothetical bottom displacement that generates a tsunami with an intensity approximately equal to four was centered in three of the segments. An explicit finite difference numerical code was used to simulate generation and propagation of the resulting tsunami to the west coast of the continental United States. Additionally, the tsunami of May 22, 1960, was simulated, and a comparison was made to gage records at selected open coast locations along the U.S. Pacific Coast. Contour plots of surface elevation of the few leading waves of the tsunami at selected times are presented. An analytical technique is used to normalize the amplitude of the leading wave of the tsunami to its amplitude at a 600-ft water depth. For purposes of comparison, the amplitudes of the tsunamis generated in each segment are plotted as a function of distance along the Pacific Coast from the Mexican to the Canadian borders. These plots allow an evaluation of the relative vulnerability of coastal locations to tsunamis generated in different locations of the Peru-Chile trench.

● **2.6–2** Pararas-Carayannis, G., **Catalog of tsunamis in Hawaii,** revised by Calebaugh, J. P., *SE-4*, World Data Center A for Solid Earth Geophysics, National Oceanic and Atmospheric Admin., Boulder, Colorado, Mar. 1977, 78.

This catalog is a revision of *Catalog of Tsunamis in the Hawaiian Islands*, which was issued in 1969 by World Data Center A Subcenter for Tsunami Data when it was a separate activity in Honolulu operated by the former U.S. Coast and Geodetic Survey. The subcenter was moved to Boulder in 1974 and became part of World Data Center A for Solid Earth Geophysics, sharing the data service staff and facilities of seismology and other related geophysical disciplines. This center is operated in parallel with the corresponding national data services by the National Geophysical and Solar-Terrestrial Data Center of NOAA's Environmental Data Service.

To the original manuscript, a total of 24 new tsunami entries have been added, including 11 tsunamis reported for the period 1968–74 (there were none reported for 1970 and 1972), together with necessary additions to the references. Included are a map of Hawaii with the locations of the places named in the catalog and a chart of the Pacific area with travel times for tsunamis to Hawaii from any place of origin in the Pacific.

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2.6-3 Aida, I., **Relation between tsunami inundation heights and water surface profiles on a 200 m depth contour** (in Japanese), *Zisin, Journal of the Seismological Society of Japan*, 30, *1*, Apr. 1977, 11-23.

Reliability of the tsunami source model inferred from the dislocation theory of faulting was approximately verified previously, comparing the result of a numerical experiment with tsunami records on a coast. On the basis of these results, water surface profiles of a 200 m depth contour have been computed numerically by use of the tsunami source model obtained from the fault parameters of the Tokachi-oki earthquakes of 1952 and 1968. Computed water elevation-time histories indicate the significant character for the directivity of the wave radiation from the source, that is the rise or fall in the leading wave and in the wave period. The wave heights (H_0) vary depending on locations on a 200 m depth contour. Tsunami inundation heights (R) on a coast and H_0 are plotted along the Pacific coast of the Tohoku and Hokkaido districts. Then, average inundation heights (R_{avg}), which are used instead of the rather scattered (R), are fairly parallel to H_0 and R_{avg}/H_0 = 2~3. It is recognized by a simple calculation that the values of R_{avg}/H_0 are reasonable as the amplification factor of water oscillations on a continental shelf. The tsunami source model based on the dislocation theory of faulting seems to give results fairly consistent with the major character of a tsunami, at least, on the Pacific coast of the Tohoku district. Thus, this kind of numerical experiment may be a useful method for prediction of tsunami inundation on a coast.

2.6-4 Everingham, I. B., **Preliminary catalogue of tsunamis for the New Guinea/Solomon Islands region, 1768-1972**, *180*, Australia, Bureau of Mineral Resources [Canberra], n.d. (Available as Bureau of Mineral Resources microform MF14.)

A preliminary catalog of 65 tsunamis in the New Guinea/Solomon Islands region between 1768 and 1972 has been compiled from descriptions and basic data from many sources, including eye-witness accounts. The tsunamis were triggered by local earthquakes (54), distant earthquakes (4), volcanism (3), and submarine slumping (1). They were more frequent along the northern New Guinea and Solomon Sea coastlines than elsewhere in the region, and most had maximum heights of 2 m or less; only 14 of the tsunamis had maximum heights of 4 m or more and were potentially dangerous. Buildings constructed 3 m above highest tide level should be safe from the damaging effects of most tsunamis.

2.7 Artificially Generated Ground Motions or Seismic Events

● 2.7-1 Awojobi, A. O. and Sobayo, O. A., **Ground vibrations due to seismic detonation of a buried source**, *Earthquake Engineering and Structural Dynamics*, **5**, *2*, Apr.-June 1977, 131-143.

The problem of vibrations at the free surface of an elastic halfspace due to detonation of a buried source is studied from the viewpoint of the geophysical seismic technique employed in oil exploration. The fundamental assumption of the theory, therefore, is that the pressure at the source is impulsive and a Dirac delta function of time. The depth of the source below the surface of the medium is considered as an additional parameter which has hitherto made the buried source problem formidable and has limited almost all previous work to the relatively easy problem of surface blasting.

An exact formulation is presented by dividing the halfspace into a stratum above the level of the buried source and a halfspace below it. For boundary conditions at the interface, it is assumed that the vertical displacement and shear stress at the level of the source are continuous while the direct stress is discontinuous. A numerical evaluation of the contour integration in the analysis is presented for the components of vibrations at any point of the free surface for a Poissonian medium. Finally, the analysis provides a theoretical justification for the interesting results obtained from field experiments recorded in an earlier work.

● 2.7-2 Daly, W., Judd, W. and Meade, R., **Evaluation of seismicity at U.S. reservoirs**, U.S. Committee on Large Dams, Committee on Earthquakes, Pasadena, California, May 1977, 27.

This paper summarizes the current state of knowledge regarding reservoir-associated seismicity and evaluates the results of the survey of seismicity at large dams in the United States.

● 2.7-3 Hannon, W. J., Rodean, H. C. and Barnett, C. S., **Earthquake triggering by earthquakes and nuclear explosions at rates of less than 1°/day**, *UCRL-52096*, Lawrence Livermore Lab., Univ. of California, Livermore, July 6, 1976, 47.

The authors have examined seismicity data and teleseismic distances for one-year periods following ten selected nuclear explosions using a parameter called the "rate" (the distance from a selected "trigger" event to a subsequent "triggered" event divided by the time interval

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between the two events). The authors were unable to distinguish the distribution of rates in the range 0.01 to 1.00°/d following the nuclear explosions from distributions which they calculated for years without explosions. They observed no evidence for earthquake triggering by explosions in the previously unexamined time and distance range.

Additional work suggests that the features of the distributions of rates observed in the case of earthquakes being triggered by other earthquakes at these slow rates can be directly related to geometric constraints imposed by the fact that the earthquakes occur in relatively fixed locations along plate boundaries. The authors question the inference of causal triggering at rates of 0.01 to 1.00°/d at teleseismic distances.

● 2.7-4 Johnson, S. J., Krinitzsky, E. L. and Dixon, N. A., **Reservoirs and induced seismicity at Corps of Engineers projects,** *Misc. Paper S-77-3,* U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, Jan. 1977, 18.

A review was made of major reservoirs of the Corps of Engineers with regard to their experiences with induced seismicity. The Corps has 24 large reservoirs, i.e. those with dams 60.96 m (200 ft) in height or greater, and with volumes of 1233.5 x 10^6 m^3 (1,000,000 acre-ft) or greater. There also are 10 dams that are 91.44 m (300 ft) high, or higher, but with reservoirs of less than 1233.5 x 10^6 m^3. Thirteen reservoirs have been instrumented for monitoring microearthquakes. They range from large to small and are located in aseismic to highly seismic areas. Only one Corps reservoir, Clark Hill in Georgia-South Carolina, has experienced a felt earthquake, but the 21-year interval between impoundment and the occurrence of the earthquake is not typical of reservoir-induced earthquakes and a direct relationship appears improbable.

● 2.7-5 Patrick, D. M., **Microearthquake monitoring at Corps of Engineers facilities,** *Technical Report S-77-2,* Soils and Pavements Lab., U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, Feb. 1977, 90.

Awareness of potential seismicity induced by the filling of large reservoirs or by the injection of fluids in boreholes has prompted the Corps of Engineers and others to conduct microseismic monitoring programs at ten dam sites and at one injection well. Two other dam sites have been monitored by other organizations in cooperation with the Corps of Engineers and one is proposed. These programs were developed to measure and compare ambient seismicity in the environs of the installations and the seismicity present during either reservoir filling or fluid injection. These programs have been performed chiefly under contract with the U.S. Geological Survey and academic institutions. Monitoring consists of judiciously positioning an array of three to eight short-period, vertical component seismometers around the reservoir or injection well. Earthquake events detected by the seismometers are amplified at the site and either modulated there or at a location central to the array. The modulated signals are telemetered to a removed location where demodulation, recording, and data analyses are performed. These programs have been designed to monitor for a period of four to five years in order to include pre-, during-, and post-filling or injection phases. The average cost per year of monitoring is approximately $35,000. Equipment purchase, telemetry, and data analysis are the principal expenses. Analysis of available data from completed or nearly completed programs reveals no definite indications of seismicity induced by reservoir filling or fluid injection at Corps of Engineers installations. There is some indication that reservoir pool level may induce low-level seismicity at one site in the southeastern U.S., although there is other evidence that this may not be true. Monitoring programs must be based upon firm geological and seismological understanding of the installation sites. Arrays should be so situated, and consist of sufficient instruments, that accurate epicentral locations, focal depths, and magnitudes may be determined. Duration of monitoring must be sufficiently long so that a reliable statistical significance can be attached to the data. The analysis of data must consist of either graphical or statistical comparisons of levels of seismicity during various phases of either filling or injection. Unless the monitoring period starts two or more years before reservoir filling, misleading results can be obtained.

● 2.7-6 Shudde, R. H. and Barr, D. R., **An analysis of earthquake frequency data,** *NPS55Su76121,* Naval Postgraduate School, Monterey, California, Dec. 1976, 11. (NTIS Accession No. AD-A035 813)

Analyses of the times of occurrence of the major earthquakes recorded in periods of several years' duration are presented for two regions along the San Andreas fault in central California. Components of tidal force and jerk are computed for each earthquake in the sample. The distributions of these quantities over the sample periods are compared with corresponding distributions obtained under the hypothesis of random occurrence. It is concluded that the evidence provided by this analysis does not support a hypothesis of significant tidal effects upon earthquake occurrences in the populations considered.

● 2.7-7 Murphy, J. R., **Seismic source functions and magnitude determinations for underground nuclear detonations,** *Bulletin of the Seismological Society of America,* 67, *1,* Feb. 1977, 135-158.

A variety of near-regional, regional, and teleseismic ground-motion data have been used to evaluate proposed models of the nuclear seismic source function for underground detonations in tuff/rhyolite emplacement media. It

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has been found that both the near-regional broad-band seismic data and the teleseismic body-wave magnitude data are consistent with the modified source model proposed by Mueller and Murphy, but not with the simple cube root of the yield-scaling source model. In particular, the observed linearity and slopes of the body-wave magnitude-yield curves as well as the observed variation of *P*-wave period with yield have been found to be fully compatible with the modified source model. On the other hand, it has been concluded that the observed long-period surface-wave data are inconsistent with a simple, spherically symmetric source model. The results of a preliminary analysis have suggested that this discrepancy may be related to the spall closure phenomenon.

● **2.7-8** Shudde, R. H. and Barr, D. R., **An analysis of earthquake frequency data,** *Bulletin of the Seismological Society of America,* **67,** *5,* Oct. 1977, 1379–1386.

Analyses of tidal forces present during occurrence of the major earthquakes recorded in periods of several years' duration are presented for two regions along the San Andreas fault in central California. Components of tidal force and their time rates of change are computed for each earthquake in the sample. The distributions of these force and jerk quantities for actual earthquakes are compared with corresponding distributions obtained for pseudo-earthquakes generated under the hypothesis of random occurrence. It is concluded that the evidence provided by these analyses does not support the hypothesis that tidal-force levels or rates of change have significant effects on earthquake occurrences in the regions considered.

2.7-9 McGarr, A., **Seismic moments and volume changes,** *Journal of Geophysical Research,* **81,** *8,* Mar. 10, 1976, 1487–1494.

For many situations of practical interest, the seismicity that results from ground deformation associated with a change in volume, ΔV, is related to the volume change by $\Sigma M_0 = K\mu$ (absolute value of ΔV), where ΣM_0 is the sum of the seismic moments of the earthquake population, μ is the modulus of rigidity, and K is a factor close to 1. A necessary condition for this relationship is that the change in volume is accommodated only by seismic failure. The relationship between seismic moment and volume change was used to analyze earthquake sequences resulting from ground deformation that could most easily be described as volume changes. First, in the East Rand Proprietary Gold Mine, South Africa, total seismic moments of mine tremors were found to be in good agreement with total moments inferred from the volume of rock mined. Second, annual sums of seismic moments for the Denver earthquakes were compared with yearly total moments estimated from the volume of fluid injected at the Rocky Mountain arsenal well. There was good agreement between calculated and observed moments from 1962 to 1965 when annual rates of fluid injection were high. Nearly all the Denver earthquakes in these years correspond to the accommodation of injected fluid at depth. The association between earthquakes that occurred after Feb. 1966, when the fluid injection was stopped, and the Rocky Mountain arsenal well is not clear. Finally, the total seismic moment for earthquakes near Matsushiro, Japan, that occurred from Mar. to Sept. 1966 was found to be in good agreement with the moment calculated from the expansion of the seismic source region determined from measurements of horizontal and vertical ground movement. Energy changes were calculated for the volumetric changes considered here and compared with the total energy radiated in the seismic waves. The seismic energies ranged from 0.2% to 3% of the total energy changes.

● **2.7-10** Beck, J. L. and Housner, G. W., **Oroville reservoir, California and the earthquakes of August 1, 1975,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 215–220.

On Aug. 1, 1975, following some foreshock activity beginning June 28, 1975, an earthquake ($M_L = 5.7$) occurred 11 km from a large reservoir near Oroville, California. The effects of this earthquake at Oroville and at the Oroville dam, the highest in the U.S., are briefly discussed. The possibility that the seismicity may have been reservoir induced is discussed. It is shown that it is unlikely that the water load triggered the main shock, but no definite conclusions can be made with respect to triggering by the water pressure.

● **2.7-11** Abrahamson, G. R., Lindberg, H. E. and Bruce, J. R., **Simulation of strong earthquake motion with explosive line source arrays,** *SRI PYU 6004,* Stanford Research Inst., Menlo Park, California, Oct. 1977, 43.

The need for an in-situ test technique to guide the design of earthquake-resistant structures has long been recognized. Over the past year SRI (Stanford Research Inst.) International has been conducting an investigation on the feasibility of simulating earthquakes by contained explosions in line source arrays. The technique consists of detonating a plane array of vertical line sources placed in the vicinity of the structure to be tested. In a full-scale test, the array might measure 100 x 30 ft, consist of 10 to 20 vertical bore holes 30-ft deep, spaced on 5- to 10-ft centers, placed about 30 ft from the structure to be tested.

Reusable hardware was developed for producing contained explosions in a 1/3-scale source, instrumentation was incorporated for hardware diagnostics and output measurements, reasonable acceleration and frequencies were obtained in soil with the 1/3 scale source, and repeatable results were demonstrated. Estimates based on our current experiments show that in a 100- x 30-ft array, a

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5-Hz pulse with a 0.5-g peak acceleration can be produced with less than 100 lb of explosive. A complete train of oscillations typical of strong earthquake motion with a total duration of 10 s and peak accelerations reaching 1 g is estimated to require about 500 lb of explosive, fired in 10 detonations.

● 2.7-12 Grover, A., **A coupled response of explosive energy and its implications to seismic risk in various media,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 637-642.

The effect of a medium immediately surrounding an explosive source on primary compressional wave properties has been studied by means of a three-dimensional model. An approximate theory for radiation from a single harmonic line source of P-waves on the axis of the solid circular body embedded in an infinite homogeneous solid medium has been developed. The results obtained in time and frequency domains are greatly dependent on the characteristics of the medium around the explosion. The degree of seismic risk, cavity radius size, and medium surrounding the source are evaluated.

2.8 Earthquake Prediction

● 2.8-1 Ergunay, O., **Prediction of earthquakes** (Depremlerin onceden bilinmesi, in Turkish), *Deprem Arastirma Enstitusu bulteni,* 3, *12,* Jan. 1976, 36-55.

Every time an earthquake occurs in Turkey, the public and the press discuss whether earthquakes can be predicted or not. After the Lice earthquake of Sept. 6, 1975, when 2385 people were killed and 8165 houses were damaged, many articles appeared in the press and the prediction problem was discussed widely. This paper has been prepared as a result of these articles and discussions, which created a great deal of public interest. The concept of earthquake prediction is explained. The studies being conducted in this area in Turkey and in other countries, the practical results of such studies, and the clarification of many questions are presented. All discussions are nontechnical.

2.8-2 Fitterman, D. V., **Theoretical resistivity variations along stressed strike-slip faults,** *Journal of Geophysical Research,* **81,** *26,* Sept. 10, 1976, 4909-4915.

A simple model of a strike-slip fault is analyzed to see if observable resistivity variations would be produced by strain accumulation. Fault displacement is found to produce only pure shear loading. Estimates of the effect of this stress system on rock resistivity are made by using existing resistivity data measured during isotropic and triaxial loading. For a model of the San Andreas fault, 50 cm of fault displacement are found to produce apparent resistivity variations of about 1%. The model considered requires long periods of time (>20 years) to produce 1% resistivity variations. This is in contrast to reported observations of large resistivity changes (>20%) developing in a period of several months before an earthquake. If more observational data confirm preearthquake resistivity changes, very different models must be considered to explain the observations.

2.8-3 Wyss, M., **Local sea level changes before and after the Hyuganada, Japan, earthquakes of 1961 and 1968,** *Journal of Geophysical Research,* **81,** *29,* Oct. 10, 1976, 5315-5321.

The two Hyuganada, Japan, earthquakes of 1961 and 1968 (M = 7.0 and 7.5, respectively) each broke 80-km segments of the thrust zone abutting to the southern end of the Nankaido 1946 rupture. Mean annual sea levels (period 1942-1973) at eight tide gage stations along a 400-km segment of the Japanese coast centered in the epicentral area were compared to each other. It was found that mean annual sea level differences between the northernmost and southernmost stations (Kochi and Kagoshima) remained constant over a 16-year period, with a standard deviation from the 16-year mean of 1.2 cm. Compared with these northern and southern reference stations, four central tide gages located within 50 km of the two Hyuganada ruptures showed the following changes: (1) At the station between the two earthquakes (Hosojima), sea level dropped by about 5 cm in 1957, and it rose by about 5 cm after the earthquake in 1968. (2) At two stations north of the ruptures (Tosashimizu and Uwajima), sea level rose by about 4 cm in 1957. (3) At a station close to the southern end of the ruptures (Aburatsu), sea level remained constant with a standard deviation of about 1.1 cm between 1950 and 1966. These sea level changes are interpreted to reflect local crustal uplift at Hosojima and subsidence in the Tosashimizu-Uwajima area in 1957. If one assumes that these vertical movements were precursors to the two Hyuganada earthquakes, the data support the dilatancy-diffusion hypothesis: Near the center of the ruptures, dilatancy may have caused uplift (Hosojima), and beyond the northern end of the aftershock area, subsidence may have occurred due to reduction in crustal pore pressure. Precursor times of these magnitude 7.0 and 7.5 earthquakes may have been approximately 3.5 and 5 years, respectively.

2.8-4 Bischke, R. E., **Secular horizontal displacements: A method for predicting great thrust earthquakes and for assessing earthquake risk,** *Journal of Geophysical Research,* **81,** *14,* May 10, 1976, 2511-2516.

Island arcs may be subject to large strain variations during the first few decades following an earthquake. As the stress returns to preearthquake levels, the strain differences across an island arc may become much more uniform. This suggests a method for qualitatively assessing

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long-term earthquake risk and for earthquake prediction. A continuous extension of some island arcs during both the preseismic and the seismic stages of ground displacement may be responsible for the origin of the active interarc basins described by Karig.

● 2.8-5 Ward, P. L., ed., **Earthquake prediction studies in southern California: research in progress**, *Open-File Report 76-456*, National Center for Earthquake Research, U.S. Geological Survey, Menlo Park, California, June 1976, 152.

The report of an uplift of the ground of up to 0.25 m over at least 12,000 sq km of southern California has increased significantly the interest in earthquake prediction studies in this region. A substantial increase in research is planned, particularly by the U.S. Geological Survey and its many contractors and by the California Div. of Mines. The purpose of this report is to briefly summarize most of the work in progress to encourage communication among the wide variety of researchers and to assist all present or prospective researchers in this region to understand what work is under way and not yet published. Information in this report was collected primarily in response to a brief questionnaire. Funding for the research listed comes primarily from the U.S. Geological Survey, the National Science Foundation, and the State of California.

● 2.8-6 Weisbecker, L. W. *et al.*, **Earthquake prediction in society**, Center for Resource and Environmental Systems Studies, Stanford Research Inst., Menlo Park, California, Feb. 1977, 40.

This report, which is designated as a "User Summary," is based upon an in-depth comprehensive technological assessment of earthquake prediction. The full report of the assessment is based upon some 22 unpublished working papers covering various aspects of the study.

The study was accomplished by a 13-person team from the Stanford Research Inst. and a number of subcontractors and consultants. In addition, persons knowledgeable in different subject areas provided valuable insights to the study team. An oversight committee for the assessment provided valuable comments on the main report. A users' panel assisted in structuring this report.

● 2.8-7 USGS/NSF Staff Planning Group and Advisory Group on Earthquake Prediction and Hazard Mitigation, **Earthquake prediction and hazard mitigation: options for USGS and NSF programs**, U.S. Geological Survey/National Science Foundation Staff Planning Group, Reston, Virginia, 1976, 110. (NTIS Accession No. PB 258 451)

This document constitutes a plan based upon staff working papers, two Advisory Group meetings (June and Aug. 1976), input from Advisory Group members and subpanels, and comments, suggestions, and criticism received from others. Contents include four chapters: (I) Introduction, including list of Advisory Group members; (II) Brief assessment of available social, political, and economic measures for mitigating impacts of earthquakes and the current state of the technological basis for these measures; (III) Discussion of current earthquake research efforts and options for future augmentation of earthquake research, including activities, funding levels, technical milestones, and public benefits; (IV) Discussion of the efforts and options for improving utilization of research results and coordination mechanisms. The document builds upon numerous studies of the earthquake problem and analyses of strategies for response to it that have already been made. Some of the most significant of these are listed in Appendix I.

● 2.8-8 Kato, T. and Kasahara, K., **The time-space domain presentation of levelling data**, *Journal of Physics of the Earth*, **25**, *3*, 1977, 303–320.

This paper introduces a computer program which compiles a series of repeated leveling data along a route and displays a comprehensive pattern of vertical land movement in the time-space domain. This technique presents an intuitive view of the dynamic character of the crust. A difficulty for this purpose is faulty data. Surveying intervals are relatively long and not uniform. In addition, each set is sometimes contaminated by partial missing of data. Therefore, a proper interpolation of these faulty portions must be made in order to derive a uniform array of data in the time-space domain. After interpolation, legible symbols are assigned to data at each grid point in the domain and a two-dimensional chart is drawn by use of a computer. A cubic spline function is used for interpolation, assuming that the faulty part of data is represented by a smooth curve in both the time and space axes. The program is applied to several examples in order to demonstrate the usefulness of the present technique. Several aspects of strain accumulation are noted on a chart. A brief discussion from the tectonic viewpoint is included.

● 2.8-9 Cramer, C. H., Bufe, C. G. and Morrison, P. W., ***P*-wave travel-time variations before the August 1, 1975 Oroville, California earthquake**, *Bulletin of the Seismological Society of America*, **67**, *1*, Feb. 1977, 9–26.

On Aug. 1, 1975, a magnitude 5.9 (m_b), (BRK, M = 5.7), normal dip-slip earthquake occurred 10 km south of Oroville, California. *P* arrivals for teleseismic and regional sources at the few seismographs in the area have been carefully timed to an accuracy of ±0.02 sec and the relative residual technique has been applied to these data. The data cover the period from Aug. 1968 through Mar. 1976. A significant delay of about 0.1 sec in travel-time residuals for Russian nuclear blasts was observed over a 3-year period preceding the Oroville earthquake at station ORV 10 km

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north of the epicenter. A 0.1-sec delay in travel-time residuals for U.S. nuclear blasts occurred after the Oroville event at station MGL, 40 km north of the main shock's epicenter. *P* arrivals from deep Tonga-Figi earthquakes have also been analyzed but reveal no systematic time variations beyond ±0.05 sec from their mean values. *P* arrivals from moderate-sized earthquakes along the San Andreas fault system in central California proved to be an unsatisfactory source of data because of ambiguities created by multiple *P*-phase arrivals and the emergent nature of the arrivals. The sparse station coverage does not allow adequate delineation of the extent and character of the anomalous *P* velocity zones, but the data do provide some limitations. The postearthquake travel-time delay at MGL may be precursory to a future earthquake or may only be related to the redistribution of stress in the Oroville area.

● **2.8-10** Peake, L. G., Healy, J. H. and Roller, J. C., **Time variance of seismic velocity from multiple explosive sources southeast of Hollister, California,** *Bulletin of the Seismological Society of America,* **67**, *5*, Oct. 1977, 1339–1354.

As part of an intensive effort to monitor the section of the San Andreas fault south of Hollister, California, for any seismic velocity time variance associated with earthquakes, 17 explosive charges were fired between Aug. 1974 and May 1975 at a point 35 km east of the fault zone and recorded by a special network of 51 seismometers within and west of the San Andreas fault zone. The shot point was selected to minimize differences in rock properties for successive shot locations, and all the data were recorded on a single tape recorder. By cross correlating the records from successive shots, it was possible to determine differential travel times to an accuracy of 4 msec or better using a 250-kg explosive charge recorded in the distance range between 30 to 50 km. No definitive changes in seismic velocity were detected during the time covered by this experiment. The spatial distribution of stations and the temporal distribution of shots were designed to reveal anomalies that might be associated with a magnitude 4 or greater earthquake. Nine small earthquakes (M = 2.9 to 3.3) did occur during the experiment, but anomalies associated with these earthquakes could have escaped detection. This work demonstrates the feasibility of monitoring large regions with a high density of instruments to detect systematic changes in arrival times from artificial sources with an accuracy approaching 1 msec and a cost commensurate with the task of earthquake prediction.

● **2.8-11** Wyss, M., **The appearance rate of premonitory uplift,** *Bulletin of the Seismological Society of America,* **67**, *4*, Aug. 1977, 1091–1098.

Sea-level data are used to show that 5 to 8 cm of aseismic uplift accumulated in a few months before the Long Beach, Hyuganada, and Peru earthquakes. This uplift rate is an order of magnitude faster than tectonic loading rates. Alternative mechanisms to explain this observation are anelastic creep at depth or dilatancy in a jointed crust. The energy expended during uplift is larger than the seismically radiated energy by approximately a factor of ten.

● **2.8-12** Steppe, J. A., Bakun, W. H. and Bufe, C. G., **Temporal stability of *P*-velocity anisotropy before earthquakes in central California,** *Bulletin of the Seismological Society of America,* **67**, *4*, Aug. 1977, 1075–1090.

An examination of *P*-wave travel-time residuals from small earthquakes (source events) located near three larger earthquakes ($4 \leq M \leq 5$) that occurred on the San Andreas fault, near Bear Valley in central California, shows no temporal variations in the residuals extending over broad azimuth ranges ($\Delta\phi > \sim 40°$). Such variations could have resulted from changes in horizontal velocity anisotropy precursory to the larger events. The examination also shows (1) numerous azimuthal variations in the residuals within narrow azimuth bands ($\Delta\phi < \sim 30°$), apparently due to spatial heterogeneity of crustal velocity, (2) a dependence of residual on magnitude for a few stations but not for most stations, and (3) trends of residual versus source event focal depth for about one-third of the 75 source region-station pairs examined. Residuals in these cases typically change by 0.1 sec, and occasionally by 0.2 to 0.3 sec, over the 2- to 12-km focal depth range sampled. The trends vary from station to station in a complex manner. The assumption that travel-time changes are reflected in the residuals is tested by modifying arrival times from some source events according to assumed forms for the travel-time change, relocating those events, and comparing the resulting residuals with those from the unmodified data.

● **2.8-13** Mortensen, C. E., Lee, R. C. and Burford, R. O., **Observations of creep-related tilt, strain, and water-level changes on the central San Andreas fault,** *Bulletin of the Seismological Society of America,* **67**, 3, June 1977, 641–649.

Several simultaneous observations of surface fault creep, tilt, strain, and water-level fluctuations have been recorded along the San Andreas fault in the vicinity of the Almaden-Cienega Winery south of Hollister, California. Creep events recorded on the winery creepmeters on Feb. 16, 1975, and by the winery and Harris Ranch creepmeters on Sept. 17, 1975, were modeled as migrating dislocations with geometries chosen to give results that match the observed tilt and strain data. Source depths for the Feb. 16th and Sept. 17th creep events were found to be relatively shallow, the depth to the lower boundary of the slip surface being 0.4 and 2.0 km, respectively. In both cases, slip was found to propagate from the northwest toward the southeast, which is consistent with changes in water level

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observed in a well near the winery. Since the installation of the tiltmeter and strainmeter 0.8 km northwest of the Cienega Winery, six tilt and strain signals with durations typical of creep events have been related to observed surface creep, while 11 such signals appear unrelated to recorded surface creep. The latter may result from surface creep of limited extent or creep at depth.

● 2.8-14 Castle, R. O. *et al.*, **Preseismic and coseismic elevation changes in the epicentral region of the Point Mugu earthquake of February 21, 1973,** *Bulletin of the Seismological Society of America,* **67,** *1,* Feb. 1977, 219-231.

Comparisons among five repeated level surveys between Santa Monica and Ventura during the period 1960-1970 indicate that the epicentral region of the 1973 Point Mugu earthquake (M_L = 6.0) sustained measurable vertical crustal movements that preceded and may have accompanied or followed the main shock. These crustal movements, which have been referred chiefly to bench mark Tidal 3, Santa Monica, as invariant in elevation, were largely episodic and commonly oscillatory. Interpretation of the data is complicated by the relatively intense compaction-induced subsidence developed within the Oxnard Plain and by the fact that the apparent movements are only marginally above possible survey error.

Elevation changes disclosed through comparisons of 1960 with 1968 surveys show that the upper (northern) plate of the west-trending Santa Monica fault rose by 30 to 40 mm at least as far west as Point Mugu; whereas still farther to the west, any such positive movements were apparently overwhelmed by differential subsidence within the Oxnard Plain. The 1960-1968 uplift is consistent with continuing thrusting along the north-dipping southern frontal fault system of the Transverse Ranges. Between 1968 and 1971, the 1960-1968 vertical movement pattern was crudely reversed in both form and magnitude; down-to-the-west tilting between a point about 20 km west of Santa Monica and Point Mugu was accompanied by the cessation of differential subsidence and the development of up to 35 to 40 mm of localized uplift within the Oxnard Plain. The 1968-1971 movements suggest left-lateral reverse creep at depth, the possible onset of dilatancy, or some combination of these. Between 1971 (pre-earthquake) and 1973 (post-earthquake), the upper plate of the Santa Monica fault underwent uplift that nearly restored the 1971 surface to its 1968 configuration within the area east of Point Mugu characterized by the 1968-1971 anomalous downwarping. Because the maximum measured rebound of more than 30 mm during the period 1971-1973 was approximately an order of magnitude greater than the coseismic uplift calculated from dislocation modeling, this uplift may have been principally preseismic and perhaps indicative of an evolving dilatant volume.

2.8-15 Kean, W. F. *et al.*, **The effect of uniaxial compression on the initial susceptibility of rocks as a function of grain size and composition of their constituent titanomagnetites,** *Journal of Geophysical Research,* **81,** *5,* Feb. 10, 1976, 861-872.

An ac bridge method has been used to study the effect of uniaxial compression on the initial magnetic susceptibility of rocks and separated titanomagnetites. The samples used were basalts containing titanomagnetite with varying grain size and morphology and dispersions of magnetite and titanomagnetite powders in epoxy. The results indicate that (1) the effect of pressure on susceptibility decreases with decreasing grain size, not in a continuous manner but rather depending upon whether the dominant magnetic grain size is multidomain, pseudo single-domain or single-domain, (2) the pressure response increases with the composition parameter x of the titanomagnetite in the solid solution series xFe_2TiO_4 $(1-x)Fe_2O_4$, and (3) the morphology of the grains influences the pressure response. The results are interpreted in terms of the behavior of the multidomain, pseudo single-domain and single-domain material. Changes in thermoremanent magnetization and the acquisition of pressure remanent magnetization under uniaxial compression, which were observed in conjunction with the susceptibility studies, suggest that rocks containing coarse grain titanomagnetites as the dominant magnetic mineral phase are the most efficient stress transducers. Hence, a definitive seismomagnetic experiment would be easiest in a seismically active region in which such rocks are prominent.

● 2.8-16 Bolt, B. A., **Constancy of *P* travel times from Nevada explosions to Oroville dam station 1970-1976,** *Bulletin of the Seismological Society of America,* **67,** *1,* Feb. 1977, 27-32.

The suggestion is tested that variations in *P* travel-times to a station near a large dam might signal an impending local earthquake, perhaps induced by the reservoir. *P* travel-times to the Oroville station (relative to Jamestown) from precisely located underground nuclear explosions at the Nevada Test Site from Dec. 1970 to Mar. 1976 provide a suitable stochastic time series of uniform precision. Fluctuations before, during, and after the 1975 Oroville earthquake sequence (main shock M_L = 5.7) are analyzed for significance. Although the ray paths pass within 5 km of the focal region, no fluctuations in semi-annual mean residuals occurred greater than 0.01 ± 0.06 sec. At a high probability level, there is no evidence that the seismic source process at Oroville altered the constancy of *P* velocities in crustal rocks between the focal zone and the dam.

● 2.8-17 Stiller, H., Wagner, F. C. and Vollstadt, H., **A two-phased model for the description of the influence of**

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cracks on the P- and S-wave velocities in dry and saturated rock samples, *Tectonophysics,* **43,** *3-4,* Dec. 10, 1977, 181–197.

The premonitory variations of seismic-wave velocities before earthquakes originate from various cracking processes before the fracture. It can be shown that these variations are comparable with wave-velocity variations in porous model samples with defined pore sizes. Furthermore, it is possible to describe analytically the variation of wave velocities as a function of the parameter K_0, which describes the fracturing process, and a material/depth parameter *AP*. On the basis of the wave velocity versus pressure curves of rocks, it is possible to determine K_0 and *A*. Using the material/depth parameter, *AP*, sediments in covering strata and eruptive rocks in regions of earthquakes of shallow-to-medium depth can be described. A relationship between Δv_p and ΔK_0 can be established. For acid to ultrabasic rocks, a variation of up to 2 km/s indicates a variation of K_0 of 0.1–1.0.

Moreover, it is possible to establish a relationship between K_0 and the number as well as the mean length of cracks in the rock. The solutions differ depending on the cracks being closed or open. For closed cracks, a wave-velocity minimum of 6% results. For open cracks, the variations of the number and mean length of cracks are taken into account by means of a stochastic process; the resulting variations of K_0 and the wave velocity can adequately explain the variations in seismic-wave velocity. Variations of the pore pressure have an influence on K_0 and the wave velocity only under the most favorable geological conditions; generally they are insignificant. Also, for S-wave velocities and for the ratio v_p/v_s, the wave velocity versus pressure equations are valid; it is possible to state K_0- and *A*-values.

● **2.8-18** Wesson, R. L. *et al.,* **Search for seismic forerunners to earthquakes in central California,** *Tectonophysics,* **42,** *2-4,* Oct. 20, 1977, 111–126.

The relatively high seismicity of the San Andreas fault zone in central California provides an excellent opportunity to search for seismic forerunners to moderate earthquakes. Analysis of seismic traveltime and earthquake location data has resulted in the identification of two possible seismic forerunners. The first is a period of apparently late (0.3 sec) P-wave arrival times lasting several weeks preceding one earthquake of magnitude 5.0. The rays for these travel paths passed through—or very close to—the aftershock volume of the subsequent earthquake. The sources for these P-arrival time data were earthquakes in the distance range 20–70 km. Uncertainties in the influence of small changes in the hypocenters of the source earthquakes and in the identification of small P-arrivals raise the possibility that the apparently delayed arrivals are not the result of a decrease in P-velocity. The second possible precursor is an apparent increase in the average depth of earthquakes preceding two moderate earthquakes. This change might only be apparent, caused by a location bias introduced by a decrease in P-wave velocity, but numerical modeling for realistic possible changes in velocity suggests that the observed effect is more likely a true migration of earthquakes. To carry out this work—involving the manipulation of several thousand earthquake hypocenters and several hundred thousand readings of arrival times—a system of data storage has been designed, and manipulation programs for a large digital computer have been executed. This system allows, for example, the automatic selection of earthquakes from a specific region, the extraction of all the observed arrival times for these events, and their relocation under a chosen set of assumptions.

● **2.8-19** Fedotov, S. A. *et al.,* **Long- and short-term earthquake prediction in Kamchatka,** *Tectonophysics,* **37,** *4,* Feb. 2, 1977, 305–321.

This paper presents the results of long- and short-term earthquake prediction obtained during 1971–1974. They are summarized as follows. The map of long-term prediction for the Kurile-Kamchatka zone compiled in 1965 and supplemented in 1972 by S. A. Fedotov is in good agreement (in four of four possible cases) with recorded seismicity. The results obtained suggest that the areas for which the log (E_p/E_s) of small earthquakes is low may be the areas of future large earthquakes. Prediction of active periods for the Kamchatka earthquakes with $M \geq 7$ has been made on the basis of studying the correlation of seismicity with the lunar tide with an 18.6-year period. A possibility has been found for using the phenomenon of "induced foreshocks" for earthquake prediction, i.e., when a large remote earthquake induces small preceding events in the zone of preparation of a large earthquake.

The following three methods were used for operative short-term prediction of the time and place of future earthquakes with $M \geq 5.5$: (1) Use of specific electrotelluric field anomalies, from 5 to 20 days in duration, which are recorded by a specially designed network of stations. (2) Method of V_p/V_s anomalies. The anomalously high and low V_p/V_s values for a seismic station point to the possibility of large earthquakes near the latter. (3) The earthquake statistics method described by Fedotov et al. in 1972. Short-term seismic prediction is being made twice a week in two versions: Forecast I (for the whole of Kamchatka) and Forecast II (for each of six overlapping segments of the Kamchatka seismic zone). This paper discusses the results of successful testing of short-term earthquake prediction during two years. During the "alarm" periods, the probability of large earthquakes is double the average.

● **2.8-20** Gay, Jr., T. E. *et al.,* **Earthquake prediction evaluation guidelines,** *California Geology,* **30,** *7,* July 1977, 158–160.

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California's Office of Emergency Services has established the California Earthquake Prediction Evaluation Council, composed of nine earth scientists qualified to judge the technical merit of earthquake predictions. The council's purpose is to assess the reliability of the data and the scientific validity of the technique used to arrive at a specific prediction. Guidelines for evaluating predictions are listed.

2.8-21 Avchyan, G. M., Belyaevskiy, N. A. and Polshkov, M. K., **A single mechanism for the generation of shallow earthquakes and their precursors,** *Physics of the Solid Earth,* **12,** *12,* 1977, 763–768.

One of the possible hypothetical mechanisms of the generation of shallow earthquakes, the rupture of rocks with a rapid decrease in interstitial pressure, is examined. The nature of the phenomena which are observed prior to earthquakes and are regarded at present as their precursors is explained on the basis of this mechanism. The results of laboratory modeling of this mechanism are given.

● **2.8-22** Saxena, A. K., Gaur, V. K. and Khattri, K. N., **P-residual studies around Koyna Dam region, Maharashtra, India,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 648–654.

An anomalous increase of about 0.4 sec occurred in the P-wave residuals around the Koyna Dam seismograph station prior to the earthquake of Dec. 10, 1967, which occurred in the vicinity of the dam. P residuals were calculated from about 800 shocks, most of which originated in the Hindukush and Burma regions. These were averaged over intervals of six months from Jan. 1964 to June 1969. However, owing to large inaccuracies in the arrival times of events prior to Jan. 1966, definitive changes in the P-wave velocity can be considered to have begun a year before the event of Dec. 10, 1967. Residuals of the Satara station, about 42 km NE of Koyna, were also studied, but they did not yield useful information owing to a large scatter in the residual data.

● 2.8-23 Wood, M. D. and King, N. E., **Relation between earthquakes, weather, and soil tilt,** *Science,* **197,** *4299,* July 8, 1977, 154–156.

Two years of local earthquake, temperature, and rainfall data taken near a tiltmeter site were used in a study of the numerical relation between these phenomena and the recorded tilt response. A least-squares shaping and predictive error filter approach was used. The relations were ranked in part according to the root mean square (r.m.s.) error of fit across the entire sample space. The tilt data with an annual range of tilt of approximately 10 microradians were fitted to the combined weather data of temperature and rainfall with a 0.75 microradian r.m.s. error. The best fit of earthquakes to these same tilt data is the subclass of events with magnitude (M) > 2.5 within 30 km of the tilt site. The filter that mapped earthquakes to tilt yielded a 1.03 microradian r.m.s. error. The most unusual tilt anomaly over the entire two-year period has the best fit of rainfall to the data for any single month of the entire data set. This unusual anomaly was the basis of an erroneously predicted earthquake ($M \sim 5$). These data indicate that if there are premonitory earthquake signals they are buried in local meteorological noise. Separating an earthquake anomaly from the response to surface phenomena becomes more difficult as the earthquake anomaly lead time approaches the rise time of the soil to weather and seasonal variations.

2.8-24 Evison, F. F., **Precursory seismic sequences in New Zealand,** *New Zealand Journal of Geology and Geophysics,* **20,** *1,* 1977, 129–141.

The sequence of precursory swarm and gap has been confirmed in the local seismic activity preceding major earthquakes at Gisborne (1966), Seddon (1966), Inangahua (1968), and Hastings (1973). The magnitude of the main shock appears to be related to that of the largest swarm earthquake as well as to the precursor time. This suggests a method of prediction which might yield long-term estimates of location, magnitude, and time of occurrence.

2.8-25 Noguchi, M. and Wakita, H., **A method for continuous measurement of radon in groundwater for earthquake prediction,** *Journal of Geophysical Research,* *82,* *8,* Mar. 10, 1977, 1353–1357.

A practical method of continuous measurement of radon in groundwater for earthquake prediction, especially suitable for field observation, is described. The detector consists of a ZnS scintillation chamber for alpha counting and a separation chamber in which radon is emanated from groundwater to the gaseous phase. The radon concentration in the gaseous phase equilibrates with that in water according to the distribution coefficient. Alpha activities of radon and its daughters in the scintillation chamber are counted and recorded continuously. The counting rate is obtained by the detector, which has an effective volume of 1.6 l.

2.8-26 Hodych, J. P., **Single-domain theory for the reversible effect of small uniaxial stress upon the remanent magnetization of rock,** *Canadian Journal of Earth Sciences,* **14,** *9,* Sept. 1977, 2047–2061.

This paper on small uniaxial stress changing the remanent magnetization of rock is a companion to a previous paper by the same author on stress changing susceptibility, both phenomena being of current interest in attempts at earthquake forecasting. Theoretical expressions are derived (using rigorous energy minimization but ignoring thermal activation) for reversible change in remanence parallel to the stress axis for samples containing single-domain grains

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of a ferromagnet with cubic magnetocrystalline anisotropy (K_1 positive or negative) and anisotropic magnetostriction. The grains are assumed to be noninteracting and randomly oriented spheres or ellipsoids of revolution. Also, approximate expressions are given for samples containing multidomain grains with very strongly pinned walls. Thermal (or chemical), anhysteretic, and saturation remanence are discussed. For remanence change perpendicular to the stress axis, one expects -1/2 the above expressions for change parallel to the stress axis, which is easily proven for thermal remanence.

2.8-27 Utsu, T., **Probabilities in earthquake prediction** (in Japanese), *Zisin, Journal of the Seismological Society of Japan*, **30**, 2, Aug. 1977, 179–185.

Two kinds of probabilities—p_1 and p_2—are considered in connection with earthquake prediction. p_1 is the probability that a prediction will be successful and p_2 is the probability that an earthquake will be predicted. Both "prediction" and "earthquake" have been defined by certain criteria. The status of the prediction based on each observational element can be indicated by a point on the p_1-p_2 plane. The effectiveness of the prediction is related to p_1 and p_2 by an equation.

2.8-28 Iizuka, S., **A search for temporal variations in V_p/V_s around Suruga Bay** (in Japanese), *Zisin, Journal of the Seismological Society of Japan*, **30**, *3*, Dec. 1977, 307–316.

The suggestion was tested that anomalously low velocity of compressional waves around Suruga Bay at the northeast end of Nankai trough might be a precursor of an impending great Tokai eathquake. Temporal variations in the ratio of compressional to shear velocities in the region were analyzed during the period from Jan. 1951 to June 1976 by using the events which have frequently occurred in the eastern part of the Kanto district. Long-term premonitory variations in V_p/V_s ratios relating to a forthcoming large earthquake with a magnitude of about 8.0 could not be detected. On the other hand, a significant decrease of about 15% in V_p/V_s was observed over a 5-year period preceding the 1965 Shizuoka earthquake (M 6.1, H = 20 km.). This anomalous time seems to be unusually longer for its magnitude. The circumstances might be explained by a new model which takes the thermal process into consideration for earthquake forerunners.

2.8-29 Sakata, S., **Crustal strain changes due to the changes of elastic constants** (in Japanese), *Zisin, Journal of the Seismological Society of Japan*, 30, 2, Aug. 1977, 143–149.

The change of Vp/Vs value before an earthquake corresponds to the change of the elastic constants of a part of the crust. The change of elastic constant distribution in the crust under an initial stress field will induce strain changes. If those strain changes can be detected by ordinal geodetic surveys, they will serve as good earthquake precursors as well as Vp/Vs changes. In order to estimate the values of expected strain changes, a simple plane strain model was adopted for calculations. The results are as follows: (1) It is very difficult to detect any substantial horizontal strain changes using an ordinal one-wave optical distance meter. (2) It is possible to detect the tilting of the free surface as vertical movement by ordinal leveling.

2.8-30 Garg, S. K., Brownell, Jr., D. H. and Pritchett, J. W., **Dilatancy-induced fluid migration and the velocity anomaly**, *Journal of Geophysical Research*, **82**, *5*, Feb. 10, 1977, 855–864.

This article describes a numerical model capable of simulating the fluid-rock processes that appear to precede some earthquakes. More precisely, the two-dimensional finite element–finite difference model can be used for investigating dilatancy and the associated migration of the pore fluids in a water-saturated rock. The numerical model is applied to a tectonically simple region containing a thrust fault. Fault creep is used to produce local stress concentrations (at fault ends) and dilatancy. Numerical results illustrate the somewhat conflicting requirements on rock permeability in the dilatancy-fluid diffusion model. During the first stage of the process, the permeability must be low enough to allow a drop in fluid pressure sufficient to cause vaporization; at a later stage, the permeability needs to be large enough to enable the fluid to move sufficiently rapidly into the dilated region. The numerical solution also yields such quantities as surface displacements, which can be used for constraining the choice of model parameters for a particular earthquake region.

2.9 Special Topics

2.9-1 **Seismic transparency of focal zones** (Seismicheskoe prosvechivanie ochagovykh zon, in Russian), Inst. *Fiziki Zemli AN S.S.S.R., Moscow, 1976, 194.*

The fundamental techniques employed in investigations of the transparency of focal zones are described. The experimental methods and instrumentation used in connection with standardized underwater explosions near the shores of the Kamchatka peninsula are examined. A change in wave propagation velocity correlated with the change in regional seismicity during the period of these investigations (1966-1972) is noted. A survey of worldwide data is given in support of the conclusions regarding the relationship between changes in wave characteristics and the formation of earthquakes.

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● **2.9-2** Adams, R. D., comp., **Seismology and related research in New Zealand, 1971-1974,** *Seismological Observatory Bulletin S-218, Information Series No. 110,* Seismological Observatory, New Zealand Dept. of Scientific and Industrial Research, Wellington, Mar. 1975, 32.

The contents of this report are as follows: (A) Seismograph stations, (B) Macroseismic studies, (C) Seismological research, (D) Physics of the Earth's interior, and (E) Bibliography. The report was presented at the Aug. 1975 meeting of the International Union of Geodesy and Geophysics, Grenoble.

2.9-3 Steeples, D. W. and Pitt, A. M., **Microearthquakes in and near Long Valley, California,** *Journal of Geophysical Research,* **81,** *5,* Feb. 10, 1976, 841–847.

Sixteen portable seismograph stations were deployed in the vicinity of the Long Valley geothermal area, California, from Apr. 27 to June 2, 1973. Only minor microearthquake activity was detected in the Long Valley caldera, but a high level of activity was detected to the south and east of the caldera. The abrupt spatial seismicity decrease at the southern boundary of the caldera suggests that the caldera is either structurally less competent than the surrounding crust or is at a junction of different regional tectonic deformation trends. No significant attenuation or delays occurred for either local *P* or S waves that traversed the caldera.

2.9-4 Johnson, T. L. and Scholz, C. H., **Dynamic properties of stick-slip friction of rock,** *Journal of Geophysical Research,* **81,** *5,* Feb. 10, 1976, 881–888.

Rupture and particle velocities were measured for stick-slip events occurring during frictional sliding on surfaces composed of Westerly granite and Twin Sisters dunite and on a combination of the two rocks. The rupture velocities observed for stick-slip events on sliding surfaces of Westerly granite typically had values of 2–3 km/sec. A few rupture speeds greater than the shear wave velocity v_s were measured, but these values could be artificially high, since the times of rupture arrival may have been measured in a direction oblique to the direction of rupture propagation. Rupture velocities observed from stick-slip events on surfaces of Twin Sisters dunite also had values of 2-km/sec, although lower velocities were occasionally measured. Rupture speeds on frictional surfaces composed of granite sliding on dunite ranged from about 1 km/sec to the S wave speed of the dunite (4.6 km/sec). Particle velocities were measured for stick-slip events on a similar suite of sliding surfaces. Compared to Westerly granite, stick-slip events on Twin Sisters dunite sliding surfaces are characterized by smaller stress drops $\Delta\tau$ and lower particle velocities. Stick-slip motion takes place in a nearly constant time when mass and stiffness of the loading system are fixed. This explains the observed linear relations between $\Delta\tau$ and particle velocity. A simple harmonic oscillator is a good model for the friction machine, predicting rise times and displacement time functions, which are in good agreement with observations.

2.9-5 Hadley, K., **The effect of cyclic stress on dilatancy: Another look,** *Journal of Geophysical Research,* **81,** *14,* May 10, 1976, 2471–2474.

Twelve granite and gabbro cylinders have been stress-cycled under confining pressures of 0.5, 1.5, and 5.0 kbar. No consistent trend appeared in the minimum stress required for dilatancy as a function of cycle number at the highest pressure, but at 1.5 kbar and below a decrease was indicated. Dilatant volumetric strain at differential stresses corresponding to the frictional strength of the rock persists at a value of a few parts in 10^4 at all confining pressures and is essentially unaffected by cycling.

2.9-6 Rice, J. R. and Simons, D. A., **The stabilization of spreading shear faults by coupled deformation-diffusion effects in fluid-infiltrated porous materials,** *Journal of Geophysical Research,* **81,** *29,* Oct. 10, 1976, 5322–5334.

A mathematical solution is developed for the steady, quasi-static, plane strain advance of a shear fault in a fluid-infiltrated elastic porous material. As revealed through analysis of some elementary fracture mechanics models, the coupled deformation-diffusion effects in such a material lead to a required "force" to drive the fault that increases continuously with fault velocity up to a maximum value. The nominal fault tip energy release rate required for spreading at this maximum is greater than that for very slow speeds by a factor approaching $(1-v)^2/(1-v_u)^2$, where v and v_u are the elastic Poisson's ratios under drained and undrained conditions, respectively. The effect is numerically significant and provides a mechanism by which a spreading shear fault can, within limits, be stabilized against catastrophic (seismic) propagation. Predictions of the model are compared to data representative of creep events on the San Andreas system. It is concluded that the speeds and slipping lengths of the observed events are consistent with their being stabilized by the effect discussed, and hence the model would seem to provide a viable mechanism for fault creep. Similar effects may be operative also in setting the time scale of progressive landslide failures in overconsolidated clay soils, in which rupture occurs by propagation of a narrow slip surface.

● **2.9-7** **Micro-earthquake monitoring of the Hanford region. Annual progress report,** Geophysics Program, Univ. of Washington, Seattle, Mar. 30, 1976, 60.

In July 1975 the Univ. of Washington assumed responsibility for the seismic network in eastern Washington. This network consists of 29 vertical seismograph stations plus three horizontal components. The entire network is broken

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down into two subnets. The Hanford net consists of 16 stations, located on or near the Hanford reservation and south of Smyrna, and enables the procurement of epicenter locations of all earthquakes of magnitude 1.0 and greater within the Hanford reservation. North of this network is the central net, which consists of 13 stations, two of which (Vantage and Warden) are closely spaced at the north edge of the Hanford net. The eleven remaining stations are spaced farther apart, to the north, in order to obtain a regional seismic picture of the entire eastern Washington area.

● **2.9-8** Udias, A., **Time and magnitude relations for three microaftershock series near Hollister, California,** *Bulletin of the Seismological Society of America,* **67**, *1*, Feb. 1977, 173–185.

Three series of aftershocks occurred in the region near Hollister, less than 15 km from the San Andreas Geophysical Observatory (SAO) in 1968, 1970, and 1971 following shocks of magnitudes 2.3, 3.2, and 4.0, respectively. The high-gain system operating at SAO permitted a full study of these series for events as small as magnitude -0.7. The distribution of time intervals between consecutive shocks and magnitudes shows certain definite differences in the time, space, and magnitude structure of the three sequences. The 1968 and 1970 series are highly concentrated in time and originate from small source volumes. Both correspond to the mainshock-aftershock type with high values of b (1.1 and 0.91). The 1971 series, located in the Bear Valley area about 15 km south of the other two, belongs to the swarm type with a low value of b (0.41). The situation suggests different physical conditions in two neighboring areas of the same fault with less homogeneous material and higher stresses acting at the area of the 1971 series. Variations in the b value with time for an area of 2500 km^2 around SAO can be interpreted as due to migration of seismic sources from one area to another, or definite changes in rock properties with time.

● **2.9-9** Weaver, J., **Three-dimensional crack analysis,** *International Journal of Solids and Structures,* **13**, *4*, 1977, 321–330.

A method is presented for the stress analysis of plane cracks of any shape in a stressed three-dimensional linear elastic space. The approach utilizes a system of integral equations which is defined over the crack area only. When these equations are solved for the unknown dislocations, all other quantities related to the crack and the space can then be found. The text contains sections concerning equation system derivation, numerical procedures, stress intensity factors, rectangular cracks, and earthquake control.

● **2.9-10** Rundle, J. B. and Jackson, D. D., **Numerical simulation of earthquake sequences,** *Bulletin of the Seismological Society of America,* **67**, *5*, Oct. 1977, 1363–1377.

Numerical simulation of earthquake occurrence using a one-dimensional fault model demonstrates that (1) the linear behavior of the magnitude-frequency relation is not an immutable law but rather is dependent on the mechanical properties of the fault, (2) "randomness" as measured by adherence to Poissonian statistics does not preclude useful prediction by statistical means, (3) the rate of occurrence of simulated earthquakes is in good agreement with the Kolmogorov model in which seismicity is related primarily to the stored elastic energy in a fault system, and (4) the occurrence of foreshocks and aftershocks can be well explained by the occurrence of stress-induced crack nucleation.

● **2.9-11** Reichle, M. and Reid, I., **Detailed study of earthquake swarms from the Gulf of California,** *Bulletin of the Seismological Society of America,* **67**, *1*, Feb. 1977, 159–171.

Three earthquake swarms originating in the Gulf of California spreading centers have been studied with sonobuoys. Two of the sequences were microearthquake swarms detected during a seismicity survey of the Gulf. Both were located beneath spreading center grabens. All events are quite shallow, less than 3 km into the crust. One of the swarms, which consisted of 1000 events in 8 hr, showed an episodic seismicity, which is associated with episodic magma movement and/or slip along the graben boundary fault. The third swarm occurred near the Delfin Basin trough. Sonobuoy epicenters show a seismicity trend parallel to local spreading center trends but about 17 km from the trough itself. Surface-wave radiation patterns of the larger events indicate primarily normal faulting. Normal faulting is not predominant in this region, however. Analysis of surface waves recorded at Pasadena indicates that most swarms have a strike-slip mechanism. The data from the Gulf suggest that sediments may play a major role in the character of earthquake swarms. In fact, some swarm events studied may occur in consolidated sediments.

● **2.9-12** Lockner, D. and Byerlee, J., **Acoustic emission and creep in rock at high confining pressure and differential stress,** *Bulletin of the Seismological Society of America,* **67**, *2*, Apr. 1977, 247–258.

Two samples each of Weber sandstone and Westerly granite were tested under triaxial compression at 1 kb confining pressure. The axial load was increased in steps, and the acoustic emission generated in the samples during primary creep was monitored. The rate v of acoustic emission events was found to decrease exponentially at each stress level, obeying the law $\log v = \beta - \alpha N$, where N is the total number of microseismic events that have occurred and α and β are constants. By assuming that acoustic emission is proportional to inelastic deformation, this relation can be compared to empirical creep laws. It is similar to the relation given by Lomnitz and fits the data more closely than other creep laws that were functions of time rather than the number of acoustic emission events. The value of α was found to decrease systematically with increasing differential stress, and in one experiment became negative before sample failure.

● 2.9-13 Levy, N. A. and Mal, A. K., **The influence of source parameters on strong ground motion,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 458-463.

Near-field ground motions are calculated from a rapidly propagating vertical fault located in a homogeneous isotropic viscoelastic halfspace. A model of the source is constructed by retaining the essential kinematic features of the faulting process. Synthetic displacements and velocities are calculated by means of an efficient computer program. The influence of the rupture speed on the nature of the ground motion is investigated by varying the speed between the Rayleigh and the P-wave speed of the medium. The differences in the nature of the motions produced by strike-slip and dip-slip faultings of equal seismic moments are studied. The relative magnitudes of the body waves and the surface waves in near-field motions are compared. An effort is made to determine the length of the fault segment which produces the significant strong motion at a given location near a long fault.

● 2.9-14 Singh, S. K., **Force pulse on a circular area as an earthquake model for near-field ground motion,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 529-534.

Near-field ground motion due to an earthquake is modeled in terms of tangential stress pulse applied over a circular area. The formulation is mathematically tractable and physically reasonable. Finite-rupture velocity and coupling sources may also be incorporated. The case of suddenly applied tangential stress is considered in detail and numerical results are given.

● 2.9-15 Ishida, K. and Osawa, Y., **Strong earthquake ground motions due to a propagating fault model considering the change of dislocation velocity - Parkfield earthquake of 1966,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 535-540.

Among factors exerting influence on short-period ground shaking, the source mechanism of the fault seems to be one of the most important problems. From this point of view, the authors propose the formulation of a source function taking account of the change of dislocation velocity in order to explain the short-period component of ground shaking which exerts an important effect on the majority of buildings. To compare the theoretical ground motion calculated from this source function with the observed one, the Parkfield earthquake of 1966 was analyzed. From this comparative study, it was found that the theoretical ground motions were able to well represent the feature of observed displacement, velocity, and acceleration.

● 2.9-16 Nair, K. and Cluff, L. S., **An approach to establishing design surface displacements for active faults,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 811-816.

The concept of subjective probability is suggested for defining the uncertainties associated with specification of seismic design criteria. Techniques for assessing the uncertainties associated with different amounts of surface fault displacement are discussed. It is argued that the explicit consideration of uncertainty is necessary for determining the risk associated with the location and design of structures in seismically active areas, especially those that must be located across active faults.

2.9-17 Briggs, P., Press, F. and Guberman, Sh. A., **Pattern recognition applied to earthquake epicenters in California and Nevada,** *Geological Society of America Bulletin,* 88, 2, Feb. 1977, 161-173.

Areas of California and Nevada that are particularly earthquake prone are identified with a computer by applying a pattern-recognition algorithm to standard geological data. The algorithm, which is designed for varied application, defines suites of characteristic traits, and successfully discriminates earthquakes characteristic of the San Andreas fault system from those characteristic of the western Basin and Range province. It also pinpoints areas in California and Nevada that are unlikely to be the epicenters of strong earthquakes. Control experiments to test the predictive ability of this technique have shown imperfect but positive results. Predictions of future earthquake epicenters are made. Features characteristic of earthquake-prone areas emerge that appear physically meaningful in terms of large-scale geology. Some of these characteristics were not obvious before.

● 2.9-18 Papastamatiou, D., **Near field stochastic modelling,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 643-647.

A stochastic model is suggested for seismic dislocations. The model gives insight into generalized seismic sources and a guide to aseismic design for surface ruptures.

2.9-19 Shamina, O. G. and Khanutina, R. V., **The nature of fracture at weak-shock sources prior to and after a strong earthquake,** *Physics of the Solid Earth,* 13, 5, 1977, 328-334.

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The behavior of the shear plane and the movements at the source of weak shocks preceding and accompanying a strong earthquake are analyzed from the standpoint of fracture mechanics. The regularities obtained are compared with the results of tectonophysical studies.

2.9-20 Budnikov, V. A., Volarovich, M. P. and Fayzullin, I. S., **Velocities of elastic longitudinal waves during nonelastic deformations of rock specimens,** *Physics of the Solid Earth,* **13,** *3,* 1977, 189-195.

Results are described of an experimental study of the variation of the propagation velocity of longitudinal elastic waves during the loading of crystalline rock samples up to deformations of about 10%. The deformation was carried out in accordance with a modified matrix method which revealed a region of developed nonelastic deformations in rocks that previously had been considered brittle. Longitudinal wave velocity decreases in the region of nonelastic deformations. Complete water saturation of the specimens did not affect the deformation characteristics but resulted in reduced relative variations of the velocities during loading.

● 2.9-21 Sugimura, Y., **Seismic strain induced in the ground during earthquakes,** *UCB/EERC-77/14,* Earthquake Engineering Research Center, Univ. of California, Berkeley, June 1977, 58.

This report develops a method of estimating seismic ground strain by using techniques for strong-motion data and presents some computed results from these techniques. Simple plane wave forms are applied to seismic records for an estimation of strain.

The two earthquakes selected for this study have shallow focal depths and strong ground motions. For these earthquakes, the parallel and normal directions to the causative fault line are shown to form a pair of orthogonal axes, along which the mean square intensities of the components of ground motion have maximum and minimum values. The velocity curves are transformed to this coordinate system. Then the seismic shear strain component at each frequency is evaluated by applying Fourier analysis and the simple plane wave solution.

2.9-22 Riznichenko, Yu. V., Seyduzova, S. S. and Matasova, L. M., **A macroseismic program,** *Physics of the Solid Earth,* **13,** *3,* Oct. 1977, 150-158.

A technique and a computer program are suggested for the processing of macroseismic observational data concerning a number of earthquakes with a view to determining the theoretical intensity field parameters of seismic tremors optimally conforming to the entire observational data. A program for the application as well as the improvement of the methods of processing and the interpretation of the macroseismic data are discussed. Physical and engineering characteristics of the seismic tremors, the foci parameters, and the surrounding medium are also taken into account.

● 2.9-23 Olsson, R., **Some aftershock sequences in the Japan-Kamchatka region,** *3-76,* Seismological Inst., Uppsala, Sweden, 1976, 17.

The aftershock sequences of 1952, 1969, 1973 at the south end of the Kurile arc have been compared with respect to the deformation release. A similarity, which has been interpreted as similar stress states in the aftershock volumes, is indicated. The magnitude-frequency law parameters have also been compared for the three sequences and with the Kurile sequences of 1958 and 1963, the Tokachi-oki sequence of 1968, and the Kamchatka sequence of 1952. There is a significant increase of the b-coefficients from south to north within this belt, suggesting a corresponding increase of heterogeneity.

2.9-24 Kieckhefer, R. M., **Microseismicity in the vicinity of the Clarence fault, New Zealand,** *New Zealand Journal of Geology and Geophysics,* **20,** *1,* 1977, 165-177.

A thirteen-day microearthquake survey in April 1975 showed little activity along the Clarence fault; however, moderate upper crustal microseismicity (1.8 events per day with $-0.8 \leq M \leq 1.9$) occurred along the neighboring Fowlers fault, a relatively minor active fault, as well as some scattered activity (2.1 events per day) unrelated to major faults. Focal mechanisms of shallow events, with the possible exception of microaftershocks in one area, were consistent with geologically observed dextral faulting on mapped faults. A marked decrease in microseismicity occurs below 13 km depth. A composite first-motion plot for subcrustal microearthquakes implies that a variety of focal mechanisms may exist. One of these, probably the dominant one in the region, disagrees with the surface tectonics.

● 2.9-25 Khattri, K. N., Singh, V. N. and Gaur, V. K., **Tectonic stress estimate for the Koyna earthquake of December 11, 1967,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 829-835.

An analysis of the strong-motion accelerogram of the Koyna earthquake of Dec. 11, 1967, has been carried out to determine the values of the initial tectonic and the effective stresses using the acceleration spectrum and the peak velocity. The estimated effective stress obtained from the acceleration spectrum is 350 bars, which is an order of magnitude higher than that obtained using peak velocity. This figure, however, is considered to be more reliable.

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2.9-26 Leblanc, G. and Buchbinder, G., **Second microearthquake survey of the St. Lawrence Valley near La Malbaie, Quebec,** *Canadian Journal of Earth Sciences,* **14,** *12,* Dec. 1977, 2778-2789.

A second microseismicity survey was conducted in the La Malbaie region during two summer months of 1974. As many as 19 stations were occupied simultaneously for most of the period. Thirty-four microearthquakes were located using at least four stations; the epicenters cover a region of about 70 x 40 km, centered in the St. Lawrence River. Thirty-three of the events are within the station configuration. All hypocenters are in the Precambrian rocks of the Grenville Province at an average focal depth of 11 km; none was located in the Ordovician sediments. Six fault-plane solutions were obtained, all having one nodal plane striking from north to northeast; thrust motion is predominant. On the basis of the confined and clearly delineated distribution of the hypocenters the influence of the Charlevoix impact structure on the length of the active zone is again proposed; this idea had already been advanced after the first microearthquake survey in 1970. The conjunction of this structure with Logan's line could also have some influence on the seismicity. Strikingly, both the focal parameters and rate of occurrence of one event per two days observed in the two surveys are in agreement. Finally, it is suggested, in view of the confined zone of microactivity, that the epicentral distribution of historical events could be regarded as less extensive than it appears.

2.9-27 Basili, A. *et al.,* **Earthquake occurrence as stochastic event: (1) theoretical models,** *Bollettino di Geofisica,* **XX,** *73-74,* Mar.-June 1977, 3-10.

The present article intends to combine the stochastic approach in the description of earthquake processes suggested by Lomnitz with the experimental evidence reached by Schenkova that the time distribution of some earthquake occurrences is better described by a negative bionomial distribution than by a Poisson distribution. The final purpose of the stochastic approach might be a new way to label an area in terms of seismic risk.

2.9-28 Maeda, I., **On the preshock, preslip, and the time dependent effect of stick-slip motion** (in Japanese), *Zisin, Journal of the Seismological Society of Japan,* **30,** *1,* Apr. 1977, 55-72.

Dynamic and static characteristics of stick-slip motion are studied experimentally for Horoman peridotite. The pressure system is similar to that of Hoskin et al. Normal stress is constant (80 bar) throughout all experiments. The stress rate is varied from 20 bar/sec to 0.001 bar/sec. Strain is measured at the side surface which is parallel to two pressure axes. Shocks accompanied by slips are detected by a piezoelectric transducer which is attached to a slip plane of central rock block. The overall dynamic range and frequency range of the recording system are about 90dB and DC~10 MHz respectively.

2.9-29 Noguchi, S. and Abe, K., **Earthquake source mechanism and M_S-m_b relation** (in Japanese), *Zisin, Journal of the Seismological Society of Japan,* **30,** *4,* Dec. 1977, 487-507.

The relation between surface-wave magnitude M_S and body-wave magnitude m_b is studied on the basis of measurents of M_S and Haskell's deterministic fault model. The fifty-four measurements of M_S are made over a wide range of magnitude for three large shocks and their aftershocks which occurred in 1968, 1969, and 1973 along the southern Kurile trench. For a constant m_b, M_S values of the aftershocks of the 1968 earthquake are systematically larger by 0.5 to 1.0 than those of the aftershocks of the 1973 earthquake. This systematic difference can be explained in terms of the difference in stress drop; stress drops of the 1968 events are considered to be about four to five times lower than those of 1973 events. This is comparable to the difference of stress drops between the two main shocks. There are abnormal aftershocks which show a remarkable deviation from the average M_S-m_b relation. It is suggested that this deviation represents a breakdown of the assumption of one similarity law. Among various parameters such as stress drop, rupture velocity, and rise time of dislocation function, the ratio of rise time to fault length is found to account for the abnormal values of magnitude. This ratio tends to increase with an increase of M_S for some aftershocks of the 1973 earthquake.

2.9-30 Fitterman, D. V. and Madden, T. R., **Resistivity observations during creep events at Melendy Ranch, California,** *Journal of Geophysical Research,* **82,** *33,* Nov. 10, 1977, 5401-5408.

A small-scale resistivity monitor along the San Andreas fault at Melendy Ranch, California, was unable to detect resistivity variations associated with fault creep. No resistivity variations which could be attributed to creep activity were detected, even when sensitivities of 0.005% were employed. The absence of a creep-associated resistivity change is attributed to the electrical behavior of clay during stressing and to the narrowness of the creep zone compared to the measurement geometry.

2.9-31 Rudnicki, J. W., **The inception of faulting in a rock mass with a weakened zone,** *Journal of Geophysical Research,* **82,** *5,* Feb. 10, 1977, 844-854.

This paper investigates models for the inception of earth faulting based on the deformation of a rock mass containing an embedded weakened zone. Constitutive laws appropriate to dilatant, frictional inelastic behavior are used to characterize the weakened zone material. Two distinct types of instability corresponding to possible models of seismic mechanisms are identified. These are "localization" instabilities, at which essentially homogeneous deformation gives way to localized shearing, and "runaway" instabilities, at which no further quasistatic deformation is possible and inertial effects dominate. Conditions derived for the onset of these instabilities demonstrate that the amount of postpeak deformation in the weakened zone prior to instability is strongly dependent on the deviatoric state of stress induced within the weakened zone and on the detailed nature of the inhomogeneities. In particular, instability is predicted much nearer to peak load for very narrow weakened zones and for states of deviatoric pure shear than for states of axisymmetric compression. Hence, the premonitory events predicted by "dry crack" precursor models, which associate crack closure with the postpeak regime, would be dramtically different for these two cases. More generally, systematic differences may be observed between strike slip and thrust type faults. A discussion of the qualitative effects of coupled stress-pore fluid diffusion on instability suggests a new interpretation of the dilatancy-diffusion model and indicates that premonitory events predicted by this model may also depend on the amount of postpeak deformation prior to instability.

2.9-32 O'Connell, R. J. and Budiansky, B., **Viscoelastic properties of fluid-saturated cracked solids,** *Journal of Geophysical Research,* 82, *36,* Dec. 10, 1977, 5719–5736.

The effective elastic moduli of a fluid-saturated solid containing thin cracks depend on the degree of interconnection between the cracks. Three separate regimes may be identified: (1) dry (drained), in which fluid in cracks can flow out of bulk regions of compression, (2) saturated isobaric, in which fluid may flow from one crack to another but no bulk flow takes place, and (3) saturated isolated, in which there is no communication of fluid between cracks. Transitions between these cases involve fluid flow, resulting in dissipation of energy. Relaxation of shear stresses in viscous fluid inclusions also results in dissipation. Viscoelastic moduli are derived, by using a self-consistent approximation, that describe the complete range of behavior. There are two characteristic frequencies near which dissipation is largest and the moduli change rapidly with frequency. The first corresponds to fluid flow between cracks, and its value can be estimated from the crack geometry or permeability. The second corresponds to the relaxation of shear stress in an isolated viscous fluid inclusion; its value may also be estimated. Variations of crack geometry result in a distribution of characteristic frequencies and cause Q to be relatively constant over many decades of frequency. Fluid flow between cracks accounts for attenuation of seismic waves in water-saturated rocks and attenuation observed in laboratory measurements on water-saturated rocks and partially molten aggregates. Attenuation in a partially molten upper mantle is probably due to fluid flow between cracks, although grain boundary relaxation in an unmelted upper mantle could also account for the seismic low-velocity zone. Grain boundary relaxation in the mantle may cause the long-term shear modulus to be around 20% less than that measured from seismic observations.

3. Engineering Seismology

3.1 General

● **3.1-1** Terashima, T. and Santo, T., **A new scale representing the "quake sensitivity" at a certain region,** *Wind and Seismic Effects,* IV-47–IV-54. (For a full bibliographic citation, see Abstract No. 1.2-4.)

The nature of ground motions generated by earthquakes depends upon the properties of the ground surface, which vary in different areas. In this paper, the different characteristics of ground motions are normalized by a new scale designated as "Quake-Sensitivity." This scale is defined as a ratio of N(I)/S, where N(I) is the annual mean frequency of seismic intensity of more than III (JMA scale) and S is the seismicity index which has been defined previously by the authors. A seismic zoning map was then made for the Japan Islands using the new scale. The area having a large value of N(I)/S means that the area is sensitive to earthquake motions or that the neighboring area has moderate earthquakes.

● **3.1-2** Vanmarcke, E. H. and Lai, P. S., **Strong-motion duration of earthquakes,** *Evaluation of Seismic Safety of Buildings, No. 10, R77-16,* Dept. of Civil Engineering, Massachusetts Inst. of Technology, Cambridge, July 1977, 32.

A simple procedure is proposed to estimate the strong-motion duration of earthquake ground motions. The proposed strong-motion duration is nearly proportional to the quantity of I_o/a^2_{max}, in which a_{max} is the maximum ground acceleration and I_o is the integral of the squared accelerations. The proportionality factor is weakly dependent on the predominant period of the ground motion. The procedure has been applied to 140 horizontal components of strong earthquake ground motions. The results are tabulated and summarized in the form of histograms. The dependence of strong-motion duration on earthquake magnitude and epicentral distance is also examined.

● **3.1-3** Cosentino, P., Ficarra, V. and Luzio, D., **Truncated exponential frequency-magnitude relationship in earthquake statistics,** *Bulletin of the Seismological Society of America,* 67, *6,* Dec. 1977, 1615–1623.

A doubly truncated exponential probability density distribution is proposed for the earthquake occurrence. The relation has been obtained carrying out a simple model based on a number of assumptions, among which the more characterizing is the existence of a maximum regional finite magnitude value. This assumption, derived by evidence recognized by most seismologists, allows a simple explanation of the known behavior of the experimental cumulative frequency-magnitude graphs. Some results of application of the model to six seismic regions are presented.

● **3.1-4** Murphy, J. R. and O'Brien, L. J., **The correlation of peak ground acceleration amplitude with seismic intensity and other physical parameters,** *Bulletin of the Seismological Society of America,* 67, *3,* June 1977, 877–915.

An analysis of acceleration/intensity correlations has been carried out using a new worldwide data sample compiled for this study from data measured from nearly 1500 strong-motion accelerograms. This new data sample has been extensively analyzed using a variety of statistical models. It has been found that the correlation equation relating peak horizontal ground acceleration (a_H) to modified Mercalli intensity (I_{MM}) which best describes the trends in the subset of this new sample consisting of the nearly 900 observations for which $a_H \gtrsim 10$ cm/sec^2 is $\log a_H = 0.25\, I_{MM} + 0.25$ for a_H given in cm/sec^2. Analyses of the dependence of this correlation on other variables such as local earthquake magnitude (M), epicentral distance (R), and the geographical region in which the earthquakes are occurring suggest that over the range of these variables

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encompassed by the available data, the correlation equation which best fits the data is $\log a_H = 0.14\ I_{MM} + 0.24M - 0.68 \log R + \beta_k$ where R is given in kilometers and $\beta_{\text{western United States}} = 0.60$, $\beta_{\text{Japan}} = 0.69$, and $\beta_{\text{southern Europe}} = 0.88$.

Thus, the southern European data indicate peak horizontal accelerations at fixed values of intensity, magnitude, and epicentral distance which are about a factor of two higher than the corresponding values for the western United States and Japan. Sufficient data are not yet available to determine whether this difference is due to a consistent measurement bias associated with the assignment of intensities in southern Europe or to variations in the regional tectonic environment in which the earthquakes are occurring.

● **3.1-5** Street, R. L. and Turcotte, F. T., **A study of northeastern North American spectral moments, magnitudes, and intensities,** *Bulletin of the Seismological Society of America,* **67**, *3,* June 1977, 599–614.

Thirty-two earthquakes in northeastern North America ranging in seismic moments of 1.5E19 to 6.0E26 (dyne-cm) are used to develop relationships between spectral moment, magnitudes (m_{bLg} and M_S), and intensity. It is shown that northeastern North American events are readily characterized by a well-behaved, steadily increasing stress drop relative to increasing magnitude. The relationship between the regional m_{bLg} and M_S as implied by the spectral results is given. Published relationships between the m_{bLg} magnitude, the $\log_{10}$ of the area within the intensity IV isoseismal contour, and the falloff of the intensity technique described by Nuttli are checked for suitability. The results are presented.

● **3.1-6** Dobry, R. *et al.*, **Influence of magnitude, site conditions and distance on significant duration of earthquakes,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 464–469.

A study was made of significant duration of horizontal motions recorded in the western U.S.A. on site conditions ranging from rock to soft clays. Consistent correlations were found between the duration on rock, magnitude, and distance to the source. Durations at soil sites show more scatter, with the duration on rock for similar magnitude or distance being a lower bound. These larger durations for soils are usually caused by long-period motions at the end of the record, which may be associated with surface waves.

● **3.1-7** Blume, J. A., **The SAM procedure for site-acceleration-magnitude relationships,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 416–422.

Early work on the relationships of site characteristics, horizontal peak accelerations, magnitude, attenuation with distance, and probabilistic variations is extended with more data, refined and simplified. New estimation procedures called SAM (site-acceleration-magnitude) IV and SAM V are provided to supersede previous SAM versions. The data used include all California and western Nevada strong-motion records from 1933 through 1970 and, for studies of rock and alluvial motion, statistics from 2713 records of ground motion induced by underground nuclear explosions were used. Comparisons with studies and estimation procedures of others are provided.

● **3.1-8** Saragoni, G. R., **The $\alpha\beta\gamma$ method for the characterization of earthquake accelerograms,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 357–364.

A characterization method for the time evolution of acceleration amplitudes of earthquake accelerograms using three parameters is presented. The method considers that mean square acceleration tends to a chi-square function when the parameters characterize the time evolution of acceleration amplitudes for each type of record. The parameters are easily estimated using a time-moment technique. The method has been applied to 32 accelerograms from the U.S.A., Mexico, Peru, and Chile with satisfactory results. In addition, a duration of strong-motion region is defined in terms of two of the parameters. Using this definition, the distribution of expected energy among buildup, strong motion, and endup is studied for different values of the parameters. Expressions for the parameters in terms of epicentral distance and Richter magnitude are also presented for earthquake design on the west coast of the U.S. Finally, an expression for the estimation of the expected maximum ground acceleration and instrumental intensities in terms of the parameters are also given.

● **3.1-9** McGuire, R. K., **The use of intensity data in seismic-hazard analysis,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 709–714.

Relations between peak ground-motion parameters and modified Mercalli (MM) intensity, based on strong-motion data obtained in California, are presented. Peak ground acceleration and displacement, when related to MM intensity, are found to depend also on source-to-site distance, whereas velocity does not. Several alternate methods of deriving design ground-motion values for the eastern United States, where only intensity data are available, are discussed and illustrated. The predicted ground velocity for an event of given size and distance in the eastern United States is the same whether strong ground motion as a function of intensity is assumed to be similar to California, or whether high intensities at long distances in the East are assumed to result from lower levels of shaking but longer

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durations. Design levels of acceleration and displacement, however, do depend on which hypothesis is assumed.

● 3.1-10 Espinosa, A. F. and Lopez-Arroyo, A., **Earthquake instrumental intensity from strong ground motion records: San Fernando earthquake,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 722–728.

The accelerograms recorded at 61 stations following the Feb. 9, 1971, earthquake and their first integration are used to evaluate a new definition of instrumental intensity. This new quantity, calculated from each component of ground motion in the acceleration and velocity time domain, takes into consideration: (1) the maximum accelerations and velocities, and (2) the time duration of strong motions. These parameters are of utmost importance in the evaluation of intensity and energy. The Arias intensity is computed and compared with the new instrumental intensity and with modified Mercalli intensity (MMI). A number of predictive equations expressing the maximum particle velocity as a function of MMI have been derived along with attenuation relationships in terms of MMI, Arias, the new instrumental intensity, and epicentral distance. The low instrumental intensity correlates well with hard rock; moderate intensities, with unconsolidated material; and higher intensities, with alluvial deposits.

● 3.1-11 Trifunac, M. D., **An instrumental comparison of the modified Mercalli (M.M.I.) and Medvedev-Karnik-Sponheuer (M.K.S.) intensity scales,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 715–721.

For many parts of the world, scales similar to the Modified Mercalli Intensity Scale, which is employed in the United States, are still widely used for mapping the descriptive characteristics of the effects of strong earthquake ground motion on man-made structures and near-surface rocks and soil. In spite of the fact that these intensity maps depend in an important way on the methods employed to prepare them and on the type of construction in the shaken area, in many seismic regions this information on the earthquake-generated effects still represents the useful data which are available for seismic risk analysis. As instrumental data on strong shaking are now becoming available, primarily for the western United States and Japan, it is possible to correlate the reported intensity levels with different characteristics of recorded accelerograms and thus derive empirical relationships that may be useful for approximate estimation of strong-motion amplitudes in terms of the modified Mercalli intensity or its equivalent. The results of this analysis show that, although in the intensity range between IV and VII MKS and MMI scales have similar definitions, the actual assigned intensity levels can differ by approximately one intensity unit for the average instrumental strength of shaking as measured by the SBM seismometer.

● 3.1-12 Gupta, I. N., **Attenuation of intensities and design earthquakes in central U.S.A.,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 948–949.

● 3.1-13 Herrmann, R. B., **Earthquake generated SH waves in the near field and near-regional field,** *Misc. Paper S-77-12,* Dept. of Earth and Atmospheric Sciences, St. Louis Univ., Missouri, Aug. 1977, 51.

The prediction of earthquake stong-motion values for aseismic design purposes is inhibited by scatter in previous correlations. Some of the scatter may be eliminated by a better understanding of the process of seismic wave transmission. This report is concerned with developing a methodology for predicting SH wave ground motion from earthquake sources in the distance range of 5-500 km for waves with frequencies less than 1.0 Hz. Suggestions for extending the methodology are also made.

● 3.1-14 Poceski, A., **An intensity definition of the strong motion earthquakes,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 872–877.

The intensity of strong-motion earthquakes is defined in this paper as a combination of the average pseudorelative response velocity and the root mean square of ground velocity. This intensity is normalized so that, for an earthquake of an intensity of V on the MM scale, a unit intensity is assigned. For MM intensities of VI and VII, the proposed intensities are 2 to 3 and 6 to 12, respectively. The strongest intensity, 43, is obtained for the Pacoima Dam records of the 1971 San Fernando earthquake. As compared to Housner's spectral intensity, the proposed definition results in an increased intensity for long duration earthquakes and a decreased intensity for short duration earthquakes.

● 3.1-15 Whitman, R. V., Aziz, T. S. and Wong, E. H., **Preliminary correlations between earthquake damage and strong ground motion,** *Seismic Design Decision Analysis Report No. 29,* Dept. of Civil Engineering, Massachusetts Inst. of Technology, Cambridge, Feb. 1977, 50.

Correlations are made between damage to buildings during the San Fernando earthquake of 1971 and the recorded strong ground motions. The area where the modified Mercalli intensity was VII is divided into square zones, each about 4 miles on a side; for each zone, several strong-motion records were averaged. Data regarding damage is taken from the Massachusetts Inst. of Technology building damage survey. In one type of comparison, mean

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damage ratio for each zone is correlated with peak acceleration, peak velocity, Housner intensity, and Arias intensity. In a second comparison, damage ratio is correlated to measures of structural response derived from response spectra. Some trends are observed, and the results serve as a starting point for further studies.

● 3.1-16 Krinitzsky, E. L. and Chang, F. K., **State-of-the-art for assessing earthquake hazards in the United States. Report 7: specifying peak motions for design earthquakes,** *Misc. Paper S-73-1,* U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, Dec. 1977, 34.

The large dispersion of data for components of earthquake motion requires that the spread be appraised in design applications. Instrumental data also must be related to historic records of intensity. The near field and the far field contribute greatly to differences in peak motions. Site conditions, soil versus rock, affect duration. With these considerations, and with geological studies and the probability of recurrence, peak values can be specified from parameters of motions related to modified Mercalli intensities. These peak values can be used for rescaling selected strong-motion records or alternatively for the generation of synthetic seismograms. The procedure incorporates the wide variability in ground motions that have occurred during earthquakes.

3.2 Strong Motion Records, Interpretation, Spectra

● 3.2-1 Stephenson, W. R., **Using the Caltech strong motion earthquake records,** *500,* Physics and Engineering Lab., Dept. of Scientific and Industrial Research, Lower Hutt, New Zealand, Feb. 1976, 9.

The report contains instructions for the use of the Caltech strong-motion records. It consists of the following major sections: specifying records; transferring records from magnetic tape to a labelled core region; inputting accelerations from a labelled core region into a FORTRAN program; and finding site descriptions. The report also describes the use of some of these records which are stored at the Vogel Computing Centre, Wellington, New Zealand.

● 3.2-2 Kameda, H., **Earthquake motions parameters affecting structural response statistics,** *Proceedings of U.S.-Japan Seminar on Earthquake Engineering Research with Emphasis on Lifeline Systems,* Japan Society for the Promotion of Earthquake Engineering, Tokyo, Nov. 1976, 79–92.

Dynamic properties of earthquake motion are reviewed in relation to the uncertainty of structural response. Effects of randomness of earthquake motion on the variability of response spectra are analyzed with emphasis on the frequency content and duration of the earthquake motion. Effect of duration is also discussed in regard to inelastic response, from which an appropriate definition of the duration of earthquake motion is proposed.

3.2-3 Stoykovich, M., **Development and use of seismic instructure response spectra in nuclear plants,** *Nuclear Engineering and Design,* 38, 2, Aug. 1976, 253–266. (For an additional source, see Abstract No. 1.2-2.)

This paper encompasses methods for the development of in-structure response spectra as well as the use of these spectra in the seismic design and analysis of nuclear plant components. The time history modal analysis method to generate in-structure response spectra is described. This includes the effects of rigid body transformation associated with angular accelerations of the lumped mass nodal points due to eccentric locations of the equipment or system support points. A general method of generating and using the in-structure response spectra associated with both translational and angular input motion is presented. An approximate way of treating light equipment mounted on relatively flexible floors or shear walls is considered.

Various numerical techniques for the integration of differential equations of motion are outlined. The time interval to be used is chosen so as to avoid mathematical instability and inaccuracy. In order to verify solution accuracy, comparison of the results using different techniques for the numerical integration of a sample problem is provided. Consideration is given to determining how small the period interval should be in generating in-structure response spectra to avoid truncation of resonance peaks. The use of three-dimensional in-structure response spectra developed for each of the three orthogonal translational directions of ground motion is presented. Methods of developing in-structure response spectra other than the time history method are discussed.

● 3.2-4 Shteinberg, V. V., Pletnev, K. G. and Graizer, V. M., **Ground motion accelerograms of the strong Gazli earthquake on May 17, 1976** (Akselerogramma kolebanii grunta pri razrushitelnom gazliiskom zemletryasenii 17 maya 1976 g., in Russian), *Seismostoikoe stroitelstvo, 1,* 1977, 45–61.

A magnitude 7.2 earthquake with a focal depth of 25–30 km and the epicenter located at the village of Gazli took place on May 17, 1976. The main shock was recorded with a strong-motion apparatus located at the epicenter and utilizing direct optical recording. Digitized records showing three ground acceleration components are presented. These data are unique and have great practical

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interest for researchers in the field of earthquake engineering.

● 3.2-5 Mamaeva, G. V., Fedyakov, M. V. and Fedyakova, S. N., **Analysis of ground motion and building response based on results of initial processing of earthquake records from Petropavlovsk-Kamchatskii** (Analiz kolebanii gruntov i zdanii po rezultatam pervichnoi obrabotki zapisei zemletryasenii v Petropavlovske-Kamchatskom, in Russian), *Seismostoikoe stroitelstvo, 2,* 1977, 47–52.

The stations of the engineering seismology recording network in Petropavlovsk-Kamchatskii registered 19 earthquakes of medium-to-high intensity during 1974 and 1975. The foci of these earthquakes were within 300 km of the city and focal depths varied between 10 km and 170 km. The behavior of buildings is analyzed on the basis of initial processing of records of these 19 earthquakes. Qualitative comparisons of the earthquake parameters are made.

● 3.2-6 Fedyakova, S. N., **Methods for processing instrumental data from the engineering seismology recording network** (Metodika obrabotki instrumentalnykh dannykh inzhenerno-seismometricheskoi sluzhby, in Russian), *Seismostoikoe stroitelstvo, 2,* 1977, 40–46.

Seven stations of the engineering seismology recording network capable of recording ground motion and structural response are located in Petropavlovsk-Kamchatskii. About 40 medium-to-strong earthquakes have been recorded to date. In order to process such a great volume of data, a computerized technique was developed to calculate ground motion spectra by means of the fast Fourier transform. A detailed discussion of the numerical methods used is given.

● 3.2-7 Iwasaki, T., Wakabayashi, S. and Tatsuoka, F., **Characteristics of underground seismic motions at four sites around Tokyo Bay,** *Wind and Seismic Effects,* III-41–III-56. (For a full bibliographic citation, see Abstract No. 1.2-4.)

This paper discusses the dynamic behavior of subsurface soil and rock layers on the basis of acceleration records triggered during small-to-moderate earthquakes. Borehole accelerometers are installed at four sites around the Bay of Tokyo. These were installed in 1970–74, in connection with the Tokyo Bay Loop Highway Project proposed by the Ministry of Construction.

Important acceleration records were obtained during sixteen moderate earthquakes (magnitude 4.8 to 7.2) which occurred near the area in Sept. 1970 through Feb. 1975. From distributions of maximum accelerations at the four stations, it seems that the surface magnification factors (ratios of the surface acceleration to the base acceleration) are large (2.5 to 3.5) at the soft clayey soil site, small (about 1.5) at the rocky site, and medium (1.5 to 3) at sandy soil sites.

Response spectrum curves from typical acceleration records are shown. Comparison of the spectral curves from records obtained at three levels of one station during an earthquake suggests that frequency characteristics at the several depths are similar. Also, it seems that frequency characteristics of earthquake ground motions are influenced by seismic conditions (such as magnitudes, epicentral distances, etc.), as well as soil conditions at the sites.

● 3.2-8 Hayashi, S., Tsuchida, H. and Kurata, E., **Observation of earthquake response of ground with horizontal and vertical seismometer arrays,** *Wind and Seismic Effects,* III-16–III-25. (For a full bibliographic citation, see Abstract No. 1.2-4.)

A horizontal seismometer array, having six observation points along a straight line of 2500 m in length, has been established at the Tokyo International Airport. Each observation point is equipped with two horizontal seismometers. Downhole seismometer arrays have also been established at two points, one at the end of the observation line and the other at a point 500 m inside from the other end of the line. The observation started in Apr. 1974 and since then 28 earthquakes have been recorded as of June 1975. Correlations among the ground motions at the points on the ground surface and the two points in the ground where the downhole seismometers have been installed have been studied. The relative displacements between the points have also been studied. It was assumed that there had been a straight pipeline made of steel along the observation line and the pipe motions had been equal to the observed ground motions. The stresses in the pipe were estimated, and it was found that the stresses due to axial deformations were remarkably larger than those due to the bending deformations.

● 3.2-9 Toki, K., **Strain amplitude by body and surface waves in a near surface ground,** *Proceedings of U.S.-Japan Seminar on Earthquake Engineering Research with Emphasis on Lifeline Systems,* Japan Society for the Promotion of Earthquake Engineering, Tokyo, Nov. 1976, 15–28.

The phase velocity dispersion curves for the frequency range from 0.2 to 2 Hz were detected from strong-motion earthquake records which were obtained during the San Fernando earthquake, Feb. 9, 1971. The resulting dispersion curves indicate that the phase velocity is greatly dependent on frequency, especially in the frequency range from 0.3 to 1.0 Hz. Using the dispersion curves, the author computes the time traces of strain at the ground surface for a type of strain component attributed to surface and body waves traveling along the ground surface. The maximum strain amplitudes associated with surface waves are comparable with the shear strain induced on a horizontal plane of a soil element by the vertical incidence of an SH wave in near-surface ground.

● **3.2-10** Tsuchida, H. and Kurata, E., **Observed earthquake ground displacements along a 2500 meter line,** *Proceedings of U.S.-Japan Seminar on Earthquake Engineering Research with Emphasis on Lifeline Systems,* Japan Society for the Promotion of Earthquake Engineering, Tokyo, Nov. 1976, 29–42.

A horizontal seismometer array having six observation points along a straight line of 2500 m length has been established. Downhole seismometer arrays also have been established at two points. Records of four earthquake events are selected and analyzed, while 35 earthquake events had been recorded as of Sept. 1976. The relative displacements among the observation points on the ground surface are studied, and stresses in a buried pipeline due to the relative displacements are estimated.

● **3.2-11** Iwasaki, T. and Katayama, T., **Statistical analysis of strong-motion earthquake response spectra,** *Proceedings of U.S.-Japan Seminar on Earthquake Engineering Research with Emphasis on Lifeline Systems,* Japan Society for the Promotion of Earthquake Engineering, Tokyo, Nov. 1976, 59–77.

This paper briefly describes the present status of strong-motion observation for engineering structures in Japan. It also presents results of an ongoing analysis of the characteristics of strong-motion records. Numerous acceleration records triggered during moderate to strong earthquakes are being analyzed.

From the analyses, the effects of earthquake magnitude, epicentral distance, and subsoil conditions on the absolute acceleration response are quantitatively evaluated. Since the analysis has not been completed yet, this paper is a progress report of the work being conducted to estimate design seismic forces for structures.

● **3.2-12** Hall, Jr., J. R., Shukla, D. K. and Kissenpfennig, J. F., **Cyclic characteristics of earthquake time histories,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(a), Paper K 1/7, 11. (For a full bibliographic citation, see Abstract No. 1.2-5.)

This paper presents a computer approach which describes a time history by an amplitude envelope and a phase curve. Using fast Fourier transform techniques, an earthquake time history is represented as a projection along the x-axis of a rotating vector—the length of the vector is given by the amplitude spectra—and the angle between the vector and x-axis is given by the phase curve; thus one cycle is completed when the vector makes a full rotation. Based upon Miner's cumulative damage concept, the computer code automatically combines the cycles of various amplitudes to obtain the equivalent number of cycles of a given amplitude. To illustrate the overall results, the cyclic characteristics of several real and synthetic earthquake time histories have been studied and are presented in the paper, with the conclusion that this procedure provides a physical interpretation of the cyclic characteristics of earthquakes.

● **3.2-13** Watabe, M., **Research on design earthquake,** *BRI Research Paper No. 67,* Building Research Inst., Tokyo, Sept. 1976, 21.

The maximum values of accelerations, velocities, and displacements of earthquake records are first discussed utilizing historical and instrumental data as well as some theoretical approaches. Then, duration time and deterministic intensity function of the accelerograms are introduced. The predominant periods and spectral shapes of the strong-motion accelerograms are also reported. Historical earthquake data were utilized to assess the earthquake risks in Japan. Finally, an explanation of procedures to follow in estimating a design earthquake concludes the report.

● **3.2-14** King, K. W. and Hays, W. W., **Comparison of seismic attenuation in northern Utah with attenuation in four other regions of the western United States,** *Bulletin of the Seismological Society of America,* **67**, *3,* June 1977, 781–792.

Aftershocks of the M_S = 6.0 Pocatello Valley, Idaho, earthquake of Mar. 27, 1975 (UTC), provided data for characterizing the seismic attenuation in northern Utah. Ground-motion records were recorded on a linear array of L-7 velocity seismographs deployed over an epicentral distance range of 12 to 150 km from the source area. Frequency-dependent distance attenuation exponents derived from pseudo-relative velocity response spectra show that the high-frequency (5 to 20 Hz) spectral components attenuate with epicentral distance (R) approximately as $R^{-1.8}$ and the low-frequency (0.2 to 1.0 Hz) components attenuate approximately as $R^{-1.0}$. Comparison of the attenuation data derived from the Pocatello Valley, Idaho, aftershocks with corresponding data obtained from analysis of earthquake and nuclear explosion records recorded in portions of Nevada, California, Colorado, and New Mexico reveals distinct regional similarities and differences in seismic attenuation. The distance attenuation exponents derived from the northern Utah and Colorado data are very similar in the 1 to 10 Hz range, but they indicate a more rapid attenuation rate than the exponents derived for southern Nevada and California which are very similar across the entire frequency spectrum.

● **3.2-15** Shoja-Taheri, J. and Bolt, B. A., **A generalized strong-motion accelerogram based on spectral maximization from two horizontal components,** *Bulletin of the*

Seismological Society of America, **67**, *3*, June 1977, 863–876.

A new form of strong-motion accelerogram ("spectrally maximized record" or "SMR") and its associated generalized spectrum are proposed for earthquake engineering use. Parameters (e.g., spectral, duration, peak amplitude) of strong-motion records at a given site generally depend significantly on the (arbitrary) azimuthal component, sometimes leading to a crucially deficient description of these parameters if only a single component is used. In this paper, combination of horizontal components using spectral maximization is shown to be effective in minimizing the difficulty. The spectra of the two horizontal components at each site are combined to maximize the resultant spectrum, independently of azimuthal orientation. SMRs of 33 important strong-motion accelerograms (including some New Guinea records) are then calculated from their corresponding spectra. In only 60% of the cases is the peak acceleration from a maximized spectrum greater than that of the single components. The bracketed duration from the maximized spectrum is always greater. After filtering to provide records in ten frequency bands (0–1 Hz, 1–2 Hz, ..., 9–10 Hz), correlations for each band are made between acceleration peaks, spectral energy, magnitudes, and source distances. Reasonably stable estimates of strong-motion parameters appear to be given by spectrally maximized seismograms.

● **3.2-16** Boore, D. M., **Strong-motion recordings of the California earthquake of April 18, 1906,** *Bulletin of the Seismological Society of America*, **67**, *3*, June 1977, 561–577.

Recordings from a low magnification ($V = 4$) intermediate period ($T \approx 5$ sec) seismograph at Mt. Hamilton, within 35 km of the San Andreas fault, show about 8 sec of *P*-wave energy and the first half cycle or so of the initial S-wave before going off scale. The times, polarities, and overall amplitudes are consistent with Bolt's conclusion (1968) that the main shock began closer to San Francisco than Olema, as originally proposed by Reid (1910). This and the distribution of surface slip imply a bilateral rupture, but the seismic moment for the segment to the northwest was on the order of 2.5 times greater than for the southeasterly segment. Although Mt. Hamilton is only 35 km from the rupture surface, the most massive faulting apparently took place at least 75 km away. The recording at Mt. Hamilton returned to scale after about 60 sec; the duration of energy with periods close to 5 sec was comparable to that from more recent strong-motion recordings. Theoretical modeling using both body and surface waves showed that the surface waves dominated the motion at Mt. Hamilton. The modeling also emphasized the sensitivity of the ground motion to directivity effects and rupture velocity. The general characteristics of the data (polarity, amplitude, period content, and duration) were matched reasonably well by a simple dislocation model using fault lengths, depths, and offsets determined from independent data.

● **3.2-17** Leivas, E., Cramer, C. H. and Toppozada, T. R., **Low magnitude earthquakes, high ground accelerations,** *California Geology*, **30**, *9*, Sept. 1977, 212–213.

Two minor earthquakes occurred near Oroville, California, on May 4, 1977, with Richter magnitudes of 3.1 and 3.4, respectively. Eight strong motion records were recovered from four accelerograph stations operated by the Strong Motion Instrumentation Program. The highest values of acceleration ever recorded for earthquakes in the magnitude range 3 to 3.5 were recorded at two accelerograph stations.

● **3.2-18** Umemura, H. *et al.*, **An approach to the modeling of 3-dimensional strong motions,** *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. I, 1977, 351–356.

The maximum values of accelerations, velocities, and displacements of earthquake records are first discussed utilizing historical and instrumental data as well as some theoretical approaches. Then, duration time and deterministic intensity function of the accelerograms are introduced. The predominant periods and spectral shapes of the strong-motion accelerograms are also reported. Finally, the concept of principal axes of the 3-dimensional accelerograms is demonstrated so that the simulation of 3-dimensional strong accelerograms may be stochastically possible in the very near future.

● **3.2-19** Tocher, D., Patwardhan, A. S. and Cluff, L. S., **Estimation of near field characteristics of earthquake motion,** *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. I, 1977, 470–476.

Estimation of strong earthquake ground motion in the near field is a complex problem because, in addition to the earthquake magnitude and distance, source characteristics and geometry, and the characteristics of transmission path significantly influence the attenuation of ground motion. Available strong-motion data in the near field are scanty and frequently need reevaluation. Because uncertainties are associated at present with the actual values and physical relationships governing source characteristics, a probabilistic approach offers considerable advantage for a systematic evaluation of data. The importance of reevaluation of geologic and seismologic data for engineering purposes is illustrated by two examples.

● *See **Preface**, page v, for availability of publications marked with dot.*

● 3.2-20 Knudson, C. F. and Perez, V., **Accelerograph records from Lima, Peru,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 338–344.

Prior to 1972, Peru had only one accelerograph (C&GS) which was installed in downtown Lima during 1944. Although 22 earthquake records were obtained from this instrument between 1946 and 1972, only four of these produced significant records (records with peak accelerations greater than .05g). These four records were from the earthquakes of Jan. 31, 1951, Oct. 17, 1966, May 30, 1970, and Nov. 29, 1971. Lima experienced three destructive earthquakes in 1974 on Jan. 5, Oct. 3, and Nov. 9. Significant records from the C&GS accelerograph and a new SMA-1 accelerograph located in Lima were obtained from all three earthquakes. The seven earthquakes which produced these ten records were of Richter magnitude 5 to 7.7, and their distances from the recording stations ranged from 73 km to 370 km. Many of the spectra from these 10 records confirm the presence of predominant short-period components in the period range of 0.1 to 0.5 sec.

● 3.2-21 Bernreuter, D. L., **Estimates of the epicentral ground motion in the central and eastern United States,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 332–337.

Seismic ground motion from earthquakes in the central U.S. recorded at regional distances is used as a basis to determine estimates of the strong ground motion that could be expected from future earthquakes in this same area. The method used to back extrapolate the data into the near field is shown to be valid by use of data from earthquakes and underground nuclear explosions. The results of this study indicate that the estimated peak accelerations in the epicentral region are similar to those observed elsewhere throughout the world. The differences between these results and the lower values predicted by Nuttli are also discussed.

● 3.2-22 Johnson, J. A., **Possible source dependence of ground motion along a pipeline,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 383–388.

A spectral model incorporating source, travel path, and local site characteristics is used to simulate ground motion along a pipeline. Emphasis is placed on the possible effects of the source on the spectral character of ground motion. Preliminary results indicate that the overall shape of the predicted spectra can differ as a function of the corner frequency of the source function. Variation in corner frequency may in part explain observed variability in spectral shape and location of spectral peaks for (1) several events recorded at the same site, and (2) the same event recorded at several locations of similar geologic conditions.

● 3.2-23 Campbell, K. W., **Design earthquakes based on the statistics of source, path and site effects,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 378–382.

A method is proposed in which strong earthquake motion and its uncertainty can be predicted from statistical measures of source and propagation characteristics of a region. Three characteristics are used to represent earthquake strong motion: the shape of the power spectral density function, an energy-related intensity parameter, and two time domain shaping factors. The risk associated with the estimation of these parameters can be computed from probability and statistical models. Design earthquake motion in the form of accelerograms and response spectra can easily be developed from these parameters for any desired level of risk.

● 3.2-24 Westermo, B. D. and Trifunac, M. D., **Recent developments in the analysis of the duration of strong earthquake ground motion,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 365–371.

Duration of strong earthquake ground motion, defined in terms of the mean-square integrals of recorded motion, has been described empirically in terms of measured as well as qualitative parameters characterizing the earthquake source, the wave transmission path, the recording site conditions, and the overall level of shaking at a point. The parameters considered include: (1) earthquake magnitude, (2) epicentral distance, (3) geology surrounding the recording station, (4) the reported modified Mercalli intensity at the recording station, and (5) the frequency content of recorded motions.

● 3.2-25 Hudson, D. E., **Strong motion earthquake measurements in epicentral regions,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 323–329.

Only a few dozen of the more than 1000 strong-motion accelerograms obtained throughout the world since the early 1930s have been recorded at distances from points of seismic energy release as small as the dimensions of the earthquake source. Uniformly processed data for such near-field measurements are summarized and references on geological and seismological features of the events are given. The author concludes that an examination of all references for the earthquakes for which near-field accelerograms exist reveals a lack of detailed information. Only for the San Fernando earthquake, and perhaps to a lesser extent for the El Centro and the Parkfield events, are there enough data to make it possible to carry out meaningful

● *See **Preface**, page v, for availability of publications marked with dot.*

source mechanism studies. It is held that this conclusion emphasizes the tentative nature of much of strong-motion earthquake knowledge.

● 3.2-26 Joannon, J. G., Arias, A. and Saragoni, G. R., **The time variation of the predominant frequency of earthquake motions,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 560-568.

A new method to estimate the time variation of the predominant frequency of earthquake acceleration and velocity records is presented. The method uses the ratios between the mean-square functions of velocity, acceleration, and the derivative of acceleration. It has been applied to 32 earthquake accelerograms from the U.S.A., Mexico, Peru, and Chile. The determined functions for the predominant frequency of acceleration records are in agreement with functions estimated by other authors assuming a nonhomogeneous Poisson process of exponential decay trend for the zero crossings. This time variation is characterized by a chi-square decay trend which is particularly significant in some 1971 San Fernando earthquake records.

● 3.2-27 Sato, N., Kubo, K. and Katayama, T., **Characteristics of earthquake displacement motions with emphasis on their long-period components,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 569-574.

The characteristics of velocity and displacement motions derived from the accelerograms recorded at four sites during the main shock and the largest aftershock of the 1968 Tokachi-oki earthquake are discussed. The effects of earthquake magnitude and the depth to the seismic bedrock are examined on the peak and the rms amplitudes of velocity and displacement. The properties of displacement motions in the frequency domain are analyzed by using the output from a digital bandpass filter.

● 3.2-28 Tsuchida, H., Kurata, E. and Hayashi, S., **Observation of earthquake response of ground with horizontal and vertical seismometer arrays,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 509-515.

A horizontal seismometer array having six observation points along a straight line of 2500 m length has been established. Downhole seismometer arrays also have been established at two points. Records of three earthquake events are selected and analyzed, while 32 earthquake events had been recorded as of June 1976. Correlations among the ground motions at the observation points are studied. The relative displacements among the observation points on the ground surface are also studied, and stresses in a buried pipeline due to the relative displacements are estimated.

● 3.2-29 Liang, G. C. and Duke, C. M., **Separation of body and surface waves in strong ground motion records,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 553-559.

A method to separate body and surface waves in strong-motion accelerograms is presented. This method incorporates a linear system model which accounts for the behavior of body and surface waves. To demonstrate the method, Fourier spectra of the main shock records from the 1971 San Fernando earthquake are used. Spurious peaks occurred in many calculations because of divisions of Fourier spectra. A multistation scheme, which eliminates the spurious peak problem, is presented.

● 3.2-30 Toki, K., **Disintegration of accelerograms into surface and body waves,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 547-552.

The phase velocity dispersion curves for the frequency range from 0.2 to 2 Hz are detected from strong-motion earthquake records which were obtained during the San Fernando earthquake of Feb. 9, 1971. The resulting dispersion curves indicate that the phase velocity is greatly dependent on frequency, especially in the frequency range from 0.3 to 1 Hz. Making use of the dispersion curves, time traces of strain at the ground surface are computed for several kinds of strain components, attributed to surface and body waves. The maximum strain amplitude incorporated with surface waves is comparable to strain amplitude which is estimated under the assumption of a vertically incident SH wave in a near-surface ground.

3.2-31 Savarenskiy, Ye. F., Kosarev, G. L. and Sadikov, F. S., **Reconstruction of the true ground movement from an earthquake recording,** *Physics of the Solid Earth,* 13, *1,* 1977, 49-53.

An attempt is made to improve the spectral method of reconstruction of the actual ground movement from an earthquake recording. For this purpose, an algorithm is suggested which provides a choice of optimal integration limits with respect to frequency and an optional form of the low-frequency part of the spectrum of the actual ground movement. Examples of calculations are given.

● 3.2-32 Trifunac, M. D., **Statistical analysis of the computed response of structural response recorders (SRR) for accelerograms recorded in the United States of America,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2956-2961.

● *See* ***Preface****, page v, for availability of publications marked with dot.*

Since only a few strong-motion accelerograms have been recorded in India so far, there are not enough data at this time to compare characteristics of strong earthquakes there with related experience in the U.S., Japan, and elsewhere. Such comparisons would enable one not only to understand better the nature of strong earthquake ground motion in India, but would also provide a basis for the transfer and use of strong-motion data which have been recorded in other seismic regions for earthquake-resistant design on the Indian continent. As a first step in the direction of establishing some basis for such correlations in the future, the analysis of the computed response of structural response recorders (SRR) is presented for excitations consisting of 186 digitized accelerograms which have been recorded in the western U.S. during the period from 1933 to 1971. This is analogous to having 186 actual records of SRR for earthquakes recorded in the U.S.

● 3.2-33 Amin, M., Moinfar, A. A. and Mehrain, B., **Spectral characteristics of earthquake records registered in southern Iran,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 878–883.

Calculated spectra which depart from the corresponding design spectra obtained from an amplification factor approach are compared with records registered in southern Iran and similar records registered in the western U.S. The similarity of records is established on the basis of the shape of the ground velocity diagram. Comparisons indicate that the use of amplification factors obtained from an extensive study of the records of one region, to establish design spectra for another region, can be associated with a maximum error of 25% on design forces in the period range of interest.

● 3.2-34 Shoja-Taheri, J., **Seismological studies of strong motion records,** *UCB/EERC-77/04,* Earthquake Engineering Research Center, Univ. of California, Berkeley, Jan. 1977, 209. (NTIS Accession No. PB 269 655)

A number of problems pertinent to seismological and engineering interpretations of strong ground motions in earthquakes are studied. The main new results are as follows: (1) A new form of strong-motion accelerogram (spectrally maximized records or SMR) and its associated generalized spectrum are proposed for earthquake engineering use. Parameters (e.g., spectral, duration, peak amplitude) of horizontal-component strong-motion records at a given site generally depend significantly on the (arbitrary) azimuthal direction, often resulting in a crucially deficient description of these parameters if only a single component is used. Combination of horizontal components using spectral maximization is shown to be effective in minimizing the difficulty. The spectra of the two horizontal components at each site are combined to maximize the resultant spectrum independently of azimuthal orientation. SMRs of thirty-three important strong-motion accelerograms (including twelve New Guinea records) are then calculated from their corresponding spectra. In only 60% of cases is the peak acceleration from a maximized spectrum greater than that of the single components; the bracketed duration from the maximized spectrum is always greater. After filtering to provide records in ten frequency bands (0-1 H2, 1-2 H2, . . . 9-10 H2), correlations for each band are made between acceleration peaks, spectral energy, magnitudes, and source distances. More stable estimates of these strong-motion parameters appear to be provided by spectrally maximized accelerations compared with single-component estimates.

(2) Statistical analysis of all accelerograms of the 1966 Parkfield, California earthquake and the 1952 Taft, California earthquake indicates that the usable long period of ground displacements obtained from double integration of accelerogram records are limited by two major sources of errors: human reading and baseline corrections. Usable long-period limits are estimated to vary between 7 to 14 sec, depending on the individual earthquake.

(3) It is shown that the integration of ragged functions such as strong-motion accelerograms by regular quadrature formulas leads to significant errors. The conventional method of frequency domain integration also leads to indeterminacy of zero frequency information and distortion of the shape of the resulting integral. A modification of the conventional method of frequency domain integration was developed to avoid these deficiencies. The new technique extrapolates the integrand by joining its mirror image to the end of reflection. The zero frequency information is also determined by this technique.

(4) A detailed seismological interpretation of the strong-motion records was attempted for the 1966 Parkfield earthquake. Velocity and displacement traces integrated from the corresponding recorded accelerograms were found most valuable in studying the earthquake mechanism and wave forms. A double-couple right-lateral strike-slip mechanism (along the San Andreas fault) is consistent with the recorded direct S waves originating from the hypocenter. High energy arrivals observed on the velocity traces are interpreted as S waves ("stopping phases") that originated at the termination of the rupture towards the southeast of the San Andreas fault.

From particle velocity diagrams of the stopping phases in the horizontal plane, the rupture length was between 20 to 28 km. Corresponding rupture velocities are estimated to be 2.5 ± 0.1 km/sec and 3.1 ± 0.5 km/sec. The inference from the strong-motion records is that Love waves were more excited at the southwestern than the northeastern side of the fault, whereas the Rayleigh waves were more energetic at the northeastern than the southeastern side of the fault. The late arrivals of both Love and

Rayleigh waves of long periods (about 6 sec), together with the observed reversed dispersion of Rayleigh waves observed at the temblor station, all indicate a low velocity zone within the crust.

● 3.2-35 **Seismic engineering data report: strong-motion earthquake accelerograms, digitization and analysis,** *Open-File Report 76-609,* U.S. Geological Survey, Menlo Park, California, July 1976, 124.

This is the first of a series of reports planned to include the results of digitization and routine analyses of strong-motion earthquake accelerograms published by the U.S. Geological Survey. Serving as a model for this effort is the collection of data reports published by the Earthquake Engineering Research Lab. of the California Inst. of Technology during the years 1969-1975, which covers the significant records from 1933 to the San Fernando earthquake of Feb. 9, 1971. This report includes the significant records of 1971 subsequent to the San Fernando earthquake. The following records are included: (1) Isabella Dam, California (auxiliary dam abutment), Mar. 8, 1971; (2) Adak, Alaska, U.S. Naval Base (seismic vault), May 1, 1971; (3) Santiago, Chile (Univ. of Chile), July 9, 1971; (4) Ferndale City Hall, California (ground level pier); Sept. 12, 1971; (5) Lima, Peru (Instituto Geofisico), Nov. 29, 1971. Presented are summaries of records, graphs, and charts.

● 3.2-36 Morris, L., Smookler, S. and Glover, D., **Catalog of seismograms and strong-motion records,** *SE-6,* World Data Center A for Solid Earth Geophysics, U.S. National Oceanic and Atmospheric Admin., Boulder, Colorado, May 1977, 76.

The catalog lists the World Data Center A holdings of seismograms and strong-motion records, provides price lists, and describes formats in which the records are available. The guidelines for collecting these data are presented.

3.2-37 Bahar, L. Y., **Optimal digitization of earthquake records,** *Nuclear Engineering and Design,* 44, 2, Nov. 1977, 263-267.

The optimal time interval for sampling an earthquake time history in order to recover the original record in a unique manner is derived. If the sampling time interval is too large, the high-frequency components of the time history can exemplify low-frequency components. This results in aliasing, which is a major source of distortion in the reconstruction of the original time history from its discretized values. In the present study, the optimal sampling time interval known as the Nyquist interval in signal processing is derived. The results indicate that if the optimal time interval is exceeded loss of information occurs. The use of a smaller time interval results in additional computer cost without any increase in the information necessary to reconstruct the original time history.

● 3.2-38 **Analysis of strong motion records of the Vrancea, Romania earthquake of March 4, 1977, obtained in Nis, Yugoslavia,** *55,* Inst. of Earthquake Engineering and Engineering Seismology, Univ. Kiril and Metodij, Skopje, Yugoslavia, May 1977, 114.

Results of analysis of the records of the Vrancea-Romania earthquake of Mar. 4, 1977, 21 h 22 min, recorded at Nis, Yugoslavia, are presented. One record of an SMA-1 accelerograph and three records of WM-1 seismoscopes are examined. By analysis of the accelerograph records uncorrected data regarding acceleration, response spectra, and Fourier's amplitude spectrum were obtained; by analysis of the seismoscope records, the spectral displacement and pseudo-relative velocity values for 10% of the critical damping and for the natural free vibration period of the seismoscopes were obtained. Of special interest is the comparison of the results obtained by seismoscope No. 1126 and the calculated spectral values of the acceleration time history, obtained by the accelerograph for the E-W and N-S directions, since they relate to the same site. There was a difference of about 10% in the results.

● 3.2-39 **Seismic engineering data report: strong-motion earthquake accelerograms—digitization and analysis—records from Lima, Peru: 1951 to 1974,** *Open-File Report 77-587,* U.S. Geological Survey, Menlo Park, California, Apr. 1977, 164.

This report is the second of a series of reports planned to include the results of digitization and routine analyses of strong-motion earthquake accelerograms published by the U.S. Geological Survey. This report contains plots of the data reduction and spectral analysis of ten selected strong-motion earthquake accelerograms recorded in five different stations, generated by seven chosen earthquakes that occurred in Peru during the period Jan. 31, 1951, through Nov. 9, 1974. Information on the earthquakes, stations, and records are summarized. Plots of uncorrected accelerograms; corrected acceleration, velocity, and displacement; spectra for velocity response, pseudovelocity response, and Fourier amplitude; and duration of velocity response envelope, for each of the components of the records studied, are presented in the set of figures in this report.

● 3.2-40 **Digital data of the strong-motion earthquake accelerograms in Matsushiro earthquake swarm area** (in Japanese and English), *1,* Strong Earthquake Motion Observation Center, Earthquake Research Inst., Univ. of Tokyo, Japan, Apr. 1976, 161.

● *See **Preface**, page v, for availability of publications marked with dot.*

The earthquake swarm in the Matsushiro area, approximately 10 km south of Nagano City, began in August 1965 and almost ceased at the end of 1970. The total number of felt earthquakes during this period amounted to 62,821 of which 50 and 9 were the large shocks, with intensities of IV and V on the JMA scale, respectively. According to the statistics of damage, the numbers of totally, seriously, and partially destroyed houses were 10, 4 and 8,750, respectively. Sixteen persons were injured. The Strong Earthquake Motion Observation Center of the Earthquake Research Inst. carried out temporary observations of strong earthquake motions in the area from September 1965 to August 1968, while the earthquake activity was high. A total of 941 accelerograms from 537 earthquakes were obtained at 13 observation stations.

The significant strong-motion accelerograms from the Matsushiro swarm have been digitized for research purposes, and the digital values for 22 accelerograms (66 components) are presented in this report. The plots of corrected accelerograms and the computed ground velocities and displacements are included. The strong-motion records collected during the Matsushiro earthquake swarm reflect some specific features on near-field earthquakes with a comparatively small magnitude and shallow focal depth.

● 3.2-41 **Interpretation of strong motion records for earthquake design criteria and structural response,** Earthquake Engineering Research Inst., Oakland, 1976, 300.

The publication comprises a collection of graphs, charts, tables, and diagrams from a series of lectures cosponsored by the Earthquake Engineering Research Inst. and the Structural Engineers Assn. of California between April 6 and June 2, 1976. The titles of the topics discussed and the authors' names are as follows; none of these is abstracted in this volume of the *AJEE*: *Introduction to the dynamics of earthquake-resistant systems,* Chopra, A. K.—*Strong motion instruments: history of their development and their operating characteristics,* Benuska, K. L.—*Planning of instrument stations and networks–network operation and data acquisition,* Matthiesen, R. B.—*Reading of records–data processing for ground motions and structural response,* Hudson, D. E.—*How do accelerograph records help us to understand and predict soil behavior?,* Seed, H. B.—*What can the accelerograph record tell us about structural behavior?,* Jennings, P. C.—*How do accelerograph records help us to predict structural behavior?,* Penzien, J.—*Correlation of accelerograph records with damage, and general guidelines for use of strong motion data in earthquake-resistant design,* Housner, G. W.

3.2-42 Oda, H. *et al.*, **S wave attenuation in the vicinity of ground surface** (in Japanese), *Zisin, Journal of the Seismological Society of Japan,* 30, 3, Dec. 1977, 275–282.

The quality factor, *Qs*, of attenuation for shear waves in the vicinity of the ground surface is estimated by spectral analysis of body waves from microearthquakes observed at Mitsumata in Akita prefecture. The analysis is made for thirty earthquakes in which S-P times are between 1 and 2 sec. Based on some reasonable assumptions, it is found that the value of *Qs* can be easily estimated from the average slope of the spectral amplitude ratio between the direct P or S wave and the S or P wave converted at the boundary of the surface layer and the substratum. The result indicates that *Qs* in the surface layer takes a value from 5 to 20 in the frequency range between 6 and 30 Hz.

3.3 Artificial and Simulated Earthquake Records

3.3-1 Levy, S. and Wilkinson, J. P. D., **Generation of artificial time-histories, rich in all frequencies, from given response spectra,** *Nuclear Engineering and Design,* 38, 2, Aug. 1976, 241–251. (For an additional source, see Abstract No. 1.2-2.)

One objective of this paper is to present a method of synthesizing such time histories from a given design response spectrum. The design response spectra may be descriptive of floor responses at a particular location in a plant, or they may be descriptive of seismic ground motions at a plant site.

The method described allows the generation of time histories that are rich in all frequencies in the spectrum. This richness is achieved by choosing a large number of closely spaced frequency points so that the half-power points of adjacent frequencies overlap. Examples are given concerning seismic design response spectra, and a number of points are discussed concerning the effect of frequency spacing on convergence.

In generating time-history representations of seismic ground motions, two points demand particular attention. First, the time history should be recognizable as a realistic earthquake motion which is achieved in the present approach by the introduction of a modulating envelope. The second point arises when it is desired to excite the plant structural model with simultaneous seismic excitation components in two perpendicular horizontal directions and in the vertical direction. Usually the specified vertical spectrum differs from the horizontal spectrum, so that the corresponding time histories differ as well. However, it is desirable in any event to generate two horizontal time histories, which should be as unrelated as are earthquake records themselves. Statistically, one desires that their correlation coefficients should be low. In the second section of this paper, a method is suggested for generating a second time history from the first time history so that they have

● *See* ***Preface,*** *page v, for availability of publications marked with dot.*

low correlation, while at the same time they have response spectra that approximate to a satisfactory degree the original design response spectrum. Finally, a number of alternative schemes of dual time-history generation are inspected, and the resulting correlation functions and spectra are discussed.

● 3.3-2 Vanmarcke, E. H. and Gasparini, D. A., **Simulated earthquake ground motions,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(a), Paper K 1/9, 12. (For a full bibliographic citation, see Abstract No. 1.2-5.)

The paper reviews current methods for generating synthetic earthquake ground motions. Emphasis is on the special requirements demanded of procedures to generate motions for use in nuclear power plant seismic response analysis. Specifically, very close agreement is usually sought between the response spectra of the simulated motions and prescribed, smooth design response spectra. The features and capabilities of the computer program SIMQKE, which has been widely used in power plant seismic work, are described. Problems and pitfalls associated with the use of synthetic ground motions in seismic safety assessment are also pointed out.

● 3.3-3 Lin, C.-W., **Criteria for the generation of spectra consistent time histories,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(a), Paper K 1/11, 8. (For a full bibliographic citation, see Abstract No. 1.2-5.)

Several methods are available to conduct seismic analysis for nuclear power plant systems and components. Among them, the response spectrum technique has been most widely adopted for linear modal analysis. However, for designs which consist of structural or material nonlinearities such as frequency-dependent soil properties, the existence of gaps, single acting tie rods, and frictions between supports where the response has to be computed as a function of time, the time history approach is the only viable method of analysis. Two examples of time history analysis are (1) soil-structure interaction study, and (2) a coupled reactor coolant system and building analysis to either generate the floor response spectra or compute nonlinear system time history response. The generation of a suitable time history input for the analysis has been discussed in the literature. Some general guidelines are available to insure that the time history input will be as conservative as the design response spectra. Very little has been reported as to the effect of the dynamic characteristics of the time history input upon the system response. In fact, the only available discussion in this respect concerns only the statistically independent nature of the time history components.

There are several approaches currently being used by the nuclear industry to generate a design time history input. None of these produce unique results; that is, given a design response spectrum, a nearly unlimited number of synthesized time history motions can be constructed. The effects of these time history motions on the system response vary and they have not been properly evaluated. For instance, some time histories may have a high frequency content, higher than indicated by the real earthquake records. This may have an adverse influence on the system response with high frequency impact or predominately high frequency modes. Other time histories may have an unnecessarily long duration which makes a large and detailed analytical model uneconomical. The influence of the time history duration is primarily on the number of peak response stress cycles computed which can be either extrapolated from limited duration input or determined by other means. Rarely is it the case that duration has to be kept long enough for the structural response to reach its peak. Consequently, input duration should be kept no longer than necessary to produce peak response to allow the use of a more sophisticated model which enables the problem to be studied thoroughly. There are also time histories which have satisfied the generally accepted definition of statistical independent requirements but possess statistical characteristics unlike those of the real earthquakes. Finally, some time histories may require smaller integration time steps than ordinarily used to insure that certain systems will have convergent and stable solutions.

In this paper, numerical results for cases discussed above are presented. Criteria are also established which may be advantageously used to arrive at spectra-consistent time histories which are conservative and, more importantly, realistic.

● 3.3-4 King, A. C. Y. and Chen, C., **Interactive artificial earthquake generation,** *Computers & Structures,* 7, 4, Aug. 1977, 503-506.

A method for generating artificial time histories is described. Because iteration procedures are applied to generate the artificial time history with desired spectra, an interactive graphics system is incorporated into the method. Advantages of the method include: (1) The frequency content of the time history is under good control. (2) The inherent nature of the method will make the correlations among three-component earthquakes at one site simulate closely the actual recorded time histories. (3) A single time history can be generated to match spectra of different damping values. The artificial time history matches only the desired response spectrum of single degree-of-freedom oscillators.

● 3.3-5 Heaton, T. H. and Helmberger, D. V., **A study of the strong ground motion of the Borrego Mountain,**

California, earthquake, *Bulletin of the Seismological Society of America,* **67,** *2,* Apr. 1977, 315–330.

Several synthetic models are constructed to fit the first 40 sec of the transversely polarized displacement, as recorded at El Centro, of the Apr. 9, 1968, Borrego Mountain earthquake. The modeling is done in the time domain using the response computed for a distributed set of point shear dislocations embedded in a layered halfspace. The beginning 10 sec of the observed record is used to model the spatial and temporal distribution of faulting, whereas the remaining portion is used to determine the upper crustal structure based on surface wave periodicity. A natural-depth criterion was provided by comparing the amplitude of the direct arrival with the surface wave excitations. Tradeoffs are found to exist between source models and velocity structure models. Within the framework of a layer over a halfspace model, faulting of finite vertical extent is required, whereas the horizontal dimensions of faulting are not resolvable. A model which is also consistent with the teleseismic results of Burdick and Mellman indicates massive faulting near a depth of 9 km with a fast rise time producing a 10-cm displacement pulse of 1 sec duration at El Centro. The faulting appears to slow down approaching the surface. The moment is calculated to be approximately 7×10^{25} dyne-cm, which is somewhat smaller than the moment found by Burdick and Mellman (1976).

● 3.3-6 Kubo, T. and Penzien, J., **Characteristics of three-dimensional ground motions along principal axes, San Fernando earthquake,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 439–444.

Using the concept of an orthogonal set of principal axes and applying the time and the frequency domain moving-window technique to the accelerograms recorded during the San Fernando earthquake of Feb. 9, 1971, characteristics of three-dimensional ground motions along principal axes are determined. It is concluded from the resulting intensity functions and the time-dependent frequency contents that three realistic components of ground motion can be generated stochastically as nonstationary random processes along their corresponding principal axes without cross correlation with one another in a statistical sense.

● 3.3-7 Higgins, C. J. and Triandafilidis, G. E., **The simulation of earthquake ground motions with high explosives,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 489–495.

Large explosive detonations appear to be the best available means of simulating the effect of earthquake ground motions on soil and soil-structure systems. Sufficient data are available to indicate that reasonable simulation is possible using explosive arrays alone or in combination with enhancement techniques. Example data are presented and a method for expanding the explosive data base using finite-difference wave propagation calculations is described.

● 3.3-8 Minami, T. *et al.*, **Non-stationarities observed in strong motion accelerograms and their effects on earthquake response of structures,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 372–377.

Simulated accelerograms are generated to study the effects of nonstationary features appearing in strong-motion earthquake records to the nonlinear response of structures. Response displacements in simple structures during strong-motion earthquakes are calculated for three different restoring force models. In some cases, considerable discrepancies have been observed in the response spectra for different types of accelerograms which possess similar Fourier spectra but different patterns of amplitude distribution. Significant effects of nonstationarities, however, have not been recognized for relatively stable hysteretic models.

● 3.3-9 Balan, S. *et al.*, **A model for simulating ground motion,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 523–528.

Ground motion characteristics are first discussed from a qualitative viewpoint. Simulation techniques are commented on briefly. A model for simulation of scalar or vectorial nonstationary motions is then described. The different wave trains (P-, S-, etc.) are considered with attention given to the influence of geologic conditions. The algorithms and flow charts used for programming are presented. Illustrative examples for applying the programs are given, and the results obtained are discussed from the standpoint of simulation techniques and features of actual ground motions.

● 3.3-10 King, A. C. Y. and Chen, C., **Artificial earthquake generation for nuclear power plant design,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2609–2613.

This paper describes a detailed method for generating a time history which is consistent with a specified design spectra. Several advantages of this method are described. The frequency content of the time history is well under control. If one wishes to generate the three components of earthquakes at one site, the inherent nature of this method will make the correlations among these three components simulate closely the actual recorded time histories. A single time history can be generated to match a spectra for different damping values.

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● 3.3-11 Mulay, J. M. and Ramesh, C. K., **A new base-line correction for blast-records,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 911-912.

3.4 Seismic Zoning

● 3.4-1 Kulhawy, F. H. and Ninyo, A., **Earthquakes and earthquake zoning in New York State,** *Bulletin of the Association of Engineering Geologists,* XIV, 2, Spring 1977, 69-87.

A review is presented of the seismic history, proposed seismic zoning, and the frequency of seismic events in New York State. It is shown that earthquakes are a very real and important design factor in New York. The data show that the St. Lawrence, Buffalo-Attica, and New York metropolitan regions have been subjected to the highest intensity earthquakes and the greatest frequency of earthquake occurrence, but that all the state has felt large distant earthquakes as well. Design implications are briefly noted.

● 3.4-2 Cluff, L. S., Weaver, K. D. and Niccum, M. R., **Zoning for surface fault rupture: Managua, Nicaragua,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 817-822.

Damage accompanying the Managua, Nicaragua, earthquake of Dec. 23, 1972, demonstrated that surface fault rupture is a significant hazard in Managua and must be considered in seismic zoning. This paper describes the approach to the development of a fault hazard zone map and a planning matrix used to reduce the impact of surface faulting in Managua. The study was performed by a multidisciplinary team including Nicaraguan government architects and planners, and private geotechnical consultants. The method can be adapted and applied to other areas where surface faulting is expected to occur.

● 3.4-3 Udwadia, F. E. and Trifunac, M. D., **Problems in the constructions of microzoning maps,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 735-741.

Problems arising in the construction of microzonation maps have been studied by evaluating the current techniques used for their preparation. Two commonly used methods, the impedance technique and the microtremor technique, have been critically assessed. The assumptions in each method and their weaknesses are pointed out. An index for the preparation of microzoning maps has been proposed, and a general methodology for microzonation has been suggested.

● 3.4-4 Basu, S. and Nigam, N. C., **Seismic risk analysis of Indian Peninsula,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 782-790.

A statistical analysis of the seismic risk of the Indian Peninsula bounded by latitude 6°,40′ and longitude 66°,98′ is made to determine the design acceleration, velocity, and displacement for a 100-year return period. The analysis is based on available seismic data. The focal depth is assumed to be independent of time. Two distributions, uniform and log-normal, truncated at 600 km depth, are considered. The spatial properties of the earthquakes are considered homogeneous in tectonic features. Seismic zoning maps based on this study are compared with the existing map.

● 3.4-5 Alonso, J. L. and Larotta, J., **Seismic risk and seismic zoning of the Caracas Valley,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 776-781.

The main objective of this work is to provide appropriate tools to minimize the seismic risk for structures to be designed in Caracas. The seismic hazard was evaluated taking into account: (1) the seismic history based on felt reports (historical) and available instrumental data, (2) known active faults within 100 km radius, (3) geological hazards (landslides), and (4) soil conditions. Results of this investigation include: (1) a map of the north-central part of Venezuela showing active faults and the location of main historic and instrumental epicenters, (2) maps of the valley and surrounding areas containing curves of equal peak rock accelerations and velocities expected for return periods of 25 and 75 years, (3) graphs to anticipate the shapes of acceleration response spectra taking into account the fundamental period of soil deposits and the peak ground acceleration for stiff rock and deep cohesionless soil deposits (£ = 5%), (4) a map of the city showing depth of soil and hazard zones, and (5) normalized acceleration response spectra for a return period of 25 years.

● 3.4-6 Shah, H. C. *et al.*, **A seismic risk contour map for Nicaragua,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 770-775.

An iso-contour risk map is developed for Nicaragua based on previously developed probabilistic seismic risk maps for different future time periods and risk levels. The chart accompanying the iso-contour map provides the peak ground acceleration value for the desired return period for design. Peak ground acceleration values for locations other than at the contour lines are obtained by interpolation between contours. Improved acceleration zone graphs for major cities in the country are also included in this paper.

● 3.4-7 Mallick, D. V. and Morghem, F. T., **Earthquake zoning of Libya,** *Proceedings, Sixth World Conference on*

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Earthquake Engineering, Sarita Prakashan, Meerut, India, Vol. I, 1977, 907–908.

● 3.4-8 Kulshreshtha, V. K. and Singh, V. N., **Microzonation of the north-eastern part of the Indian subcontinent,** *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. I, 1977, 901–902.

● 3.4-9 Grandori, E. and Grandori, G., **An application of decision theory in seismic zoning,** *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. I, 1977, 843–849.

Seismic zoning is optimization of the distribution of funds that a community devotes to the prevention of seismic risk. The optimum solution can be found through the repeated calculation of the optimum distribution over two sites. Special control of the model used for this calculation is carried out on the basis of decision theory, taking into account the uncertainties in the quantitative definition of the hypotheses.

● 3.4-10 Bune, V. I., **The new elements on the map of seismic zoning,** *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. I, 1977, 850–858.

The zones of probable generation of earthquake sources differentiated according to their depths and M_{max}, as well as quantitative assessments of the probability of shaking, are new elements on a map of seismic zoning. An example of the map with the new elements is presented.

3.5 Influence of Geology and Soils on Ground Motion

3.5-1 Esenov, E. M., **Seismic microzoning of towns in western Turkmen on the basis of instrumental investigations** (Seismicheskoe microraionirovanie gorodov zapadunoi Turkmenii na osnove instrumentalnykh issledovanii, in Russian), Ilim, Ashkhabad, 1976, 195.

In this monograph results of investigations into soil parameter characteristics are presented. Prediction of the effects of soil parameters on the behavior of structures is studied. The methods used in seismic microzoning of the towns of Nebit-Dag, Krasnovodsk and Cheleken are described.

● 3.5-2 Kausel, E., Christian, J. T. and La Plante, F. J., **An improved algorithm for non-linear soil amplification,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology*, Vol. K(a), Paper K 1/14, 9. (For a full bibliographic citation, see Abstract No. 1.2-5.)

The most widely used techniques for the study of one-dimensional amplification of vertically propagating seismic waves involve solution of the dynamic equations in the frequency domain with linearly viscoelastic material properties. Material nonlinearities are approximately modeled by iterative use of linear solutions, adjusting values of modulus and damping until they are compatible with computed levels of strain. The program SHAKE is the best known example of a code using this procedure. While the algorithm works well for many cases, it can diverge when a large amplitude of motion is specified at, or near, the surface of a deep stratum of soft soil. The failure of the iterations to converge is the result of requiring a higher level of motion to be propagated through the soil than can be accommodated. The divergence always occurs below the level at which the control motion is specified.

The improved algorithm described is based on the fact that the solution for a given layer below the level of the control motion depends only on the properties of the layer and on the solution for the overlying layers. The procedure consists of: (1) solving in the standard iterative linear fashion for all materials above the control motion, if there are any; (2) without further iterations in the overlying material, deconvolving for one layer below the control motion, with iterations on the soil properties in that layer; (3) repeating the procedure of step (2) for each successively lower layer in the profile. Thus, one does not proceed to the next layer until all overlying layers have converged. Divergence can occur in the revised algorithm as it does with the currently used techniques. However, it is controlled to a specific layer, and the solutions for the overlying layers have already been obtained before any lack of convergence occurs. The improved algorithm thus avoids repeated computer runs to identify the convergent depth; it is substantially faster than the standard method because it avoids unnecessary calculations for layers that have already converged; and it insures that all layers converge within the same tolerance.

● 3.5-3 Trifunac, M. D. and Westermo, B., **A note on the correlation of frequency-dependent duration of strong earthquake ground motion with the modified Mercalli intensity and the geologic conditions at the recording stations,** *Bulletin of the Seismological Society of America*, 67, 3, June 1977, 917–927.

The frequency-dependent duration of strong earthquake ground motion, based on the mean-square integrals of motion, has been correlated with the reported modified Mercalli intensity at the recording site. Simple relations have been presented which describe the overall trends of computed durations for different levels of modified Mercalli intensity and for three classes of site geology.

● 3.5-4 Wong, H. L., Trifunac, M. D. and Westermo, B., **Effects of surface and subsurface irregularities on the**

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amplitudes of monochromatic waves, *Bulletin of the Seismological Society of America,* **67,** *2,* Apr. 1977, 353–368.

Measurements of surface ground motion generated by forced vibration of a nine-story reinforced concrete building at a distance of 2 to 5.5 km are described. Three components of the displacement field were measured at 13 points along a line traversing an elongated canyon underlain by a shallow and dipping alluvial layer. The variations of measured displacement amplitudes have been modeled by (a) a two-dimensional surface topographic feature corresponding to the average cross section of the canyon, and (b) by a two-dimensional model of an alluvium valley excited by a line source. Comparison of the observed and computed amplitude variations with distance suggests that, for the geometry corresponding to this experiment, the effect of the dipping layer of alluvium seems to play a considerably more important role than the canyon.

● **3.5-5** Singh, S. K. and Sabina, F. J., **Ground-motion amplification by topographic depressions for incident *P* wave under acoustic approximation,** *Bulletin of the Seismological Society of America,* **67,** *2,* Apr. 1977, 345–352.

Ground-motion amplification caused by a semispherical and a semicylindrical cavity in a halfspace for an incident *P* wave is calculated from analytic solutions under acoustic approximation. The authors expect this approximation to be valid for materials with a Poisson ratio near 0.5, e.g., saturated soils. The ground motion is zero at the edges of the cavity and is normal to the free surface. An amplification of about 2 to 3 occurs near the bottom of the cavity for large wavelengths. Detailed numerical results are given for vertical incidence.

● **3.5-6** Arnold, P. and Vanmarcke, E. H., **Ground motion spectral content: the influence of local soil conditions and site azimuth,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 445–451.

The intensity and frequency of ground motions recorded during the 1971 San Fernando earthquake are studied. The estimated power spectral density function is used as the primary tool for studying ground motion characteristics, but comparisons using peak values to parameterize ground motions are also briefly reviewed. Observations are made on the variations in ground motion characteristics, followed by a tentative examination of the possible causes of these variations. In particular, the distance and direction from the source (azimuth) and the characteristics of local soil conditions are identified as factors which appear to exert an important influence on the characteristics of ground motions.

● **3.5-7** Singh, S. and Donovan, N. C., **Seismic response of frozen-thawed soil systems,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2262–2267.

Whereas considerable research has been devoted to measurement of the dynamic properties of frozen soils, little emphasis has been placed on possible effects of a frozen soil layer when a soil profile is subjected to earthquake motions. In this paper, some estimates of the basic dynamic performance of frozen soils in earthquakes are made utilizing parametric studies. These show that a shallow frozen layer can significantly affect the anticipated motion at the ground surface.

● **3.5-8** Boncheva, H., **Soil amplification factor of surface waves,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2334–2339.

The paper presents the investigation of the effect of a horizontally stratified deposit of soil layers on the amplification of surface waves. The amplification spectra are obtained using the wave method of solution, and they are given for different layer thicknesses and different rigidites of rock. The influence of the layer thickness on the phase velocity of surface waves is studied, and a critical wave number is determined. The comparison of the amplification spectra of surface waves and those of body waves is shown.

● **3.5-9** Wong, H. L. and Trifunac, M. D., **A note on the effects of recording site conditions on amplitudes of strong earthquake ground motion,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 452–457.

Studies of vibrational properties of irregular topography and of soil and alluvial deposits excited by strong earthquake ground motions are summarized. The geometry of these features can be irregular and may lead to complicated scattering, diffraction, and focusing patterns of motion for incident seismic waves.

● **3.5-10** Duke, C. M. *et al.*, **Effects of site on ground motion in the San Fernando earthquake,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 423–431.

The records of basement and free-field accelerograph stations that recorded the San Fernando, California, earthquake of Feb. 9, 1971, were compared with geological and geophysical site data, particularly shear wave velocity profiles. Geophysical surveys were made at 47 accelerograph sites to determine the velocity profiles to a depth of about 70 ft. Both peak particle velocity and Arias instrumental intensity were found to have statistically significant

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dependence upon the mean shear wave velocity and the rate at which it increased with depth.

● 3.5-11 Berrill, J. B., **Site effects during the San Fernando, California, earthquake,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 432–438.

Fourier amplitude spectra of 71 strong-motion accelerograms are examined in the 0.4 to 16 Hz frequency band for a possible relationship between intensity of ground motion and local geology. No significant difference was found between accelerations recorded on soft sediments, stiffer sediments and sedimentary rock, and basement rock; it is concluded that at sites with these soils, at least, no allowance should be made for local geologic conditions in estimates of future ground motion.

● 3.5-12 Borcherdt, R. D., **Wave propagation in soils,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 599–604.

The general theory of viscoelasticity suggests that the characteristics of S waves transmitted across a bedrock-soil or a soil-soil interface differ significantly from those that have been considered previously in the earthquake engineering literature. The general theory predicts that type-II S waves transmitted across such boundaries are, in general, inhomogeneous with velocities and attenuations that depend on the incident angle of the incoming wave, directions of maximum energy flow that differ from those of phase propagation, and energy flow and dissipation due to interaction of the waves. Current numerical models of soil response in general do not account for these physical characteristics of the waves. Numerical results derived for an alluvium-shale interface and a shale-granite interface suggest that these theoretically predicted characteristics are significant for problems concerned with the in-situ measurement of seismic amplitudes, but that they are probably not significant for seismic traveltimes measured using present technology.

● 3.5-13 Faccioli, E., **Probabilistic assessment of seismic risk on local soil sediments,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 584–591.

Probability distributions of earthquake parameters on local soil sediments are derived from corresponding distributions for rock or hard ground conditions. The method is primarily intended for cases where the soil response spectrum is dominated by site amplification effects. As an example, seismic design parameters for Mexico City lacustrine clay sediments are presented and compared with recorded data.

● 3.5-14 Kobayashi, H. and Nagahashi, S., **Response spectra on seismic bedrock during earthquake,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 516–522.

The response spectra of earthquake motions observed on the ground surface can be illustrated as the product of seismic bedrock motion, which is explained by earthquake source mechanisms and wave propagation, and by the effects of ground characteristics on the amplification of seismic waves. In this paper, the authors determined the ground characteristics and the seismic bedrock motions from the observed data of strong-motion accelerographs. Using the results of this study, the authors show the general nature of seismic bedrock motions, which relate to the magnitude of earthquakes and hypocentral distance, and propose an empirical formula for the attenuation of intensity on bedrock.

● 3.5-15 Kobayashi, H. *et al.,* **Two-dimensional horizontal ground motions during earthquake,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 345–350.

The ground motion during an earthquake is composed of body and surface waves, which are related to the geological conditions of layered soils. Each wave has its own direction of oscillation, depending upon the direction of wave propagation. The authors separated wave components from strong-motion records obtained during the Tokachi-oki earthquake of 1968 and the Izuhanto-oki earthquake of 1974. Using several methods of analysis, two-dimensional horizontal earthquake ground motions in a plane are described.

● 3.5-16 Arai, H., Kitajima, S. and Saito, S., **Underground earthquake motions in ports and harbours of Japan,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 502–508.

Earthquake motions in soil layers were observed using borehole seismometers at observation stations in five ports and harbors of Japan. The effects of the characteristics of earthquakes and the vibrational properties of soil layers on the ground motions are investigated by analysis of the Fourier amplitude spectra of the observed earthquake motions.

● 3.5-17 Mal, A. K. and Duke, C. M., **Transfer functions for surface waves,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 631–636.

An approximate technique is developed to calculate the surface wave transfer functions for propagation between two stations with different site conditions. The

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technique is applied to Love waves propagating in a two-dimensional single-layered model of the soil containing a discontinuous change in the layer thickness. The displacement spectra produced by sudden dislocation near the free surface at two stations located on either side of the transitional zone are calculated and compared. The influence of the higher modes is shown to be significant, especially at higher frequencies.

● **3.5-18** Shahinpoor, M., Tadjbakhsh, I. G. and Ahmadi, G., **Seismic response of hills**, *Iranian Journal of Science and Technology*, **6**, *4*, 1977, 199–203.

This paper presents a simple approach for the derivation of improved upper bounds for acceleration, velocity, and displacement seismic amplification factors for various locations on a hill, treating the hill as a nonhomogeneous, anisotropic, elastic body with linear damping. A sufficiently small height-base width ratio is assumed and seismic wavelengths larger than the base width of the hill are considered such that the problem reduces to uncoupled pure shear and normal excitations of a hill. An estimate employed in a previous paper by Mostaghel and Nowroozi is clarified and the results obtained therein are expanded to improve their upper bound on the amplification factor.

● **3.5-19** Shinozuka, M. and Kawakami, H., **Ground characteristics and free field strains**, *Technical Report CU-2*, Dept. of Civil Engineering, Columbia Univ., New York, Aug. 1977, 31.

A quasi-two-dimensional analysis method proposed in a previous report has been extended to evaluate the elastic shear strains arising from spatial variability of the soil property of a surface layer subjected to shear waves incident vertically from below through a semi-infinite firm ground. The shear strain Υ_0 in the x-z plane due to the incident shear wave S_0 with the particle motion in the direction of the x-axis and the shear strain Υ_1 in the x-y plane due to the incident shear wave S_1 with the particle motion in the direction of the y-axis are considered.

Applying the method, the root mean square (RMS) values of the shear strains are evaluated for the metropolitan Tokyo area on the basis of the local soil conditions, and the correlation between such RMS values and the statistics of the damage collected on the underground water supply pipelines under the 1923 Kanto earthquake has been examined. The comparison indicates that a good correlation exists between the RMS values for Υ_1 and the degree of the damage, while the correlation is not as good for Υ_0.

● **3.5-20** Justo, J. L., Lorente de No, R. and Arguelles, A., **An integrated estimate of ground motion**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. I, 1977, 541–546.

The authors are developing a computer program which will estimate the seismic ground motion at a site.

3.5-21 Iannaccone, G. and Scarpa, R., **Seismic response of local earthquakes recorded at Lipari, Eolian Islands**, *Bollettino di Geofisica*, **XX**, *73-74*, Mar.-June 1977, 62–68.

The Haskell method is used to investigate the crustal structure with long-period body or surface waves. The application of this technique to short-period waves from microearthquakes is discussed. The preliminary results obtained are encouraging. The possibility of using the technique for computing resonant frequencies and ground amplification factors of shallow layers is shown.

● **3.5-22** Campbell, K. W. and Duke, C. M., **Correlations among seismic velocity, depth and geology in the Los Angeles area**, *UCLA-ENG-7662*, School of Engineering and Applied Science, Univ. of California, Los Angeles, June 1976, 43.

Correlations among seismic velocity, Poisson's ratio, depth, and a geotechnical classification scheme were developed from 63 in-situ velocity measurements in the greater Los Angeles area. Average shear wave velocities at the surface for 11 soil and geologic materials were found to vary from about 500 ft/sec for unconsolidated soils to about 3900 ft/sec for a fractured basement complex. A preliminary estimate of Poisson's ratios for the near-surface yielded values of approximately 1/4 for compacted fill, 1/4 for rock, 1/3 for soil above the water table, and 1/2 for soil below the water table. The functional relationship between shear wave velocity and depth was found to be given adequately by $V_s = Kd^n$ between depths of 10 and 100 ft, where the constants K and n are dependent upon the geotechnical classification.

The correlations were used to estimate low-strain shear and P-wave velocity profiles at two sites in the Los Angeles area, for which velocity data were available but had not been used to establish the correlations. The good agreement between the estimated and measured velocities at the two sites suggests that the correlations may be used in establishing synthetic near-surface velocity profiles from shallow geotechnical data when actual velocity data are unavailable.

3.5-23 Kobori, T. and Shinozaki, Y., **Scattering of SH waves from surface irregularities** (in Japanese), *Zisin, Journal of the Seismological Society of Japan*, **30**, *2*, Aug. 1977, 127–142.

This paper deals with the scattering and diffraction of SH waves from surface irregularities of a soil medium. The boundary value problems are precisely formulated in terms of coupling integral equations, which are reduced to the finite difference equations to be easily evaluated. Two types

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of irregularities are considered: alluvial valley with an arbitrary cross section, and a step-like irregularity in the surface of the soil medium. It is shown that the effects of irregularities on the amplitude characteristics of surface displacement are highly dependent upon not only incident angle and wavelength of SH waves but also the impedance ratios between the alluvial valley and the halfspace and the configuration on the surface irregularities. The results are also compared with those derived from the flat-layer theory.

3.6 Seismic Site Surveys

● 3.6-1 McGuire, R. K., **Seismic design spectra and mapping procedures using hazard analysis based directly on oscillator response,** *Earthquake Engineering and Structural Dynamics,* **5,** *3,* July-Sept. 1977, 211-234.

The calculation of design spectra for building sites threatened by seismic ground motion is approached by considering the maximum responses of linearly elastic oscillators as indicators of ground motion intensity. Attenuation functions describing the distribution of response as a function of earthquake magnitude and distance are derived using 68 components of recorded ground motion as data. With a seismic hazard analysis for several hypothetical building sites, the distributions of maximum oscillator responses to earthquakes of random magnitude and location are calculated, and spectra are drawn to indicate the maximum responses associated with specified probability levels. These spectra are compared to design spectra calculated from published methods of amplifying peak ground motion parameters. The latter spectra are found to be inconsistent in terms of risk for building sites very close and very far from faults. A ground motion parameter defined to be proportional to the maximum response of a 1 Hz, 2% damped linearly elastic oscillator is investigated; this parameter, in conjunction with peak ground acceleration, is found to lead to risk-consistent design spectra. Through these two parameters, a design earthquake magnitude and design hypocentral distance are defined for a specified building site and risk level. The use of these parameters in the seismic hazard mapping of a region is illustrated.

● 3.6-2 Grandori, G. and Petrini, V., **Comparative analysis of the seismic risk in sites of different seismicity,** *Earthquake Engineering and Structural Dynamics,* **5,** *1,* Jan.-Mar. 1977, 53-65.

Four sites of differing seismicity are considered. The calculations, for two different buildings on each site, consisted of: (1) the total monetary cost as a function of the seismic design coefficient; (2) the marginal cost of a saved life, also depending on the seismic design coefficient; and (3) the ratios between the seismic design coefficients in the various sites that lead to the same marginal cost in all sites. The marginal cost turns out to be highly sensitive to uncertainties, but the other ratios are scarcely influenced by the uncertainties met in analyzing the problem. This would suggest the adoption of a rationalized principle for comparing the levels of severity in the codes for sites with differing degrees of seismicity. If a criterion for evaluating indirect cost due to damage or collapse is fixed for each building and each site, the values of the seismic design coefficient corresponding to minimum monetary cost are not strongly influenced by the said uncertainties. This would make it possible to look into the problem of the minimum levels of severity in the codes. Of course, this assumes that the levels of severity should always be higher than those corresponding to the minimum monetary cost.

● 3.6-3 Krinitzsky, E. L. and Patrick, D. M., **Earthquake investigations at the Dickey-Lincoln School damsites, Maine,** *Misc. Paper S-77-2,* Soils and Pavements Lab., U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, Jan. 1977, 69.

The Dickey-Lincoln School dam sites are less than 50 miles from an area along the St. Lawrence River which has experienced some of the most severe earthquakes in North America. A geological and seismological investigation was made of the region in order to determine the hazards from earthquakes at the dam sites. No active faults were found in the general area of the dam sites. The source area of potentially severe earthquakes was found to be restricted to a narrow band that follows the St. Lawrence River. This band was designated as Zone A. The boundary of Zone A is located 45 miles from the dam sites. Zone B, with less seismic risk, borders Zone A and is 40 miles from the dam sites. The dam sites are situated in Zone C, which has the least seismic risk in the region. Zone D, with a level of seismic risk between that of Zones B and C, occurs 75 miles southeast of the dam sites. The most severe ground motion at the dam sites was interpreted to be from an earthquake in Zone A attenuated over a distance of 45 miles. Such movement is interpreted to have a peak acceleration of 0.35 g, a peak velocity of 65 cm/sec, and a peak displacement of 22 cm. The duration of shaking is estimated at 18 sec. Accelerographs are recommended for scaling in order to develop time histories of bedrock ground motion for dynamic analyses.

● 3.6-4 Schmoll, H. R., Krushensky, R. D. and Dobrovolny, E., **Geologic considerations for redevelopment planning of Managua, Nicaragua, following the 1972 earthquake,** *Professional Paper 914,* U.S. Geological Survey, Washington, D.C., 1975, 23.

A brief reconnaissance investigation of the geology in the vicinity of Managua, Nicaragua, was undertaken by the U.S. Geological Survey at the request of the Agency for International Development, U.S. Dept. of State. The objective was to determine whether any areas within about 15

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km of the present city are better suited than others as sites for reconstruction, taking into consideration volcanism, geologic materials and structures, and seismicity.

During the time that man has inhabited the area, several meters of materials from four volcanic centers have been deposited in the vicinity of Managua. These deposits include lava flows, mudflows, ash flows, and ash falls. Volcanic activity continues today; widespread and destructive deposition of volcanic materials similar to that of the past can occur at any time. Most of the materials underlying the Managua area range from loose or poorly consolidated pyroclastic deposits to partly indurated mudflow and ash-flow deposits; lava flows, alluvium, and soil are dominant in only small areas. The partly indurated deposits are widespread and have sufficient strength and stiffness to provide adequate foundations in most places; these characteristics are not likely to be altered by seismic shock. Linear geologic features, including faults, fractures, and lineaments, are present throughout the Managua area; they can be grouped, on the basis of orientation, into seven sets, most of which are directly related to the regional geologic structure. Faulting has occurred during the last 10,000 years along nearly all these trends; faulting and concomitant seismic activity, with varying frequency and magnitude along different trends, can be expected to continue.

The authors conclude that no site within about 15 km of Managua is substantially better suited for the capital than the present site, because all such sites share to a significant degree the geologic hazards of the present city. Clearly, it would be desirable to locate a major city in a geologically more stable part of Nicaragua, and search for such a site with a long-term goal of at least partial relocation should be undertaken. For the immediate future, it may be possible to cope successfully with the geologic hazards of the Managua area if appropriate reconstruction plans are developed and rigorously carried out.

● **3.6-5** Askar, A., Cakmak, A. S. and Engin, H., **Strong ground motion spectra for layered media,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(a), Paper K 1/13, 11. (For a full bibliographic citation, see Abstract No. 1.2-5.)

This article presents an analytical method and calculations of strong-motion spectra for energy, displacement, velocity, and acceleration based on the physical and geometric ground properties at a site. Although earthquakes occur with large deformations and high stress intensities which necessarily lead to nonlinear phenomena, most analytical efforts to date have been based on linear analyses in engineering seismology and soil dynamics. There are, however, a wealth of problems such as the shifts in frequency, dispersion due to the amplitude, the generation of harmonics, and removal of resonance infinities, which cannot be accounted for by a linear theory. In this study, the stress-strain law for soil includes different elasticity coefficients and damping for each layer. The above stress-strain law describes soils with hysteresis where the hysteresis loops for various amplitudes of the strain are no longer concentric ellipses as for linear relations but are oval shapes rotated with respect to each other similar to the materials with the Osgood-Ramberg law. It is observed that even slight nonlinearities may drastically alter the various response spectra from that given by linear analysis. In fact, primary waves cause resonance conditions such that secondary waves are generated. As a result, a weak energy transfer from the primary to the secondary waves takes place, thus altering the wave spectrum. The mathematical technique that is utilized for the solution of the nonlinear equation is a special perturbation method as an extension of Poincare's procedure. The method considers shifts in the frequencies which are determined by the boundedness of the energy. The squared bedrock displacement/layer thickness ratio is used as the perturbation parameter.

First, the general solution of shear waves in a typical layer is obtained. Based on the latter, transfer matrices for the various harmonics of the motion are constructed by relating the displacement and the stress at one face to those at the other. With the use of these transfer matrices, the solution of a multilayered system is obtained by requiring continuity in the nonlinear displacement and nonlinear stress.

Results are presented for case studies in one, two, and three layers as well as for actual data for a site with five layers at Las Vegas. Important changes in the spectra of weak motions (or microtremors) are observed for various amplitudes of strong motions. It is also observed that the nonlinear effects are more pronounced on the top layer. This latter has a much lower shear modulus as compared to the lower layers and thus undergoes higher strains. The formalism here in combination with a finite element method is readily extended to apply to irregular geometries in two dimensions.

● **3.6-6** Giardina, Jr., S., **A regional seismic evaluation of Flagstaff, Arizona,** *Bulletin of the Association of Engineering Geologists,* **XIV**, 2, Spring 1977, 89–103.

A seismotectonic study of Flagstaff, Arizona, was conducted to determine the range of possible expected ground motion parameters which could originate from local and regional earthquake activity. This study is based on correlation of regional geology and historical earthquake data obtained from multiple sources. The study concludes that the Flagstaff area could be affected by future major earthquake activity, consisting of a local event originating on the Oak Creek Canyon fault system. Distant events originating in other seismically active areas of

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northwest Arizona and extreme southwest Arizona are not likely to produce significant ground motion at Flagstaff.

● **3.6-7** Der Kiureghian, A. and Ang, A. H.-S., **A fault-rupture model for seismic risk analysis,** *Bulletin of the Seismological Society of America,* **67,** *4,* Aug. 1977, 1173–1194.

A model for seismic risk analysis consistent with existing theories of earthquake mechanism and characteristics is developed. This model is based on the assumption that an earthquake originates as an intermittent series of fault ruptures in the earth's crust, and that the intensity of motion at a site is mainly contributed by the segment of the ruptured fault that is closest to the site. Since active faults in a region may be well defined, partially defined, or completely unknown, various idealized source models are introduced in order to permit the modeling of all conceivable seismic sources. The significance of model parameters on the calculated seismic risk is studied, and certain previous conclusions in this regard are critically reexamined. In particular, it is pointed out that previous seismic risk models, which implicitly assume that the energy is radiated from a point (the focus), can seriously underestimate the real risk, especially for high-intensity motions. As an illustration of the model, the seismic risk analysis of a site in downtown San Franciso is presented.

● **3.6-8** Cleveland, G. B., **Geology at the shoreline, Topanga Beach, Los Angeles County, California,** *California Geology,* **30,** *8,* Aug. 1977, 171–176.

A site study of geologic and shoreline processes was made at Topanga Beach State Park, Los Angeles County. The park is a one-mile-long strip of beach lying in an active seismic area. The Malibu Coast fault is south of the park site, about 1/2 mile offshore. However, shaking at the site is more likely from other regional faults, especially the San Andreas fault which is about 45 miles to the northeast. The closest trace of the San Andreas fault zone to the park site has a maximum expected magnitude of 8.25 while the Malibu fault has a maximum expected magnitude of 7.5. The principal effect of a significant seismic shock near the park would be landslides, which pose a potential threat to buildings and people in the beach area. Tsunami danger appears to be greatest from waves generated by distant sources in the Pacific basin rather than by local offshore fault movements.

Also considered in the article are such geologic factors as bedrock units, surficial deposits, beach sand, deltaic deposits, and landslides.

● **3.6-9** Crouse, C. B., Guzman, R. and Espana, C., **Probabilistic evaluation of liquefaction with an application to a site near a subduction zone,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2195–2200.

The probability of liquefaction occurrence during the life of a facility is calculated based on the cyclic strength of the subsurface saturated granular soils, the seismicity of the region, and the two independent characteristics of earthquake shaking that affect the occurrence of liquefaction: (1) the level of strong shaking, and (2) the duration of shaking. The method is applied to a site on the southern coast of Java that is in a region of active subduction which has a history of large earthquakes.

● **3.6-10** Murray, R. C. and Tokarz, F. J., **Seismic evaluation of critical facilities at the Lawrence Livermore Laboratory,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2645–2651.

The performance of critical facilities at the Lawrence Livermore Lab. is being evaluated for severe earthquake loading. Facilities at Livermore, Site-300, and the Nevada Test Site are included in this study. These facilities are identified, the seismic criteria used for the analysis are indicated, the various methods used for structural analysis are discussed, and a summary of the results of facilities analyzed to date are presented.

● **3.6-11** Ohta, T. *et al.*, **Study on seismic response of soft alluvial subsoil layers by simulation analysis,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 402–408.

The seismic response characteristics of soft alluvial subsoil layers at three different sites were investigated using simulation analysis based on earthquake observations. The ground motion during an earthquake can be simulated approximately by adopting a simple method, namely assuming an SH-wave to be an incident wave, and using the same conditions for the three sites. This procedure and some results are shown.

● **3.6-12** Yoshikawa, S., Iwasaki, Y. T. and Ishii, E., **A probabilistic approach to estimate design earthquake for a site in terms of magnitude, epicentral distance and return period,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 575–583.

A method is proposed to obtain a design earthquake in terms of magnitude, epicentral distance, and return period based on a probabilistic treatment of earthquake data. Gumbel's model is applied to simulate the occurrence of the maximum magnitude earthquake in a given time and in a given epicentral distance zone. As an example, earthquake data near Osaka, Japan, are studied; and the results of the analysis are discussed.

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● **3.6-13** Whitman, R. V. and Taleb-Agha, G., **Seismic risk for multiple sites,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 799–805.

The probability that site shaking will exceed some threshold level is computed for two and three dispersed sites within a large uniform source of earthquake epicenters. Results include the probability that the threshold will be exceeded for at least one site and that it will be exceeded at several sites simultaneously. Results of a parametric study are useful for preliminary studies of the siting of key facilities.

● **3.6-14** Oliveira, C. S., **Seismic risk analysis for a metropolitan area,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 686–694.

A method for analyzing the seismic effects on a system extended in space is developed. The system is characterized by the description in space of its dynamic and resisting properties. The seismic activity is controlled by three parameters and by the spatial distribution of earthquakes. A penalizing function is used to convert the vector of performances into a scalar loss. The probability distribution of losses is generated using simulation. Comparisons between a single site and a system extended in space are analyzed.

● **3.6-15** Eguchi, R. T. and Campbell, K. W., **Seismicity and site effects on earthquake risk,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 756–761.

An earthquake risk model that incorporates a Bayesian estimate of seismicity and an amplification factor for local site conditions is developed and applied to the Los Angeles area. Recency of faulting is used as a criteria for assigning activity rates to faults with no "historic" record of earthquake epicenters. Shear-wave velocity profiles are used to characterize local site conditions. One-hundred-year expected peak velocities are computed by combining the above models with probabilistic and statistical models of earthquake occurrence.

● **3.6-16** Der-Kiureghian, A. and Ang, A. H.-S., **Risk-consistent earthquake response spectra,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1767–1772.

Earthquake response spectra corresponding to given levels of risk and their relevance in structural design are discussed. Development of such spectra, based on seismic risk analysis with a line-source model, is introduced. It is shown that spectra developed with this technique include the effects of site conditions and risk level. Response spectra corresponding to various risk levels are developed and compared for two sites with different conditions. Uncertainties associated with the method are analyzed, and their effects are estimated.

● **3.6-17** Smith, P. D., Vaish, A. K. and Porter, F. L., **Site-structure interaction for a floating nuclear plant under seismic loading,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1682–1689.

This paper describes the application of the finite element method for a floating nuclear generating facility for which dynamic interaction between the floating nuclear plant, its mooring system, other site-related structures, the sea water, and the foundation soils must be included in the determination of the seismic response. Standard finite element techniques have been augmented by the development of a fluid finite element for sea water modeling and by the formulation of a "split-modeling" procedure in which the analyses of a particular cross section are performed using two site models, one a large coarse model to obtain low-frequency site response, the other a smaller more refined model to obtain high-frequency response. The results from the two models are combined to obtain the site response over the full frequency range of interest. The advantage of this procedure is a substantial reduction in computational effort.

● **3.6-18** Kitagawa, Y., **Observation system for underground earthquake motions in Japan,** *BRI Research Paper 73,* Building Research Inst., Tokyo, Mar. 1977, 12.

Since observation of underground earthquake motions in Japan began in the 1960s, effort has been made to determine characteristics of ground motion, dynamic behavior of structures, and input motions to structures. These efforts have been aided by the development of bore-hole instrumentation techniques. The object of earthquake observation is divided into two main categories. One is to obtain information on the dynamic elastic characteristics of soils and structures during small or moderate earthquakes, including aftershocks and earthquake swarms. The other is to determine the dynamic elastoplastic characteristics of soils and structures during strong earthquakes.

● **3.6-19** Srivastava, J. P. *et al.,* **Geotechnical factors in the evolution of seismic coefficient of Kadana Dam on Mahi River, Panchmahals district, Gujarat, India,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 921–922.

A 66 m high earth and masonry dam, the Kadana Dam, is being constructed across the Mahi River to provide irrigation to Gujarat. The foundation rocks include intricately folded Aravalli quartzite and phyllites traversed by 17 faults in addition to shears, fractures, and joints. In the

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reservoir area, comprised of folded Aravalli, there is a hot spring and a major fault. The project is surrounded at a distance of 128 to 160 km by a regional fault of Aravalli, Cambay graben, and Narmada-Sone lineament. A seismic coefficient of 0.1 g for the masonry part of the dam has been recommended.

● 3.6-20 Shah, H. C. *et al.*, **A seismic design procedure for Nicaragua,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1795-1800.

Using the iso-contour map for Nicaragua and acceleration zone graphs for the principal population centers, it is possible to determine the peak ground acceleration values of earthquake events having a given risk of exceedance during a structure's life. Appropriate levels of design earthquakes can thereby be selected at a given site such that the risk of occurrence is consistent with the use priority of a proposed structure.

This paper describes a proposed seismic design procedure which employs these consistent risk earthquake levels as load input in the form of response spectra. The essential elements of the design procedure are (1) definition of structural use classifications and their corresponding risk levels; (2) statistical description of the response spectrum shape for a given region and site; (3) structural modeling and computation of response; (4) definition and grading of lateral force-resisting systems; (5) formulation of the design spectra; (6) member design criteria; (7) verification of drift control at the damage threshold earthquake and local ductility demands at the condemnation threshold earthquake; and (8) a simplified method of determining the base shear.

● 3.6-21 **Geotechnical and strong motion earthqake data from U.S. accelerograph stations: Ferndale, Cholame, and El Centro, California,** *Vol. I,* Shannon & Wilson, Inc., and Agbabian Assoc., Seattle, Washington, and El Segundo, California, Sept. 1976, var. pp.

Seismic and geotechnical data for seismograph stations located in Ferndale, Cholame (No. 2), and El Centro, California, are presented. At Ferndale, the investigations indicate that there is a surficial layer of silty clay with shear velocity (Vs) of about 500 fps, becoming stiffer between 25 and 42 ft with Vs of 800 fps. Between depths of 42 and 153 ft, Vs increases to a value of 2000 fps. Materials from 153 to 172 ft are very dense and fine to coarse sands and gravels. At Cholame No. 2, the materials are primarily stiff to very stiff clays and silts overlying dense sands and silts. The upper 150 ft has a shear wave velocity between 500 and 1000 fps. The sands below this have an increasing velocity with depth and reach 2000 fps at 200 ft. At El Centro, the subsurface soils are stiff to hard clays with very dense, fine sand. From the surface to a depth of 114 ft, Vs varies between 400 and 1000 fps. From this depth to the bottom of the borings at 400 ft, there is a silty clay with occasional zones of very dense, silty, fine sand. The shear wave velocity varies from about 1000 fps at 100 ft to about 1500 fps at 400 ft. This report includes illustrations, graphs, and detailed charts of the results.

● 3.6-22 Arnold, P. *et al.*, **The Gulf of Alaska earthquake measurement program, Exhibit A,** *Vol. I,* Shell Oil Co., Houston, Texas, Mar. 1977, 50.

A program is proposed which includes both the measurement and evaluation of earthquake ground motion data from onshore and offshore sites for the Gulf of Alaska Sale No. 39 Lease Area. The overall goal of the effort is to establish improved information on the occurrence, location, and characteristics of earthquakes, and the resulting ground motions and the manner in which they vary over the lease area. In particular, the emphasis of the proposed Earthquake Measurement Program (EQMP) is on improving understanding in three areas: the energy release, or source characteristics of earthquakes; the effects of attenuation, or transmission, of seismic waves as they travel from the earthquake source to a distant site; and the effects of local site conditions in modulating earthquake ground motions in the Gulf area.

The EQMP is to be a jointly sponsored project which is consistent with and complementary to existing oil industry earthquake engineering programs, such as the Offshore Alaska Seismic Exposure Study, the study of the behavior of offshore soil-pile-structure systems under shaking, and the Earthquake-Wave Stability Study. The primary objective of the EQMP is to obtain sufficient earthquake ground motion measurements to allow evaluation of the applicability of present attenuation/transmission and site modulation models and to better define source parameters for earthquakes affecting the Gulf of Alaska area. Other sections of the document present detailed discussions of such topics as earthquake ground motion characteristics and their influence on platform response; offshore and onshore measurement systems; instrument site selection; data analysis and evaluation; and project organization, schedule, cost, and justification.

● 3.6-23 **The design earthquake: geological and seismological evaluation of earthquake hazards at the Richard B. Russell project,** Corps of Engineers, U.S. Army, Savannah, Georgia, Mar. 1977, var. pp.

No evidence of active faults is present in the general area of the Richard B. Russell dam site. It is concluded that the faults which are present are all ancient and inactive. Present day tectonism indicates that moderate earthquakes occur in the Piedmont and adjacent regions. Severe earthquakes have been restricted to the vicinity of Charleston, South Carolina. The faults are obscured by coastal plain

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sediments. Motions attenuated from Charleston to the Richard B. Russell dam site are low with the distance being about 170 miles.

A probability of recurrence study showed that the operating earthquake has an acceleration of 0.075g (0.01 annual risk). The operating basis earthquake is generally more moderate than the maximum earthquake. It is selected on a probabilistic basis from regional and local geology and seismology studies as being likely to occur during the life of the project; in this case, 100 years. In this instance, this peak ground acceleration is considerably smaller than the design value of the Richard B. Russell Dam. In other words, the dam will be designed to withstand an earthquake several times stronger than would be likely to occur during its useful life.

● **3.6-24** Whitman, R. V. and Protonotarios, J. N., **Inelastic response to site-modified ground motions**, *Seismic Design Decision Analysis Report No. 31,* Dept. of Civil Engineering, Massachusetts Inst. of Technology, Cambridge, Apr. 1977, 45.

Coincidence of the natural period of a structure and the natural period of the supporting ground may mean unusually strong response of the structure to an earthquake if the structure remains elastic. However, if the structure yields, its period in effect changes and the structure "slides off" the peak of an elastic response spectrum. Indeed, inelastic response spectra for site-modified ground motions do not show pronounced peaks at the fundamental period of the soil profile; rather, they are as "smooth" as inelastic spectra which are not thought to be influenced by site conditions. The ratios of maximum spectral velocity for a given ductility ratio/peak velocity and maximum spectral velocity for a given ductility ratio/peak acceleration are very similar for both site-modified and normal motions. This does not mean that site conditions are unimportant, since the soil profile at a site may affect peak velocity and acceleration. Rules derived from parametric studies may be used to provide inelastic spectra as a function of site conditions and as a basis for improved site factors for building codes.

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4. Strong Motion Seismometry

4.1 Instrumentation

● 4.1-1 Seleznev, G. S. *et al.*, **Instrumental observations of the earthquake vibrations of dams in the Tadzhik, S.S.R.** (Instrumentalnye nablyudeniya za kolebaniyami plotin pri zemletryaseniyakh na territorii Tadzhikskoi SSR, in Russian), *Seismostoikoe stroitelstvo, 4,* 1977, 25–30.

The location and technical specifications of recording stations installed on hydraulic structures in the Tadzhik, S.S.R. are discussed. The data obtained from these stations are used to calculate generalized spectra for earthquakes of varying magnitudes and at varying epicentral distances. By extrapolating these results, the expected behavior of the Nurek and Ragun hydroelectric power plants under construction may be calculated.

● 4.1-2 Katen-Yartsev, A. S. *et al.*, **Special features of vibration recording in buildings with gravitational aseismic systems** (Osobennosti registratsii kolebanii na zdaniyakh s gravitatsionnymi sistemami seismoizolyatsii, in Russian), *Seismostoikoe stroitelstvo, 4,* 1977, 22–25.

The special dynamic properties of experimental buildings with aseismic design features impose specific demands on the vibration recording instruments installed. The devices must be capable of recording low-frequency vibrations with periods approximately equal to 3.6 sec and displacements up to 150 mm, and high-frequency vibrations with displacements on the order of 1 μ. Various design solutions are tested and calibrated using a shaking table. The optimal combination of devices is selected.

● 4.1-3 Kirnos, D. P., **Modern seismic apparatus for direct recording of vibrations in strong-motion earthquakes** (Sovremennye seismicheskie pribory s pryamoi registratsiei dlya zapisi kolebanii pri silnykh zemletryaseniyakh, in Russian), *Seismostoikoe stroitelstvo, 4,* 1977, 12–16.

The requirements for strong-motion recording instrumentation are discussed. Reliability and stability criteria are analyzed. The advantages of direct optical recording are pointed out. The specifications of the strong-motion accelerographs are examined in detail: the SSRZ of the Earth Physics Inst. of the U.S.S.R. Academy of Sciences and the SMA-1, manufactured by Kinemetrics in the U.S. The performance of the two accelerographs is compared in field tests.

● 4.1-4 Hudson, D. E., **Strong motion seismology,** *Bulletin of the New Zealand National Society for Earthquake Engineering,* **10,** *3,* Sept. 1977, 113–120.

Strong-motion accelerographs and data processing systems for the measurement of strong ground motion of damaging earthquakes are described with some comments on future developments. Some information is given on the earthquakes and instrumentation sites for which important records have been obtained. Studies of the areal distribution of ground motions during earthquakes are discussed in terms of transmission path effects. Some speculations are advanced concerning the establishment of limiting values of earthquake ground motions.

● 4.1-5 Cramer, C. H. and Sherburne, R. W., **Microearthquake recording systems,** *California Geology,* **30,** *1,* Jan. 1977, 16–18.

In 1976, the California Div. of Mines and Geology (CDMG) acquired eight portable microearthquake recording systems capable of recording earthquakes smaller than Richter magnitude 3.0. Each system is composed of a recorder unit and a seismometer. The CDMG field tested four of the instruments in the area north of Sacramento, recording several microearthquakes.

● 4.1-6 Sattaripour, A., **A simple instrument for determining earthquake response spectra,** *Proceedings, Sixth*

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World Conference on Earthquake Engineering, Sarita Prakashan, Meerut, India, Vol. III, 1977, 2969–2974.

A simple instrument for recording strong-motion earthquakes is described and results obtained with the instrument under test conditions are compared with those obtained by a standard spectrum analysis of accelerograph records. The instrument is composed of a regular seismoscope which is equipped with a time-scaling device. From the time-scaled record obtained by the instrument, the relative velocity is measured. Using a mathematical model of the instrument, the ground motion is determined and the response spectra are calculated by conventional methods. The simplicity and low cost of the instrument makes it possible to be installed in large numbers in earthquake regions.

● **4.1-7** Pauly, S. E., **Review of current standards and practice for earthquake instrumentation at nuclear plants,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2962–2968.

Strong-motion recording systems are in use at more than 50 nuclear power plants in several seismically active countries. Automatic seismic shutdown devices are used at several plants, primarily in Japan, whereas other plants depend on evaluation by the operating staff as to whether safe operation can continue. Criteria for instrument capabilities and locations have been developed, principally in the United States (1970 and 1974) and by the International Atomic Energy Agency (1975) as guides to adequate instrumentation. Several countries are in the early stages of instrument criteria development.

4.1-8 Muramatu, I., **A velocity type stong motion seismograph with wide frequency range** (in Japanese), *Zisin, Journal of the Seismological Society of Japan,* **30**, *3,* Dec. 1977, 317–338.

Upper boundaries of the strong earthquake motions recorded until now indicate an almost constant value of 20–30 cm/sec over the period range from about 1 sec to 20 sec. Accordingly, it may be thought that a velocity seismometer is suitable to observe strong earthquake motion. Weak ground motions which occur frequently are also valuable for researching properties of strong earthquake motions. The author constructed a velocity-type seismograph which can record earthquake motions of 100 kines to 0.01 kines with high fidelity in the period range from 0.05 sec to 40 sec. The main part of this seismograph has a coupled pendulum which was designed by T. Matuzawa in 1938 and was improved by the author. The whole system of the coupled pendulum is immersed in high viscous silicon oil. The relative displacement of the pendulum is transformed to electric voltage by means of magnetic flux to which a temperature compensating circuit with a thermistor is connected.

● **4.1-9** Agrawal, P. N., **Performance of multiple structural response recorders in India,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2975–2977.

4.2 Regional Data Collection Systems

4.2-1 Sadovskii, M. A. and Kondorskaya, N. V., **Perspectives of development of a unified seismic observation network in the U.S.S.R.,** (Perspektivy razvitiya v S.S.S.R. edinoi sistemy seismicheskikh nablyudenii, in Russian), *Vestnik Akademii Nauk SSSR, 10,* 1976, 30–36.

A historical survey is given of the development of a unified seismic observation network in the U.S.S.R. Future perspectives are discussed. It is noted that at the present time the most important task is not the accumulation of stations. The emphasis should be on their choice and site selection, but the quality should improve substantially. The instrumentation and work of the Obninsk seismological observatory are described. This is the prototype of future seismological stations. The automatic information processing and storage made possible by modern electronics open unlimited possibilities for amplifying the processes recorded, correcting instrumental readings during the recording itself, and the separation of useful signals from background noise. Magnetic or perforated tape recording allows the full utilization of earthquake records. The condensation of information at the place of registration using low speed magnetic tape in continuous or triggered mode operation is discussed. A multi-channel recording of digital data obtained from analog input signals is analyzed. Recording of other geophysical fields (gravimetric and tiltmetric measurements) is stressed. Some of the observations will function as full automatic network centers connected by telecommunications links feeding information to a single center. Information must be coded in a form suitable for direct computer input. Ordinary communications links will be used (microwave at present, satellite in the future). The types of data obtainable from the network envisaged are discussed.

● **4.2-2** Martemyanov, A. I. and Ponomarev, O. I., **The performance of the seismometric network for the registration of strong-motion earthquakes in the U.S.S.R.** (Opyt raboty inzhenerno-seismometricheskoi sluzhby po registratsii silnykh seismicheskikh dviznenii na teritorii SSSR, in Russian), *Seismostoikoe stroitelstvo, 4,* 1977, 3–7.

At present, the Unified Seismic Observation Network consists of 200 stations. However, for various reasons, the network is insufficient for the purposes of engineers and

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investigators working in the field of earthquake engineering. The history and specifications of the network are outlined. Plans for future development are discussed.

● **4.2-3** Mamaeva, G. V., **The present state of the Engineering Seismometry Network in the U.S.S.R.** (Sovremennoe sostoyanie inzhenerno-seismometricheskoi sluzhby v SSSR, in Russian), *Seismostoikoe stroitelstvo, 4,* 1977, 7–10.

Recordings of ground motion and building vibrations form the basis for the development of earthquake engineering. The Engineering Seismometry Network was organized with the participation of more than 20 research institutes. The efficiency of the network is analyzed and the effects of geological, seismological, technical, and economic factors on the development of the network are discussed.

● **4.2-4** Vypryazhkin, Yu. A., **Operational control of the engineering seismometric stations in Alma-Ata** (Kontrol raboty inzhenerno-seismometricheskikh stantsii Alma-Aty, in Russian), *Seismostoikoe stroitelstvo, 4,* 1977, 16–19.

Reliable operation of a seismic recording network can only be insured by strict systematic control of instrumentation, equipment, and communications facilities. During the nine years of operation in Alma-Ata, the techniques for controlling the operation of the network located there were established. These techniques included monthly maintenance and testing schedules. The testing procedures are discussed in detail.

● **4.2-5** Ponomarev, O. I. and Kovalchuk, M. N., **Development of the network of seismic recording stations in buildings** (Razvitie sluzhby registratsii kolebanii zdanii i sooruzhenii pri zemletryaseniyakh, in Russian), *Seismostoikoe stroitelstvo, 1,* 1977, 27–30.

The earthquake engineering recording network was created in 1965 and it has undergone substantial development since then. The number of stations (including those installed at hydraulic and transportation structures) is approximately 100 at the present time. The stations are located in buildings of various heights and design principles. The buildings were constructed on various types of soils using mass-production methods. A map showing the location of the stations and the types of soils and structures where they are installed is presented. The direction of the future development of the network is discussed.

● **4.2-6** Matthiesen, R. B., **Planning and design of strong motion instrument networks,** *Wind and Seismic Effects,* III-1–III-15. (For a full bibliographic citation, see Abstract No. 1.2-4.)

The types of research studies that utilize strong-motion data may be classified as: source mechanism studies, ground motion studies, soil failure studies, studies of the response of typical structures (including soil-structure interaction effects), and studies of the response of equipment. In planning networks and arrays to make these studies, criteria must be established based on the tectonic setting, the seismicity or recurrence of strong ground motions, the reliability of operations in different regions, and a cost/benefit analysis of the data that may be obtained. A review of the strong-motion records that have been obtained during the past 40 years indicates significant variations in the recurrence of strong ground motions in the seismically active regions of the western United States. When combined with instrument costs, maintenance costs, and the reliability of operations, these recurrence relations can be interpreted in terms of the cost per record for different levels of motion. The benefits to be derived from each type of study in each region need to be established.

Current plans call for additional arrays to be installed in California, the Mississippi embayment, the Yellowstone Park region, and Alaska to study the spectral characteristics of strong ground motions in these regions. Special studies of local site effects and structural response are being planned in the more seismically active regions of California. Similar criteria and planning should be applied in the establishment of arrays of strong-motion instruments on a worldwide basis.

● **4.2-7** Rojahn, C., **Building strong motion earthquake instrumentation,** *Wind and Seismic Effects,* III-26–III-40. (For a full bibliographic citation, see Abstract No. 1.2-4.)

Based on the recommendations of a special ad hoc committee, 21 geographic areas will be instrumented under the building instrumentation phase of the California Strong-Motion Instrumentation Program, a statewide program established by law in 1971 and funded through an assessment of estimated construction costs collected statewide from building permits. The areas were selected on the basis of population density, locations of buildings already instrumented, and the probability for potentially damaging earthquakes. Buildings to be instrumented will be of typical construction, simple in framing and design, and of various heights with the instrumentation of low-rise buildings emphasized. Remote recording instrumentation, consisting of single or multiaxial accelerometers connected via data cable to a central recorder, will be installed in each building. The accelerometers will be placed on the lowest level, at the roof level, and, in many cases, at one or more intermediate levels. The instrumentation will be situated so as to separately record both translational and torsional response. On the basis of current projected revenues, and instrument procurement, installation and maintenance expenses, it is estimated that as many as 400 buildings may be instrumented under the program.

● **4.2-8** Habercom, Jr., G. E., ed., **Seismic detection. Part 1. General studies: a bibliography with abstracts,** National

Technical Information Service, Springfield, Virginia, Apr. 1977, 237. (NTIS Accession No. NTIS/PS-77/0234)

General applications of seismic detection and seismic arrays are investigated in these research reports. Earthquake detection, personnel detection, and vehicle detection are reviewed. This updated bibliography contains 237 abstracts, 83 of which are new entries to the previous edition. The search period covered is 1964-Mar. 1977.

● **4.2-9** Stewart, S. W., **Real-time detection and location of local seismic events in central California,** *Bulletin of the Seismological Society of America,* **67,** *2,* Apr. 1977, 433-452.

A computer-based system dedicated full time to automatic detection and location of local seismic events in central California has been developed. The system monitors 108 short-period vertical-component stations from the U.S. Geological Survey central California and Oroville seismic networks. Locations and magnitudes, when determined, are printed out, along with first arrival times, within 2 to 5 min after an event occurs. Wave onsets must be clear and impulsive for best results. For this reason, regional events and teleseisms are usually rejected.

The best results have been obtained for the relatively dense, 16-station Oroville network. For the month of October 1975, 107 (91%) of the 118 events timed by hand were also timed and located by the real-time system. An additional eight events (7%) were detected in real time but were not successfully located. Of the 107 events for which both on-line and hand-timed locations are available, 92% of the on-line locations are within 2 km of the epicenters determined by hand timing.

During October 1975, the real-time system monitored 91 of the 150 stations of the central California network. Of the 260 events located by hand timing, 225 (86%) were detected by the real-time system. Magnitudes of detected events range from 0.8 to 2.9. Approximately 95% of the events of magnitude 1 1/4 and greater detected and located by hand-timing methods were also detected by the real-time system. Differences between hypocentral locations based on hand-timed and computer-timed arrivals may vary from 0.1 to 5 min of latitude or longitude.

● **4.2-10** Wootton, T. M., **Earthquake ground motion records,** *California Geology,* **30,** *4,* Apr. 1977, 86-87.

California's Strong Motion Instrumentation Program (SMIP) was established in 1971 to obtain detailed records of the response of rock and soil units and of engineered structures to ground motion generated by earthquakes. The California Div. of Mines and Geology (CDMG) is the responsible managing agency. The CDMG has installed accelerometers at more than 200 surface geologic sites, 20 buildings, and 11 dams. A more recent program involves the use of down-hole arrays to record the response of subsurface materials to motion generated by earthquakes. Two arrays have been installed, one in San Benito County and one in Humboldt County.

● **4.2-11** Mihailov, V., **Strong motion instrument network in Yugoslavia,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2937-2942.

Yugoslavia has frequently been subjected to disastrous earthquakes. To improve earthquake-resistant design and to reduce earthquake hazards on a worldwide basis, as well as to study the intense seismicity of Yugoslavia, the installation of a strong motion instrument network in Yugoslavia was undertaken at the beginning of 1972. The project is being carried out by the Inst. of Earthquake Engineering and Engineering Seismology, Skopje, in cooperation with the California Inst. of Technology, Pasadena.

● **4.2-12** Prince, J. and Rodriguez, F. H., **An evaluation of the Mexican strong motion radio telemetry network after three years of operation,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2931-2936.

The Seismotelemetric Information System of Mexico (SISMEX) became operational in 1973. Its main purpose is to record ground motion during earthquakes in the Valley of Mexico and neighboring areas. In the last three years, operation, maintenance, and modification of sensitivity ranges, as well as improvement of recording techniques, calibration procedures, protection against electrical transients and further software development, have been possible with moderate effort. Advantages of the system over other earthquake-recording instrumentation lead to the capability of computing response spectra in a matter of hours after the event.

● **4.2-13** **Western hemisphere strong-motion accelerograph station list—1976,** *Open-File Report 77-374,* U.S. Geological Survey, Menlo Park, California, May 1977, 116.

The U.S. Geological Survey (USGS) maintains a network of strong-motion instrumentation for the National Science Foundation in cooperation with other federal, state, and local agencies within the United States. In addition, cooperation is extended to similar groups in other countries throughout the world. Previous station lists published by the USGS contained information on only those stations considered to be part of the cooperative U.S. network. As more organizations throughout the world have developed networks, it has become obvious that composite lists of stations for each of the major regions of the world would be a valuable document for all concerned. The list presented is an attempt to bring into one document a

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minimum amount of information on all the stations in the western hemisphere known to the USGS. It is hoped that similar lists for all the stations in Europe, Asia, and the South Pacific regions will also be compiled.

This list is not complete. Only partial information is available on the more recently installed stations, and as a result of the rapid expansion of several of the networks, no information is yet available on some of the stations. This list is as complete as practicable as of March 1977.

● **4.2-14** Martem'yanov, A. I., Ponomarev, O. I. and Fedorov, S. A., **Experience of work of strong motion registration stations during earthquakes in the Soviet Union,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 823-828.

The functions of strong-motion measurement stations-Uniform System of Seismic Observations (USSO) in the U.S.S.R. are discussed. This paper also estimates optimal quantity and location of seismometric apparatus in different regions of the country. Seismic activity, soil conditions, and related factors are considered in determining the best locations for equipment. Problems of the USSO are also discussed.

● **4.2-15** National Research Council, Panel on Seismograph Networks, **Global earthquake monitoring: its uses, potentials, and support requirements,** National Academy of Sciences, Washington, D.C., 1977, 75.

This study was performed by the Panel on Seismograph Networks of the Committee on Seismology of the National Research Council's Assembly of Mathematical and Physical Sciences. It examines the current capabilities and accomplishments of the Worldwide Standardized Seismograph Network and presents recommendations for its future upgrading and development. Also discussed are international aspects; data handling, processing, and user services; and directions for future research.

● **4.2-16** Glover, D. P., **Directory of seismograph stations,** *SE-7,* World Data Center A for Solid Earth Geophysics, U.S. National Oceanic and Atmospheric Admin., Boulder, Colorado, July 1977, 145.

The World Data Center A for Solid Earth Geophysics has compiled a directory of seismograph stations and their technical characteristics. The directory lists 247 of the approximately 300 seismograph stations participating in the category ii data exchange (International Council of Scientific Unions, 1973) through World Data Centers. Seven categories of data are listed for most stations: 1) general information; 2) site information; 3) instrumentation; 4) timing system; 5) type of reports; 6) references to station descriptions or operations; and 7) comments.

4.2-17 Kuroiso, A. and Watanabe, H., **On the telemetered array system for microearthquake observation at Abuyama Seismological Observatory** (in Japanese), *Zisin, Journal of the Seismological Society of Japan,* **30**, *1,* Apr. 1977, 91-106.

To investigate the seismicity of microearthquakes in relation to the occurrence of large shocks, an array of observation stations spread widely over the northern part of the Kinki District has been operated since 1963. In 1975, a new telemetered system was substituted for the array, enabling simultaneous and real-time recordings. A data processing system with a digital computer was installed at the same time. The system and examples of analyses are described.

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5. Dynamics of Soils, Rocks and Foundations

5.1 General

● **5.1-1** Beeston, H. E. and McEvilly, T. V., **Shear wave velocities from down-hole measurements,** *Earthquake Engineering and Structural Dynamics,* 5, 2, Apr.-June 1977, 181-190.

Shear wave velocities of soils, which provide shear moduli for earthquake response calculations, can be measured clearly and accurately using the down-hole method. Such a method has been used at a number of sites in California with good results to depths of 200 ft. Seismic waves from hammer blows, delivered to the ends of a heavy plank loaded by the front wheels of a vehicle, are received by a three-component geophone in a carefully prepared vertical hole and recorded at 1 mm/ms with a six-channel seismograph. A series of records are obtained at various measured depths in the hole, allowing calculation of interval velocities. Shear waves are easily identified by a clear 180° phase difference between waves generated by blows on the opposite ends of the plank. Compressional waves are routinely logged by a vertical hammer blow at each recording depth. Shear velocities are reproducible to about 5% in surveys of neighboring holes. The reading uncertainty of ± 1 msec for the S arrival gives a resolution sufficient to detect a buried layer 5-10 ft thick with a velocity contrast of only 20%.

● **5.1-2** White, W., Valliappan, S. and Lee, I. K., **Unified boundary for finite dynamic models,** *Journal of the Engineering Mechanics Division, ASCE,* **103**, *EM5,* Proc. Paper 13279, Oct. 1977, 949-964.

The finite element analysis of dynamic problems in an infinite, isotropic medium is examined. To simulate the physically infinite system by a finite model, an energy-absorbing boundary is proposed. This boundary is frequency independent and proves to be very efficient in absorbing stress waves. The boundary constants are calculated for the particular cases of plane strain and axisymmetry for isotropic materials.

● **5.1-3** Schreyer, H. L., **Inverse solutions for one-dimensional seismic waves in elastic, inhomogeneous media,** *Journal of Applied Mechanics,* **44**, **Series E**, *3,* Sept. 1977, 469-474.

An inverse procedure is developed for obtaining exact solutions to the one-dimensional inhomogeneous wave equation. Transformations of the independent spatial variable and the dependent variable are introduced so that the wave equation assumes the form associated with a homogeneous material. The resulting transformation relations are nonlinear but of such a nature that they can be easily integrated if the reciprocal of the wave speed distribution can be expressed in terms of elementary functions. One functional form that yields realistic values for material properties of soil layers is investigated in detail. Amplification factors for a sinusoidal seismic shear wave in inhomogeneous and homogeneous layers are derived and illustrations of significantly different characteristics for the two types of layers are shown.

● **5.1-4** Goto, N. *et al.,* **An easy-capable and high-precise shear wave measurement by means of the standard penetration test,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2322-2327.

An experiment was carried out to develop a new technique to measure shear wave velocity simultaneously with the standard penetration test. A three-component geophone was set on the ground surface near the borehole opening, and the generated waves were recorded for successive depths of the penetration test. Signal amplitudes decrease with depths and are under the noise level at a certain depth. Thus, a simple process of stacking was

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employed with the advantage that the wave forms of signal events by N blows of the penetration at a depth are very similar. Shear wave velocities found using this new method, N values found using the standard penetration test, and other soil indexes were compared. Using the technique described, a simple and highly precise shear wave measurement can be determined in conjunction with the standard penetration test.

● 5.1-5 Viksne, A., **Evaluation of in situ shear wave velocity measurement techniques,** *REC-ERC-76-6,* Engineering and Research Center, U.S. Bureau of Reclamation, Denver, Colorado, Apr. 1976, 46.

Several geophysical methods used by the Bureau of Reclamation to obtain the in-situ shear wave velocity of earth embankments were evaluated. In-situ low-strain shear wave velocity determinations have been performed on a number of existing zoned earthfill dams and an earth dam under construction. These field measurements of shear wave velocity by geophysical exploration methods have become an integral part and standard procedure at the bureau for obtaining input parameters for dynamic analyses of earth dams. In the course of shear wave velocity measurements, various borehole methods, such as downhole, crosshole, and uphole, as well as seismic refraction have been used. Shear wave velocity measurements have been performed with explosive and nonexplosive impulsive energy sources using cased and uncased test holes. The results of these field studies have not only provided input parameters for dynamic analyses, but have also provided information as to which method or combination of methods should be used to obtain optimum results. Test methods and results of the various in-situ measurements are compared and evaluated.

5.2 Dynamic Properties of Soils, Rocks and Foundations

5.2-1 Ilichev, V. A. and Mongolov, Yu. V., **Ultimate strength of piles subjected to seismic excitations** (*Nesushchaya sposobnost svai s uchetom seismicheskikh vozdeistvii,* in Russian), *Trudy NII osnovanii i podzemnykh sooruzhenii, 67,* 1976, 97–111.

A method to calculate the ultimate strength of piles subjected to seismic loads is presented. The effects of horizontal forces arising from the vibration of the superstructure and of changes in the carrying capacity of the soil surrounding the pile on the reduction of the ultimate strength of the pile are investigated. The effects of the first factor mentioned are investigated under field conditions and the second factor is studied primarily through approximate calculations.

● 5.2-2 Kunihiro, T., Yahagi, K. and Okahara, M., **The measurement of the dynamic k-value in site and its application to design,** *Wind and Seismic Effects,* VI-118–VI-140. (For a full bibliographic citation, see Abstract No. 1.2-4.)

The determination of the dynamic soil k property is determined by a series of borehole tests. Results from these tests are discussed and the significance of these data are presented.

● 5.2-3 Nogami, T. and Novak, M., **Resistance of soil to a horizontally vibrating pile,** *Earthquake Engineering and Structural Dynamics,* **5**, *3,* July-Sept. 1977, 249-261.

The resistance of a soil layer to steady horizontal vibration of an elastic pile is theoretically investigated. The pile is assumed to be vertical and of circular cross section. The soil is modeled as a linear viscoelastic layer with hysteretic material damping. A closed-form solution is obtained for the resistance of the soil layer to the motion. This resistance depends on shear modulus of soil, frequency, pile slenderness, material damping and Poisson's ratio. A parametric study of the effect of these parameters is included. The soil layer resistance is expressed in a form which can be used directly in the solution of the soil-pile interaction problem which is treated in a subsequent paper. The approach also applies for rigid deeply embedded footings.

● 5.2-4 Castro, G. and Poulos, S. J., **Factors affecting liquefaction and cyclic mobility,** *Journal of the Geotechnical Engineering Division, ASCE,* **103**, *GT6,* Proc. Paper 12994, June 1977, 501–516.

Liquefaction is a phenomenon wherein a saturated mass of sand loses a large percentage of its shear resistance and flows in a manner resembling a liquid until the shear stresses acting on the mass are as low as its reduced shear resistance. Cyclic mobility is the progressive softening of a saturated sand specimen when subjected to cyclic loading at constant water content. Cyclic mobility has been observed in the laboratory. The authors believe that most observed cyclic mobility deformations in dilative clean sands are due to a test error, redistribution of the void ratio, which is not representative of in-situ behavior. The manner in which soil type, confining stress, and initial consolidation stress ratio affect both liquefaction and cyclic mobility are shown by means of laboratory test results, for the purpose of permitting the reader to develop rational procedures for designing against the effects of earthquake loadings on soils.

● 5.2-5 Reese, L. C., **Laterally loaded piles: program documentation,** *Journal of the Geotechnical Engineering Division, ASCE,* **103**, *GT4,* Proc. Paper 12862, Apr. 1977, 287–305.

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The documentation is presented for the computer program COM622, which solves for the deflection and bending moment of a pile under lateral loading as a function of depth. The calculations are performed on a finite difference model of the pile, and the soil is represented by a series of nonlinear curves of force per unit length versus deflection. This is the first program documented and distributed under the standards developed for the Geotechnical Engineering Div.

● **5.2-6** Lade, P. V. and Hernandez, S. B., **Membrane penetration effects in undrained tests,** *Journal of the Geotechnical Engineering Division, ASCE,* **103,** *GT2,* Proc. Paper 12758, Feb. 1977, 109–125.

It was found that membrane flexibility can change the soil behavior and pore pressure characteristics far more than may be caused by the flexibility of the pore pressure measuring system, for example. Expressions were developed for the pore pressure parameters, B and A, which include the effects of the flexibilities of the membrane and pore pressure measuring system. Measured and calculated B-values agreed well for saturated specimens in which membrane penetration could occur. The effects of time and partial saturation on the measured B-values were also studied. The effect of membrane penetration in undrained tests was to reduce the rate of change in pore pressure, and the effective stress-path, stress-strain relations, and undrained strength were modified accordingly. The strengths obtained from cyclic loading tests performed on saturated specimens of granular materials for determination of liquefaction potential were also shown to be affected by membrane penetration.

● **5.2-7** Sherif, M. A., Ishibashi, I. and Gaddah, A. H., **Damping ratio for dry sands,** *Journal of the Geotechnical Engineering Division, ASCE,* **103,** *GT7,* Proc. Paper 13050, July 1977, 743–756.

Using the torsional simple shear device, the authors investigated the relationship between damping ratio and shear strain for Ottawa, Del Monte, Golden Gardens, and Seward Park sands. Experimental results show that the type of sand, as expressed in soil gradation and angularity, influences soil damping. However, this influence can be taken into consideration by introducing a soil gradation and sphericity factor which is a function of the coefficient of gradation and soil sphericity. The authors propose an equation relating the damping ratio to shear strain as a function of the confining pressure and the soil gradation and sphericity factor. A nomograph is proposed to assist in the easy determination of the damping ratio for sands.

● **5.2-8** Forrest, J. and Ferritto, J., **An earthquake analysis of the liquefaction potential at the Naval Air Station, North Island,** *TR-847,* Civil Engineering Lab., Naval Construction Battalion Center, Port Hueneme, California, Sept. 1976, 81. (NTIS Accession No. AD-A033 493)

The loss of strength experienced by saturated cohesionless soils during earthquake or shock loading is generally referred to as liquefaction. The hazard potential existing at the Naval Air Station, North Island, California, due to this phenomenon is evaluated herein, including a statistical evaluation of potential earthquake levels and an appraisal of the magnitudes of damage. Knowledge of the in-situ soils was used in conjunction with earthquake-response predictions in the most recent state-of-the-art prediction procedures. Problems encountered in making liquefaction evaluations are discussed in some detail. These problems include predicting the magnitude and recurrence rates (frequency) of ground motions to be expected, determining the true nature of the subsurface soils, and finally evaluating the effect of the applied ground motions on these subsurface soils. This report concludes that although most of North Island is underlain by natural sands which should be fairly resistant to liquefaction, limited regions with very high liquefaction potential exist. These regions would be expected to liquefy under earthquake levels used for engineering analysis, and present a high damage potential to such critical structures as the carrier docking facilities, aviation fuel tank farms, and service lines.

● **5.2-9** Pratt, H. R. *et al.,* **Experimental and analytical study of the response of earth materials to static and dynamic loads,** *AFWL-TR-75-278,* Terra Tek, Inc., Salt Lake City, May 1976, 82. (NTIS Accession No. AD-A027 094)

A field and laboratory experimental test program was conducted at the Mixed Company (Colorado) and Cedar City (Utah) test sites to determine the effect of specimen size on the strength and deformation of intact and jointed rock. The strength of Cedar City quartz diorite decreased by a factor of four with increasing specimen size; the Kaventa sandstone decreased by a factor of less than two. A large block test was conducted at the Mixed Company site in which deformation and velocity were measured as a function of stress. The in-situ stress conditions were measured at the Mixed Company site and had a marked effect on the seismic wave velocity in the sandstone. In addition, in-situ modulus was measured by several different techniques. A constitutive model for jointed rock incorporating real data was developed for use in the calculation of rock mass response. The model for the compressive properties consists of a rate-dependent model of the Maxwell type while the plastic shear response due to the presence of joints is accounted for in terms of additional plastic deformation associated with slip on the joint planes.

● **5.2-10** Sherif, M. A. and Ishibashi, I., **Dynamic shear moduli and damping ratios for dry sands,** *Soil Engineering*

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Research Report No. 15, Dept. of Civil Engineering, Univ. of Washington, Seattle, Oct. 1976, 56.

Using the torsional simple shear device, the authors investigated the equivalent dynamic shear moduli and the damping ratio for Ottawa sand, Del Monte sand, Golden Gardens and Seward Park sands. Based on their experimental findings, the authors propose two equations relating shear moduli to shear strain (below and above 0.03%) as a function of the soil angle of internal friction and the effective confining pressure, and also propose an equation relating the damping ratio to shear strain as a function of the confining pressure, soil sphericity, and coefficient of soil gradation. Relationships are also presented that consider the effects of the number of stress cycles on equivalent shear moduli and damping ratios. Nomographs are shown to assist in the easy determination of the moduli and the ratios for dry sands. Comparisons are made between the authors' findings and the results of previous researchers.

● **5.2-11** Hall, Jr., J. R., Shukla, D. K. and Kissenpfennig, J. F., **Shear stress distribution due to shear and Rayleigh wave propagation at deep soil sites,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(a), Paper K 1/15, 9. (For a full bibliographic citation, see Abstract No. 1.2-5.)

Earthquake strong motion at ground surface may be described as the combined result of body (compression and shear) waves and surface waves. In the course of earthquake engineering site response analyses, it is usually assumed that surface motions are a result of vertically propagating body waves. In studies of liquefaction potential, it is assumed that soil shear strains are a result of vertically propagating shear waves. However, shear strains are also caused by surface Rayleigh waves and solutions to strains from each source are derived and compared in the paper. The results for shear strain in terms of surface acceleration and wave length are presented in nondimensional form.

The results of this investigation show that the distribution of soil shear strain as a function of depth below the ground surface is different depending upon whether shear waves or Rayleigh waves are assumed to be the source of horizontal motions at the ground surface. It is shown that the shear strains derived from Rayleigh waves are greater near the surface but decrease faster with depth than the shear strains derived from vertically propagating shear waves. It is noted that the more rapid attenuation of shear strains with depth as predicted by Rayleigh wave theory is more consistent with observed cases of soil liquefaction which are usually limited to shallow depths, generally not in excess of 10 to 20 m. On the contrary, vertical shear propagation theory predicts liquefaction occurring at loose sand sites at much greater depths. The differences are significant enough to influence the decision regarding liquefaction potential of a soil site.

● **5.2-12** Huang, W., Gupta, D. C. and Agrawal, P. K., **Application of method of characteristics in seismic analysis,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(a), Paper K 1/12, 12. (For a full bibliographic citation, see Abstract No. 1.2-5.)

In the seismic analysis of nuclear power plants, ground response analyses are often essential. For this purpose, one-dimensional analyses are generally performed. To account for the nonlinear nature of soils in these analyses, an equivalent linear approach is widely used. In this approach, an iterative scheme is employed. In each iteration, the soil modulus and damping values are selected corresponding to a fraction of the peak strain and then remain constant throughout the earthquake duration in that iteration. The final soil properties are obtained after a satisfactory convergence is achieved. This iterative procedure has been found to give adequate results in many cases; however, it causes a few problems sometimes, especially for a strong input motion and for high frequencies. It is possible that part of this problem may be corrected if a truly nonlinear analysis is used. For this purpose, several different methods can be used. In this paper, the application of the method of characteristics is evaluated and the results are compared with those obtained from the equivalent linear approach. The soil profile is modeled by a horizontally layered semi-infinite medium and the seismic excitation is introduced at the rock-soil interface. It is assumed that the soil profile is subjected only to the vertically traveling shear waves. In the nonlinear analysis, the soil stress-strain characteristics are represented by a Ramberg-Osgood relationship.

The comparison of results shows that the equivalent linear method consistently gives higher maximum shear stresses and accelerations at all depths within the soil profile. The comparison of shear stress time history at various layers, however, shows that the shear stresses from nonlinear and equivalent linear methods yield comparable equivalent uniform stresses. It is also found that the method of characteristics gives higher surface response spectrum at high frequencies and lower spectrum at lower frequencies than the equivalent linear method. In addition, the comparison of results from the two methods shows that the difference in numerical results depends to a great extent on the intensity of excitation. For example, it is found that the results obtained for moderate shaking using the equivalent linear method are only slightly different from those obtained by the nonlinear approach. However, the discrepancy in the results obtained from the two methods increases with the intensity of excitation.

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A study is also conducted to determine the effects of adding viscous damping to the hysteretic damping in the two analyses. It is found that the viscous damping has no effect on the maximum values of shear stresses in the nonlinear analysis. However, the surface acceleration response spectrum at high frequencies is substantially reduced. On the other hand, for the equivalent linear method, the addition of viscous damping has a negligible effect on either the maximum shear stresses or the surface acceleration response spectrum.

● **5.2-13** Ishibashi, I., Sherif, M. A. and Tsuchiya, C., **Pore-pressure rise mechanism and soil liquefaction,** *Soils and Foundations,* 17, *2,* June 1977, 17–27.

Based on undrained cyclic shear experiments conducted on a saturated Ottawa sand in a torsional simple shear device, the authors propose equations that predict pore pressure rise in the above soil under uniform and nonuniform dynamic shear stresses. These equations take into consideration the stress loading history, the number of stress cycles, and the stress intensity function. There is excellent agreement between the experimental pore pressure values and the analytical predictions.

● **5.2-14** Ishihara, K., **Simple method of analysis for liquefaction of sand deposits during earthquakes,** *Soils and Foundations,* 17, *3,* Sept. 1977, 1–17.

Effects of an irregular series of earthquake-induced cyclic stresses and the K_0-value at the time of consolidation on the liquefaction resistance of sand are discussed by examining the results of several series of cyclic triaxial tests that have been performed at the Univ. of Tokyo. Relationships are established in a graphic form between the applied shear stress and the residual pore water pressure, that is, the pore water pressure which remains after the application of irregular time histories of shear stress. On the basis of these relationships, with the effects of irregularity of wave form and the K_0-value incorporated, a practical method is proposed to evaluate pore water pressures and the factor of safety against liquefaction that can develop in a horizontal deposit during an earthquake. The method is applied to analyze the liquefaction of a sand deposit in Niigata where a detailed investigation was recently carried out for cyclic strength of undisturbed samples.

● **5.2-15** Iwasaki, T. and Tatsuoka, F., **Effects of grain size and grading on dynamic shear moduli of sands,** *Soils and Foundations,* 17, *3,* Sept. 1977, 19–35.

Resonant column tests were performed to clarify the effects of grain size distribution on shear moduli of normally consolidated disturbed sands for shear strain ranging from 10^{-6} to 10^{-4}. The sands tested include clean, uniform sands without fine particles; natural, nonuniform sands with fine particles; and other artificially graded sands. The test results indicate the following. Shear moduli of the clean sands are expressed approximately by similar empirical equations irrespective of grain shape and size. For other-than-clean sands, however, shear moduli decrease with the increase in the uniformity coefficient and also decrease with the increase in the content of fine particles. It is concluded that, in estimating the shear moduli of natural, nonuniform sands with fine particles, it is preferable to conduct resonant column tests on the sands of interest. When tests are not conducted, the effects of grain size distribution on shear moduli should be taken into account.

● **5.2-16** Prevost, J.-H., **Mathematical modelling of monotonic and cyclic undrained clay behaviour,** *International Journal for Numerical and Analytical Methods in Geomechanics,* 1, *2,* Apr.-June 1977, 195–216.

The proposed general analytical model describes the anisotropic, elastoplastic, path-dependent, stress-strain-strength properties of inviscid saturated clays under undrained loading conditions. The model combines properties of isotropic and kinematic plasticity by introducing the concept of a field of plastic moduli, which is defined in stress space by the relative configuration of yield surfaces. For any loading (or unloading) history, the instantaneous configuration is determined by calculating the translation and contraction (or expansion) of each yield surface. The stress-strain behavior of clays can thus be determined for complex loading paths and, in particular, for cyclic loadings. The stress-strain relationships are provided for use in finite element analyses. The model parameters required to characterize the behavior of any given clay can be derived entirely from conventional triaxial or simple shear soil test results. The model's extreme versatility is demonstrated by using it to formulate the behavior of the Drammen clay under both monotonic and cyclic loading conditions. The parameters are determined by using solely the results from monotonic and cyclic strain-controlled simple shear experimental tests, and the model's accuracy is evaluated by applying it to predict the results of other tests such as (1) cyclic stress-controlled simple shear tests, (2) monotonic triaxial loading compression and unloading extension tests, and (3) cyclic stress- and strain-controlled triaxial tests on the same clay. The theoretical predictions are found to agree extremely well with the experimental test results.

● **5.2-17** Goldsmith, W., Sackman, J. L. and Taylor, R. L., **Property determination and wave propagation in a block of Barre granite,** *Bulletin of the Seismological Society of America,* 67, *1,* Feb. 1977, 87–102.

The principal axes of a 666.8 by 609.6 by 489.0 mm (26 1/4 in by 24 in by 19 1/4 in) block of Barre granite, treated as an orthotropic elastic material, were determined from measured pulse velocities along directions connecting 160 pairs of surface points, encompassing the entire spectrum of possible orientations. The elastic moduli of the

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rock were ascertained by Hopkinson bar tests involving rods cored from other samples along their principal directions; this was required for the execution of a wave propagation analysis in the block treated as a halfspace.

Construction and insertion techniques were developed for transducers to be embedded in the rock at 14 locations. External and internal calibration procedures were devised to permit interpretation of the data transmitted from the interior of the sample. Transients in the block were generated by the impact of 6.35-mm (1/4 in.) diameter steel spheres on loading bars sandwiching a thin quartz disk, serving as an input transducer, against the specimen. The wave patterns sensed by the transducers were displayed and photographed on oscillographic screens.

A finite element program capable of handling arbitrary anisotropy was developed and employed for comparing the experimental results with analytical predictions based on the measured input as the boundary condition. For those stations where computations were performed, the correlation ranged from good to qualitative. It is concluded that better transducer embedment and in-situ calibration techniques are required for internal transducers used in hard rocks of this type.

● **5.2-18** Gamer, U., **Dynamic stress concentration in an elastic half space with a semi-circular cavity excited by SH waves,** *International Journal of Solids and Structures,* **13,** *7,* 1977, 675–681.

The subject of the investigation is the stress distribution and the dynamic stress concentration factor at the surface of a semicircular cavity in a halfspace excited by plane harmonic SH waves. Using wave-function expansion for the incident wave and the reflected waves, a closed-form solution is obtained. Numerical results are represented graphically.

5.2-19 Lapin, S. K., **On the determination of the dynamic stiffness properties of natural soil bases** (Ob opredelenii dinamicheskikh kharakteristik zhestkosti estestvennykh osnovanii, in Russian), *Osnovaniya, fundamenty i mekhanika gruntov, 3,* 1977, 32–34.

The effects of soil pressure, foundation depth, and size on the uniform elastic compression coefficient of foundations are investigated. The analysis is based on data obtained from dynamic testing of more than 300 experimental and existing foundations at 30 building sites. The relationships calculated are used to derive a formula for the determination of the stiffness of natural foundations based on the factors listed above. A sample calculation is given.

● **5.2-20** Yegian, M. K. and Whitman, R. V., **Soil liquefaction analysis based on field observations,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2441–2447.

Most interpretations of reported data obtained from field observations have been in terms of ground surface accelerations. This paper presents an alternate approach to the interpretation of reported case histories. The use of ground surface accelerations, which are subject to great uncertainties, has been deliberately avoided. Instead, the reported earthquake magnitudes, together with the distances to the nearest source of energy release, have been employed to evaluate the shaking at the sites of the case histories. A criterion is recommended which relates the possibility of liquefaction and soil density expressed in terms of corrected blow counts. Quantitative measures of uncertainties, both in the criterion and the parameters which define the criterion, are also presented in this paper.

● **5.2-21** Chae, Y. S. and Au, W. C., **Dynamic properties of expansive soils treated with additives,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2328–2333.

An experimental study was conducted to evaluate the dynamic response of expansive clay soils treated with lime, salt, and a lime-salt combination. Dynamic properties, in terms of dynamic shear modulus and damping, were determined by the resonant column technique. The test parameters studied were confining pressure, shear strain amplitude, moisture content, type of treatment, and treatment level. A total of 74 specimens were tested. With various combinations of the test parameters considered, 1500 tests were conducted. The results show that the use of additives to stabilize expansive soils subjected to earthquake or other forms of vibratory loading is very effective, for the additives increase the rigidity and energy dissipation characteristics of the soils.

● **5.2-22** Troncoso, J. H., Brown, F. R. and Miller, R. P., **In situ impulse measurements of shear modulus of soils as a function of strain,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2316–2321.

In-situ determinations of shear moduli at different strain levels have been accomplished by impulse-generated shear wave velocity measurements. The results show that soil deposits of different geologic origin respond with somewhat different rates of reduction in modulus at increasing levels of strain. These differences indicate the need for in-situ measurements for obtaining this important soil parameter for many earthquake engineering projects. Preliminary tests using the standard penetration test as an alternate source of energy show results consistent with other data.

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● **5.2-23** Richart, Jr., F. E., Anderson, D. G. and Stokoe, II, K. H., **Predicting in situ strain-dependent shear moduli of soil,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2310-2315.

Shear moduli evaluated from laboratory tests must be modified to approximate in-situ moduli because of factors associated with laboratory testing procedures. The form of the moduli adjustment is primarily related to the time-dependent behavior of reloaded soils subjected to constant stress conditions. Because time effects occur at all shearing strain amplitudes up to 1.0%, it appears that in-situ moduli can be best predicted by increasing laboratory moduli by a constant amount equal to the difference between moduli measured at low-strain amplitudes by laboratory and geophysical methods.

● **5.2-24** Eskin, Ju. M. and Eisler, L. A., **Plane vibrations of saturated soil in structural foundation,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2350-2355.

Presented is an algorithm for the finite difference solution to the problems of one-dimensional vibrations of a saturated soil layer. The problems of investigating dynamic vibrations and stresses are formulated for two cases: (1) a layered saturated subsoil underlying a foundation for equipment with a vibratory load, (2) a soil layer subjected to a seismic impact directed downward from the rigid foundation. Examples are given and the results are discussed. Numerous problems of practical importance involving the experimental or analytical dynamic stability evaluations of the subsoils and earth structures are solved with the aid of the analysis of the dynamic loading conditions and the strains in these soils. For the saturated soils, the solutions to such problems are performed using a system of differential-integral equations.

● **5.2-25** Sherif, M. A., Ishibashi, I. and Tsuchiya, C., **Influence of pore-pressure coefficient B on soil liquefaction potential,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2175-2182.

Liquefaction potential of loose Ottawa sand was investigated for several values of pore-pressure coefficients ranging from 0.25 to near unity. Experiments were conducted using the Univ. of Washington dynamic torsional simple shear device, which applies nearly uniform cyclic shear strains throughout the tapered ring-shaped soil specimen.

● **5.2-26** Ishihara, K. and Takatsu, H., **Pore pressure buildup in initially sheared sand subjected to irregular excitation,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2163-2168.

Dynamic triaxial torsion shear tests were performed on saturated sand samples subjected initially to horizontal shear stress. Torsional loads with irregular time histories were applied to the samples either under conditions of lateral confinement or under constant lateral pressures. In addition, irregular loads were applied either with their maximum peaks oriented towards or opposite to the direction of initial horizontal shear. The effects of these loading conditions on the pore pressure buildup characteristics of sand are discussed.

● **5.2-27** Tanimoto, K., **Evaluation of liquefaction potential of sandy deposits by a statistical method,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2201-2206.

A method for evaluating the liquefaction potential of sandy deposits is proposed. The expression for prediction of liquefaction occurrence is derived using a statistical treatment of basic data for known cases of liquefaction and for cases of no liquefaction. The examination of this expression shows that it can discriminate the cases of liquefaction from those of no liquefaction, not only in the basic data, but in other data.

● **5.2-28** Nandakumaran, P., Bhargava, S. and Mukerjee, S., **Liquefaction potential of dense sand surrounded by loose sands,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2207-2212.

The paper describes a procedure for computing pore pressures developed in saturated cohesionless soils at different stages of ground motion caused during an earthquake. The procedure takes into account the reduction in confining pressures caused by hydraulic gradients set up in heterogeneous deposits, and hence, can be used for determining the liquefaction potential of dense sand surrounded by loose sands. The paper also describes results of shaking table tests conducted to determine the validity of some assumptions in the analytical procedure.

● **5.2-29** Bhandari, R. K. M., **Countering liquefaction by subsoil densification at a refinery complex in Assam, India,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2213-2219.

The methods of compaction piling and dynamic consolidation were used to densify the subsoil at a refinery complex in Assam. The basic requirement of each method was to ensure 80 percent relative density throughout the

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compacted area. The effectiveness and limitations of each method are discussed in this paper.

● 5.2-30 Sharma, M. G. and Wang, M. C., **Rheological properties of a cohesive soil,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2220–2225.

It is generally accepted that ground motions generated by earthquakes impose severe inertial forces on structures that are periodic in nature. These forces in turn may influence the load-carrying capacity of the soil underneath structures. For a rational evaluation of soil-structure interaction, a complete knowledge of the constitutive relations for a given soil is imperative. This paper is concerned with an experimental investigation for studying the rheological properties of a cohesive soil. Creep and creep recovery tests, stress relaxation, stress-strain tests at various strain rates ranging from 0.03 to 8 per sec, and dynamic tests with repeated application of triangular stress pulses were conducted. From the results, a constitutive relation representing the elastic, viscoelastic, and viscoplastic response of the material is obtained.

● 5.2-31 Chou, I. H. and Oguntala, A., **Likelihood of liquefaction,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2183–2188.

Traditionally, a deterministic approach is utilized in liquefaction studies. Recently, a stochastic approach was proposed to study the liquefaction problem by assuming a probable shear stress envelope distribution function and simulating the liquefaction phenomena by fatigue behavior in material. Also, a discriminant analysis technique was applied to evaluate the liquefaction potential by using the available data for relative densities and corrected accelerations. Both studies utilize the probabilistic approach to determine the likelihood of liquefaction, leaving unsolved the question of how probable it is that this condition will occur. This paper will study the liquefaction potential based on the findings of the apparent scatter in correlating the soil properties and the liquefaction. A probabilistic assessment will be applied to each contributing parameter.

● 5.2-32 Youd, T. L. and Hoose, S. N., **Liquefaction susceptibility and geologic setting,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2189–2194.

Liquefaction-induced ground failures that occur during earthquakes are confined to specific geologic settings. An analysis of published earthquake reports shows that shallow, saturated, Holocene fluvial, deltaic and aeolian deposits, and poorly compacted artificial sand fills have highest susceptibilities to liquefaction and ground failure. Generally smaller susceptibilities are found in Holocene alluvial-fan, alluvial-plain, beach, terrace, and playa deposits. Pleistocene sand deposits are generally even less susceptible, and glacial till, clay-rich, and pre-Pleistocene deposits are usually immune to liquefaction.

● 5.2-33 Franciosi, V., **A three-spherical model: an approach to the dynamic study of regular coherent soil,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2280–2284.

A coherent soil constituted by elements whose dimensions are not particularly variable is considered. By idealizing this soil in a set of spherical elements joined by elastoplastic braces, an acceptable Mohr curve is drawn. The model permits the obtaining of a bond between the frequency and the amplitude of a sinusoidal motion, i.e., the conditions under which the cohesion disappears.

● 5.2-34 Esashi, Y., Yoshida, Y. and Nishi, K., **An exploratory method for dynamic properties of ground through borehole wall,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2297–2302.

To investigate the dynamic properties of soils, the authors have developed new equipment for in-situ testing, which can generate dynamic pressure on a borehole wall through a rubber tube at an arbitrary depth with a wide range of radial strain amplitude. The following information is included: (1) the mechanism and functioning of the equipment and (2) test results using the equipment, including the dynamic deformation modulus, the equivalent viscous damping coefficient, and the horizontal dynamic coefficient of soil reaction.

● 5.2-35 Iwasaki, T. and Tatsuoka, F., **Dynamic soil properties with emphasis on comparison of laboratory tests and field measurements,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2303–2309.

To analyze ground motions during earthquakes, it becomes necessary to obtain strain amplitude-dependent shear moduli and damping coefficients of soils. At two sites, in-situ seismic surveys were carried out. Sand samples from these sites were tested with a resonant-column apparatus to obtain shear moduli at small strain. Laboratory test results showed that these two natural sands have smaller shear moduli than uniform clean sands. The comparison of shear moduli from shear wave velocities and those from resonant-column tests was made, and an excellent agreement was obtained for both sites.

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● 5.2-36 Werner, S. D. and Van Dillen, D., **Use of analytical and statistical techniques to assess in-situ soil test procedures,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2291–2296.

A new field testing procedure has been developed to measure strain-dependent in-situ shear moduli. Part of the development effort has used nonlinear finite element analyses to evaluate data processing techniques for obtaining shear moduli and strains from the soil motions generated during the test. In addition, statistical techniques have been applied to results from special field tests to determine the sensitivity of the generated soil motions to various test parameters that may influence results from the in-situ soil test.

● 5.2-37 Holzlohner, U., **Earthquake induced residual settlements of foundations,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1529–1535.

Model foundations were dynamically loaded to investigate residual settlements induced by earthquakes. The settlements increase logarithmically with the number of load cycles. Other variables considered are contact area of the foundation, static and dynamic load, frequency, void ratio, and moisture content of a well-graded gravelly sand. The settlements to be expected as a result of earthquakes can be found by means of model tests.

● 5.2-38 Shikhiev, F. M. and Jakovlev, P. I., **Calculation of bearing capacity for the foundations subjected to seismic loads,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1664–1669.

A method for calculation of the bearing capacity of a foundation is developed based on the safe stress state theory of loose media suggested by V. V. Sokolovsky and S. S. Golushkevich. An inclined failure load is calculated as a function of dimensionless bearing capacity coefficients. For the calculation of slope stability, the expressions containing dimensionless coefficients are obtained. These coefficients help to evaluate the inclined load on the second side plane which corresponds to the safe stress state of an earth mass, provided the inclined uniform load on the first side plane and the direction of the inclined load on the second side plane are known.

● 5.2-39 Becus, G. A., **A uniqueness theorem in nonlinear viscoelasticity,** *International Journal of Non-Linear Mechanics,* **12,** *3,* 1977, 147–152.

A uniqueness theorem for viscoelastic media obeying a strain and time hardening nonlinear constitutive equation of the power-law type is proven.

● 5.2-40 Nandakumaran, P. and Puri, V. K., **Evaluation of dynamic shear modulus at a site,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2385–2387.

● 5.2-41 Mori, K., Seed, H. B. and Chan, C. K., **Influence of sample disturbance on sand response to cyclic loading,** *UCB/EERC-77/03,* Earthquake Engineering Research Center, Univ. of California, Berkeley, Jan. 1977, 59. (NTIS Accession No. PB 267 352)

One of the major developments in the evaluation of the liquefaction characteristics of sand deposits has been the recognition that these properties are influenced not only by the density of the deposit but also by such factors as the structure of the sand grains, the seismic history of the deposit, the coefficient of lateral earth pressure for the in-situ conditions, and the age of the deposit. Accordingly, it is necessary to obtain and test undisturbed and representative samples if meaningful evaluations of in-situ performance are to be made on the basis of laboratory tests. This report presents the results of a study of sample disturbance during extraction and handling on the liquefaction characteristics of a sand having an artificially induced increased resistance to liquefaction as a result of the application of a prescribed prior strain history. It is shown that the effects of this strain history are, for practical purposes, obliterated by the sampling and handling procedure; and suggestions are made for assessing the significance of such effects in the practical evaluation of laboratory test results.

● 5.2-42 Dasgupta, G. and Chopra, A. K., **Dynamic stiffness matrices for homogeneous viscoelastic halfplanes,** *UCB/EERC-77/26,* Earthquake Engineering Research Center, Univ. of California, Berkeley, Nov. 1977, 121. (NTIS Accession No. PB 279 654)

Analytical expressions and numerical results are presented for the complex-valued, dynamic (frequency dependent), flexibility influence coefficients for a homogeneous, isotropic, linearly viscoelastic halfspace in plane strain or generalized plane stress. These influence coefficients, defined for uniformly spaced nodal points at the surface of the halfspace, are obtained from solutions of two boundary value problems associated with harmonically time-varying stresses uniformly distributed between two adjacent nodal points. Numerical values for these coefficients are presented for a viscoelastic halfspace of constant hysteretic material. A method is developed to determine from these results the dynamic stiffness matrix associated with the nodal points at the base of a surface-supported structure for the halfspace. The resulting dynamic stiffness matrix is shown to be superior compared to the one determined from an available procedure which is based on solutions of displacement boundary-value problems for the halfspace.

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● 5.2-43 Flores-Berrones, R., **A theoretical approach for computing radiation damping in end bearing pile foundations,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1583-1589.

The radiation damping and the effective mass of soil are obtained by using the theory of flexible piles embedded in an elastic stratum. The analysis is similar to that used in the halfspace theory for a rigid plate vibrating vertically on the surface. The study indicates the existence of large values of horizontal radiation damping for frequencies above the fundamental soil frequency and a zero value for frequencies below it.

● 5.2-44 Casagrande, A., **Liquefaction and cyclic deformation of sands: a critical review,** *Soil Mechanics Series No. 88,* Harvard Univ., Cambridge, Massachusetts, Jan. 1976, 55. (Presented at 5th Panamerican Conference on Soil Mechanics and Foundation Engineering, Buenos Aires, Nov. 1975.)

Until 1966, the term liquefaction was used in literature for the response of saturated loose sand to strains or shocks that resulted in flow slides. This paper reviews investigations of, and differentiates between, two phenomena which have previously been identified as liquefaction. The term "actual liquefaction" is used to define the response of contractive (loose) sand that leads to loss of strength which can cause flow slides; and the term "cyclic liquefaction" is used for the response of dilative (dense) sand when subjected to cyclic laboratory tests.

5.2-45 Holloway, D. C., Patacca, A. M. and Fourney, W. L., **Application of holographic interferometry to a study of wave propagation in rock,** *Experimental Mechanics,* 17, *8,* Aug. 1977, 281-289.

Holographic interferometry was utilized to determine the three orthogonal components of displacement in elastic surface waves. A pulsed ruby laser was used as the light source, and techniques to improve its coherence properties are described. Procedures for the formation and reconstruction of the hologram, fringe interpretation and data reduction, and presentation are detailed. The elastic-wave velocities and material constants for pink Westerly granite were obtained. Solutions for an explosively generated Rayleigh wave in a halfspace and its reflection from a free edge are presented.

5.2-46 Grady, D. E. *et al.*, **Strain rate dependence in dolomite inferred from impact and static compression studies,** *Journal of Geophysical Research,* 82, *8,* Mar. 10, 1977, 1325-1333.

Plate impact techniques and laser interferometry instrumentation were used to obtain continuous one-dimensional compression wave data through states of loading and unloading on Blair dolomite in the stress range of 0-6.5 GPa. These data are used to infer the dynamic stress-strain response. Both the character of the wave propagation and a comparison with quasistatic uniaxial strain data obtained by other workers suggest substantial strain-rate dependence in this rock. The dynamic yield stress of 2.5 GPa determined from the wave propagation data was approximately a factor of 10 higher than that observed in the quasistatic data. Wave propagation below 2.5 GPa is suggestive of viscoelastic response, and a nonlinear Maxwell constitutive equation fitted to these data provides a characteristic relaxation time of 0.01 μs. Comparison of dynamic and quasistatic loading data suggests that the large difference in observed yield stress could be the result of a transition from failure by brittle fracture to failure by intracrystalline plastic flow.

● 5.2-47 Goldsmith, W. and Sackman, J. L., **Wave traversal and comminution of rock,** Dept. of Mechanical Engineering, Univ. of California, Berkeley, Jan. 1976, 148.

Elastic moduli and fracture strengths were obtained for Barre granite, considered to be an orthotropic elastic material, under static and dynamic conditions. Stress histories generated by surface impact on a large block of this granite were measured in the interior at selected stations by embedded crystal transducers and compared with the results of corresponding finite element analyses. Correlation ranged from excelled to qualitative, indicating the need for improved in-situ gage calibration. Finite element and finite difference models were developed for joints contained in rock masses, and numerical tests employing these programs correlated well with closed-form solutions. The comminution process produced by the impact of steel strikers on hard and soft rock was studied; in the first case, the phenomenon involved wave propagation in the target and significant ejecta, while in the second case, penetration and inelasic target deformation were predominant. The energy required to produce unit new surface was estimated for the hard rock diorite, while the behavior of soft rock, shale, under these conditions was found to be well represented by a simple viscous model.

● 5.2-48 Taga, N. and Togashi, Y., **Dynamic properties of a stratified fluid saturated soil layer,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2285-2290.

Dynamic properties of a soil layer with a microstructure are investigated by idealizing the soil, which is actually composed of soil skeletons and pore fluid, to a mixture model. A wave propagation theory in multiphase continuum mechanics is developed. This method and the finite element method are applied to a two-phase medium to

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obtain free vibration and frequency transfer properties of fluid-saturated soil layers. Wave dispersion, based on the microstructure of the ground, is shown to be one of the damping mechanisms of multiphase soil.

5.2-49 Ohta, Y. *et al.*, **Shear wave velocities in deep soil deposits—measurement in a borehole to the depth of 3500 meters and its significance** (in Japanese), *Zisin, Jounal of the Seismological Society of Japan,* **30**, *4*, Dec. 1977, 415-433.

A deep shear-wave velocity measurement was carried out down to 3500 m in the Tokyo area with use of the Iwatsuki earthquake observation well constructed by the National Research Center for Disaster Prevention of Japan. Shear waves were produced mainly by means of ordinary small explosions and supplementarily by a specially designed SH-wave generator. A set of three-component seismometers were installed in a capsule having a device clamped to the borehole wall. Sequential measurements were conducted at 16 different depths to the borehole bottom at intervals of 100-500 m. The maximum depth at which the signal was clear reached 1500 m. Because comparisons of the records between two types of seismic sources were made to the depth of 1500 m, the detection of shear events in the deeper measurements by simple detonations was easy. The obtained shear-wave velocities, with 0.2 km/sec at the ground surface are 1.3, 1.6, and 2.5 km/sec at 1000, 2000, and 3000 m in depth, respectively; and the time to the bottom is 3.2 sec. A comparison of the obtained shear-wave velocities and depths of their discontinuity shows complete agreement with known data (such as sonic log, densities and geologic section). The earthquake engineering importance of shear-wave velocity measurement for deep soil deposits is stressed based upon a rough estimation of wave amplification between ground surface and bedrock.

5.3 Dynamic Behavior of Soils and Rocks

● 5.3-1 Joyner, W. B., **A method for calculating nonlinear seismic response in two dimensions**, *Wind and Seismic Effects,* IV-18-IV-46. (For a full bibliographic citation, see Abstract No. 1.2-4.)

A method is presented for calculating the seismic response of two-dimensional configurations of soil resting on bedrock. The method, which is based on a rheological model suggested by Iwan, takes account of the nonlinear, hysteretic behavior of soil and offers considerable flexibility for incorporating laboratory data on soil behavior. An approximate treatment of the boundary conditions is employed which permits energy to be radiated into the underlying medium. Examples are shown to illustrate the method.

● 5.3-2 Valera, J. E. and Donovan, N. C., **Soil liquefaction procedures—a review**, *Journal of the Geotechnical Engineering Division, ASCE,* **103**, *GT6,* Proc. Paper 12996, June 1977, 607-625.

Various procedures used for evaluating soil liquefaction are reviewed and compared. Major emphasis is given to the equivalent uniform cycle procedure and the cumulative damage approach. Simplified versions of these two procedures are used to evaluate the factor of safety against liquefaction for 41 actual cases where liquefaction has or has not occurred during past earthquakes. Good agreement was obtained between the actual observed behavior and the analytical results. Empirical correlations developed on the basis of standard penetration test blow count data at sites that have been subjected to previous earthquakes are also mentioned. Liquefaction analyses performed for two sites using several of the methods described are presented. The results of these analyses gave similar factor of safety values. The need for careful selection of the input parameters and use of sound engineering judgment was found to be as important in liquefaction analyses as the methods used.

● 5.3-3 Liou, C. P., Streeter, V. L. and Richart, Jr., F. E., **Numerical model for liquefaction**, *Journal of the Geotechnical Engineering Division, ASCE,* **103**, *GT6,* Proc. Paper 12998, June 1977, 589-606.

A numerical model for sand deposits subjected to earthquakes is developed. The soil is idealized as a two-phase medium composed of water and a structural skeleton. The water is viscous and compressible, and its motion is described by a generalized Darcy's law. The skeleton is composed of incompressible granular solid particles, and is considered to be under constrained compression. The volume of the soil can be reduced by shearing strains. The model is composed of two interactive parts. The first part models the propagation of plane shear waves from the bedrock. The second part models the pressure wave propagation, which involves the movements of the two phases, the effective stress, and the pore-water pressure. The dependence of the constrained modulus of the skeleton upon the dynamic shear modulus and the dependence of the shearing properties upon the transient effective stress provide the coupling between the two parts of the model.

● 5.3-4 Finn, W. D. L., Lee, K. W. and Martin, G. R., **An effective stress model for liquefaction**, *Journal of the Geotechnical Engineering Division, ASCE,* **103**, *GT6,* Proc. Paper 13008, June 1977, 517-533.

The important factors affecting the dynamic response of saturated sand layers to earthquake motions are (1) the initial shear modulus in situ; (2) the variation of shear modulus with shear strain; (3) contemporaneous generation and dissipation of pore-water pressures; (4) changes in

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effective mean normal stress; (5) damping; and (6) hardening. Constitutive relations are formulated that take all these factors into account and these are incorporated into a nonlinear method for the dynamic effective stress analysis of saturated sands. The method predicts the phenomenological features of the dynamic response of saturated sand layers that commonly occur as the pore-water pressure rises in the sand during earthquake shaking. It allows the distribution of pore-water pressure and the effects that drainage and internal flow have on the location and time of liquefaction to be determined quantitatively.

● 5.3-5 Cuellar, V. *et al.*, **Densification and hysteresis of sand under cyclic shear,** *Journal of the Geotechnical Engineering Division, ASCE,* **103,** *GT5,* Proc. Paper 12908, May 1977, 399-416.

The accumulation of inelastic strain due to the irreversible rearrangement of grain configurations associated with deviatoric strains is characterized by a nondecreasing material variable, termed the rearrangement measure; this variable, in turn, forms the basis of an intrinsic time scale and other related variables termed the densification measure and distortion measure. The shear modulus is identified to be a function of the mean normal stress and the second invariant of the strain deviator. In contrast with empirical methods, the material behavior is described herein by a constitutive law that satisfies all invariance requirements of continuum mechanics; thus, this law should, in principle, be generally applicable, including the cases of nonsinusoidal loadings with varying amplitudes, general multiaxial stress states, and nonproportional stress component histories. In addition, the law automatically exhibits hysteretic damping and is fully continuous, i.e., it contains no inequalities, such as those used in plasticity to distinguish unloading.

● 5.3-6 Seed, H. B., Mori, K. and Chan, C. K., **Influence of seismic history on liquefaction of sands,** *Journal of the Geotechnical Engineering Division, ASCE,* **103,** *GT4,* Proc. Paper 12841, Apr. 1977, 257-270.

It is shown both analytically and experimentally that deposits of sand subjected to low-magnitude earthquakes that are not sufficiently strong to cause liquefaction will develop an increased resistance to liquefaction in subsequent earthquakes even though they may undergo no significant change in density. Accordingly, in order to determine the liquefaction characteristics of a sand, it is necessary to perform tests on samples having the same density and structure as the in-situ material and conduct the tests, whenever possible, with the correct in-situ value of K_0. The study also indicates that the standard penetration resistance (or any in-situ measure of penetration resistance) is likely to provide a reasonable index of the liquefaction characteristics of a saturated sand deposit. Available data on field performance have been summarized to develop a correlation between penetration resistance and the cyclic stress ratio at which liquefaction has been found to occur in the field.

● 5.3-7 Baladi, G. Y. and Rohani, B., **Development of an elastic-plastic constitutive relationship for saturated sand,** *Liquefaction Potential of Dams and Foundations, Report 3, Research Report S-76-2,* Soils and Pavements Lab., U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, Feb. 1977, 56.

This report describes the initial attempt at developing a three-dimensional constitutive relationship for saturated sand. An isotropic constitutive relationship has been constructed, within the framework of the incremental theory of plasticity, that qualitatively simulates some of the features of the stress-strain-pore pressure response of saturated sand. It accounts for the hysteretic behavior of pressure-volumetric strain response of sand, the effect of superimposed hydrostatic stress on shearing response, and the shear-induced volume change. It does not, however, treat strain-softening behavior and does not predict progressive increase of pore pressure under low-amplitude (subyield) cyclic loading conditions. The behavior of the constitutive relationship under conventional undrained triaxial test condition is examined and correlated with experimental data for saturated Reid Bedford model sand.

A recommendation is made that the present constitutive relationship be extended to quantitatively simulate the basic features of the stress-strain-pore pressure response of saturated sand, including the strain-softening behavior and the progressive increase of pore pressure observed under cyclic loading conditions.

● 5.3-8 Ghaboussi, J. and Dikmen, S. U., **LASS-II, computer program for analysis of seismic response and liquefaction of horizontally layered saturated sands,** *UILU-ENG-77-2010,* Univ. of Illinois, Urbana, June 1977, 97.

Presented in this report is a general method for analysis of seismic response and liquefaction of horizontally layered saturated soils. The saturated soil below the water table is modeled as a coupled two-phase medium with solid granular skeleton and pore water as the constituent materials. Above the water table, soil is modeled as a one-phase solid. A nonlinear material model was used in the analysis which includes yielding, failure, volume change characteristics, cyclic effects, and criteria for initial and final liquefaction. Under cyclic shear stress, the effective pressure decreases; and, as a result, the resistance and the shear strength of sand decrease, allowing larger shear strains to develop. If this process continues under sustained shear stress, the sand will approach a condition of "near failure." This condition is considered as initial liquefaction. Following initial liquefaction, sand is in a state of post-failure and the behavior is quite different than the behavior of sand

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prior to initial liquefaction. A post-initial liquefaction model is developed for this range up to the final liquefaction. This method of seismic analysis is applied in some case studies. Results appear to be reasonable and in agreement with field observations.

● 5.3-9 Van Eekelen, H. A. M., **Single-parameter models for progressive weakening of soils by cyclic loading,** *Géotechnique,* **XXVII,** *3,* Sept. 1977, 357-368.

One of the basic problems which arises in an evaluation of the stability of gravity structure foundations under severe earthquakes and storms is the modeling of soil behavior under random cyclic loading. Phenomena such as progressive weakening, cumulative generation of pore pressures and permanent strains, and ultimate liquefaction have to be described in terms of the complete cyclic loading history. Many different procedures have been proposed, most of which can be interpreted as models with one single memory parameter. In this paper the underlying assumptions are made explicit, and a general single-parameter theory is formulated and applied to compare and catalog existing models. The interrelation of stress- and strain-controlled tests is discussed, and some new methods for obtaining or testing empirical models are indicated. Special attention is paid to the question of consistency of the model as regards linear superposition and reordering of blocks of cycles.

● 5.3-10 Yoshimi, Y. and Tokimatsu, K., **Settlement of buildings on saturated sand during earthquakes,** *Soils and Foundations,* **17,** *1,* Mar. 1977, 23-38.

Shaking table tests were conducted on models of rigid structures placed on saturated sand in order to study the pore pressure development in the sand near the structure and to study the factors which influenced the settlement of the structure. The settlement records of reinforced concrete buildings during the Niigata earthquake of 1964 were reviewed for comparison with the model test results. The studies showed: (1) The excess pore pressure developed below the center of the model structure was smaller than that away from the structure. (2) The ratio of the excess pore pressure to the initial effective stress below the center of the model structure decreased as the structure became heavier. (3) For both the model and the prototype, the settlement of the structure decreased as the width of the structure increased for a given depth of liquefaction. (4) A pair of rigid walls embedded on both sides of the model structure had a considerable effect on reducing the excess pore pressure below the structure and a marked effect on reducing the settlement of the structure.

● 5.3-11 Khosla, V. K. and Singh, R. D., **Mechanical characterization of elastic-plastic work-hardening soils,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2373-2374.

Laboratory investigations were conducted to determine soil response under a variety of stress paths for both static and cyclic loading conditions. The results are compared with predictions by various theoretical models presently employed for static loading conditions. This paper considers one such model and its application to test results on Ottawa sand and grundite.

● 5.3-12 Kuribayashi, E., Iwasaki, T. and Tatsuoka, F., **A history of soil liquefaction in Japan,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2448-2454.

In order to obtain a relationship between the occurrence of liquefaction and site conditions, a literature survey of liquefaction occurring during earthquakes in the last century in Japan was made; and site conditions at the liquefied sites were investigated. It was found that liquefaction of subsoils was induced during at least 44 earthquakes since 1872. Liquefied sites were limited to alluvial deposits and reclaimed lands along seas or lakes. Seismic intensity at the liquefied sites was estimated to be 5 or higher on the JMA scale.

● 5.3-13 Tamura, C., Noguchi, T. and Kato, K., **Earthquake observation along measuring lines on the surface of alluvial soft ground,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 389-394.

Two kinds of earthquake observations were carried out to investigate the planar behavior of alluvial ground during earthquakes. One observation was conducted using several seismographs installed on orthogonal measuring lines 500 m and 200 m in length established on the ground surface of filled land. The other was conducted using a measuring line inside a steel shield tunnel in alluvial ground and a measurement point at the ground surface. The paper describes the propagation of earthquake motions in the depth direction and directions along the ground surface based on correlations of the records taken at the various measurement points.

● 5.3-14 Toki, K. and Sato, T., **Seismic response analysis of surface layer with irregular boundaries,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 409-415.

A semi-infinite halfspace consisting of the base ground is considered in the seismic response analysis of a near-surface ground. A discrete boundary equation, which is deduced from an integral equation, is used in the analysis. Illustrative examples are given to indicate the applicability of the method and to examine the accuracy of a numerical computation program.

● 5.3-15 Ichihara, M., Matsuzawa, H. and Kawamura, M., **Earthquake resistant design of quay walls,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1969–1974.

In order to investigate the stability of quaywalls, the authors measured the dynamic active earth pressures of dry sand and submerged sand using a large vibrating soil bin. Earth pressures and pore water pressures during an earthquake were analyzed, and these pressures satisfied the conditions determined from the experiment. In the analysis of earth pressures, it was assumed that the backfill was composed of a lower layer having a static angle of internal friction and an upper layer having a dynamic angle.

5.3-16 Nikitin, L. V. and Ryzhak, Ye. I., **Regularities of rock fracture with internal friction and dilatancy,** *Physics of the Solid Earth,* **13,** *5,* 1977, 318–327.

The regularities of rock fracture with internal friction and dilatancy are studied. Fracture is interpreted as a manifestation of internal instability inherent in the constitutive equation of the material. The energy criterion of instability that takes into account the boundary conditions is used as a failure criterion. It is shown that the partial or total localization of deformations is caused by the constraining effect of the boundary conditions. The orientation of the localization layers and their number are determined as a function of the constraint.

5.4 Dynamic Behavior of Soil and Rock Structures

5.4-1 Akmatov, S. A., **Experimental investigations of root shaped piles under static and seismic type dynamic loads** (Eksperimentalnye issledovaniya kornevidnykh nabivnykh svai na staticheskie i dinamicheskie (tipa seismicheskikh) nagruzki, in Russian), *Trudy Frunzenskogo politekhnicheskogo instituta, 87,* 1976, 127–135.

Results of experimental investigations of the effects of static and dynamic loads on root shaped piles are presented. Both laboratory and field testing were carried out in sedimentary soils. The increased carrying capacity and earthquake resistance of piles in silt is noted.

5.4-2 Barkan, D. D., Mezhevoi, G. N. and Shaevich, V. M., **Investigation of the response of pile foundations with intermediate cushion in seismic regions** (Issledovanie raboty svainykh fundamentov a promezhutochnoi podushkoi v seismicheskikh raionakh, in Russian), *Trudy NII osnovanii i podzemnykh sooruzhenii, 67,* 1976, 111–112.

The response of models and full scale pile foundations with intermediate cushions to static (both horizontal and vertical) and horizontal dynamic loads was investigated. The effects of the cushion material and its thickness, the length and cross-section of the piles and the soil conditions on the distribution of horizontal stresses among the piles were studied.

A technique is presented to calculate the response of cushioned pile foundations to vertical loadings. The parameters influencing the response are determined.

5.4-3 Stavnitser, L. R., **On the conditions leading to residual displacements of foundations subjected to seismic waves** (Ob usloviyakh obrouzovaniya ostatochnykh peremeshchenii fundamentov pri deistvii seismicheskikh voln, in Russian), *Trudy NII osnovanii i podzemnykh sooruzhenii, 67,* 1976, 205–214.

It is demonstrated that the interaction of an elastic seismic wave with the base of foundation may lead to the formation of a reflected plastic wave and cause residual soil deformations and foundation settlement. The conditions favoring the formation of a reflected plastic wave are analyzed and the duration and depth of penetration of the plastic wave are calculated. Residual strains in the foundation are investigated and stress distribution at the contact surface analyzed. The formation of plastic soil deformations leads to a substantial reduction of seismic loads as compared to the pure elastic case.

● **5.4-4** Okamoto, S., **Earthquake resistance of civil structures in Japan,** *Proceedings of U.S.-Japan Seminar on Earthquake Engineering Research with Emphasis on Lifeline Systems,* Japan Society for the Promotion of Earthquake Engineering, Tokyo, Nov. 1976, 1–4.

The author discusses the importance of investigating the seismic stability of dam foundations, mountain slopes, and railroad tracks in Japan.

● **5.4-5** Singh, M. P. and Agrawal, P. K., **A stochastic method for seismic stability evaluation of earth structures with strain dependent properties,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2363–2368.

A stepwise linear stochastic method is presented for the seismic stability analysis of earth structures with strain-dependent damping and shear moduli. In this method, the seismic design input, defined as response spectra curves, can be directly used. The technique of stochastic linearization is used to obtain the best estimates of the soil properties for their use in a linear step of the analysis. Stochastic definition of cumulative damage sustained by soil is used to evaluate safety under seismic excitations. The factors of safety at various locations of an earth dam are obtained and

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compared with those obtained by the conventional time-history analysis approach.

● 5.4-6 Sarma, S. K., **Stability analysis of earth dams and embankments during strong earthquakes,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2369–2370.

It is known that excess pore pressures generated by an earthquake play an important part in the stability of an earth dam. In the limit equilibrium method of analysis, it is possible to take this excess pore water pressure into account. The method proposed in this paper deals with effective stresses, and the pore pressures are characterized by pore pressure parameters, which are to be obtained at failure. Solution of the problem is obtained in the form of critical acceleration. The advantage of this method of solution over other methods of solution dealing with total stresses is that it determines the critical acceleration which is the acceleration required to bring the stresses on the failure surface to a state of limiting equilibrium, and not the factor of safety. Therefore, the solution can be used directly to determine the possible displacements when the applied acceleration is bigger than the critical acceleration for that surface. Examples are provided in the form of figures.

● 5.4-7 Moss, P. J. and Carr, A. J., **The seismic behaviour of river valleys,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2274–2279.

The results of a series of analyses to study the effects of a variety of valley shapes on the seismic behavior of river valleys is presented. The method of analysis uses a refined finite element formulation, and horizontal and vertical seismic accelerations can be used as input excitation to the finite element model. In addition to describing the probable seismic behavior of a range of idealized river valleys, the results of analyses of two actual valleys are also presented and discussed, with particular reference to the differential horizontal movement that could occur across the top of the valleys under the influence of two specific earthquakes.

● 5.4-8 Arya, A. S. *et al.,* **Some aspects of aseismic design of rockfill dams,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2244–2250.

The current trend in the seismic stability analysis of rockfill dams is to assess the earthquake-induced deformations in the maximum cross section of the dam assuming rigid. plastic behavior of the materials. Two probable sections of a 99 m high rockfill dam, one with a central core and the other with an inclined core, have been analyzed to obtain earthquake-induced deformations. The effect of confinement on the dynamic elastic properties of the dam material has also been considered. The results of the analysis have been compared with those observed in simulated model studies of the two sections; the results show fairly good agreement.

● 5.4-9 Ayala, G. A. and Aranda, G. R., **Boundary conditions in soil amplification studies,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2268–2273.

The usefulness and limitations of the different boundary conditions frequently used in numerical wave propagation formulations are discussed. New approximate boundary conditions for numerical determination of the response of earth masses and structures to earthquakes are proposed. The formulation is general and applicable in one-, two-, or three-dimensional problems. Models require definition of excitation in terms of a front of plane seismic waves. A modified earthquake simulation algorithm, which includes source parameters, is recommended for the purpose. Computer applications of the theory are presented using a two-dimensional finite element program.

● 5.4-10 Prater, E. G., **Response of an earth dam founded on a soil deposit to travelling seismic waves,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1393–1394.

The usual assumption made in the earthquake analysis of structures is that the spatial variation in the input motion applied at the base of the structure may be ignored. However, for structures with lengths of the same order of magnitude as the wavelengths of the significant components of the earthquake motion, the speed of the traveling wave can greatly influence the structural response. Various types of soil-structure systems have been investigated, but only the results for an earth dam are reported. The system and the material properties are shown. The underground consists of a horizontal isotropic soil layer, terminating at the sides by vertical dissipating boundaries of the type proposed by Lysmer and Kuhlemeyer. The wave motion characteristics are those of the underlying medium. In the analysis, it is assumed that no energy is returned to this medium. To solve the problem of the dynamic interaction of the dam-layer system, the method of finite differences is adopted. Details of the method are given. The equations of plane wave propagation are discretized using central finite differences in space and time. Simple relations at the boundaries and interfaces are achieved using interlacing nets for the field variables.

● 5.4-11 Takahashi, T. *et al.,* **Study on dynamic behaviors of rockfill dams,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2238–2243.

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The elastic constant of high rockfill dams recently constructed in Japan can be expressed as a function of the depth from the dam surface. The damping factor of any existing rockfill dam is several percent within a moderate strain range. Though the natural frequencies of the dams slightly decrease because of reservoir water, the earthquake behavior of rockfill dams is scarcely affected. The vibration modes in the stream direction largely depend upon the shape and the width of the valley. It is found that the rockfill dams behave linearly over a wide range.

● 5.4-12 Zadgaonkar, A. S. and Tarnekar, M. G., **Acoustic optical method of analysis of dynamic rock structure,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2899–2901.

When a plate (rock structure) is subjected to static or dynamic loads, the average stress density in the rock structure is substantially uniform over the plate, except for the pattern of strips in which there is an excess of energy. These patterns are traced using the acoustic optical method developed by the authors. The distribution of stress can be visualized on the photograph.

● 5.4-13 Bhargava, S. *et al.*, **Stability of downstream slope of Beas Dam at Pong under earthquake,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2251–2256.

A number of diagonal plastic gouged seams (shear zones) in clay shale bands were noted in the foundation rocks of the Beas earth and rockfill dam at Pong, India. A two-dimensional seismic stability analysis of the dam, with resistance of gouged seams considered, indicated the need for a large amount of material to act as toe protection. To obtain a more realistic analysis, models were tested on a tilting table. Sliding of the dam and the foundation wedges, which were bounded by gouged seams and vertical rock joints, was compared to the dip and strike of the seams. Three-dimensional stability analyses of the probable failure wedges were then conducted. A safety factor of more than 1.0 was found, indicating that toe weights were not necessary.

● 5.4-14 Watanabe, H., **A consideration on the seismic coefficients of rock and earth fill dams through observed accelerograms and model tests,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2226–2231.

Vibration experiments were performed on six sand and gravel dam models. The models were constructed in a steel testing box which was affixed to a shaking table. The behavior of the models is compared with that of actual dams, as derived from records obtained at the base of 14 dams. An empirical formula for the predominant periods of accelerograms located at the base of dams is then deduced. Next, the magnification factors of acceleration at the crest of the dams are estimated for three earthquakes of magnitude 7, 7.5, and 8, respectively. The peak acceleration at the base of the dams was only 200 gals. The mean values of these factors are 1.6 for rockfill and 1.94 for earthfill dams.

5.4-15 Sergeev, Z. A., **The static equilibrium method for calculating the earthquake response of free homogeneous slopes** (Staticheski ravnovesnyi metod rascheta svobodnykh odnorodnykh otkosov na seismostoikost, in Russian), *Osnovaniya, fundamenty i mekhanika gruntov, 3,* 1977, 34–36.

A method is presented for calculating the seismic stability of homogeneous slopes satisfying all equilibrium conditions. The method is based on the hypothesis of a cylindrical slipping surface. The two-dimensional problem is solved by reducing all forces to a single vector. The direction of the seismic force is determined on the basis of conditions corresponding to the moment of limit equilibrium in the shear region of the soil. The solution is obtained with the aid of a computer.

● 5.4-16 Migliore, M., **On the stability of a wall-checked embankment subjected to earthquake,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2257–2261.

Using a three-sphere soil model, the author verifies Franciosi's theoretical results regarding the seismic stability of an embankment wall.

● 5.4-17 Fang, H. Y. and Mehta, H. C., **Structural response of sulphur-bamboo reinforced earth mat to seismic loading,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 997–1003.

This paper presents preliminary test results of the structural response of a compacted embankment reinforced with bamboo earth mat under various seismic loadings. Observations include the cracking and failure patterns and the time required for failure. In addition, fracture toughness and tensile strength tests of the compacted soil were performed in order to correlate these values to the cracking pattern of the embankment during the seismic loading tests. It was found that the embankment reinforced with the bamboo earth mat gives better performance than nonreinforcement in all cases studied.

● 5.4-18 Savinov, O. A. and Natarius, Ja. I., **Operational means of increasing seismic stability of earth dams,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2345–2349.

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Proposed are methods for increasing the seismic stability of earth dams. The methods are based on the employment of special devices actuated during earthquakes by the signals from accelerator-starters of a seismometric network. Given are results of the feasibility of the methods and a preliminary estimation of their efficiency.

5.5 Dynamic Behavior of Foundations, Piles and Retaining Walls

● 5.5-1 Erguvanli, A., **A brief note on the pressures occurring during earthquakes** (Depremler sirasinda olusan basinclar hakkinda kisa bir not, in Turkish), *Deprem Arastirma Enstitusu bulteni*, 3, *12*, Jan. 1976, 67–68.

In this study, the types of methods used in different countries for computing the dynamic pressure caused by earthquakes acting on various structures, such as soil retaining walls, are listed.

5.5-2 Mongolov, Yu. V., **On the response of pile foundations to horizontal seismic excitation** (K raschetu svainykh fundamentov na gorizontalnye seismicheskie vozdeistviya, in Russian), *Trudy NII osnovanii i podzemnykh sooruzhenii, 67,* 1976, 86–97.

An approximation method is presented to analyze the response of piles in soil having linearly increasing stiffness with depths to simultaneous vertical and horizontal forces and bending moments. Pile systems and freely standing single piles are considered. A technique is given to calculate the coefficient of proportionality for various soil conditions.

5.5-3 Wolf, J. P., **Soil-structure interaction with separation of base mat from soil (lifting-off)**, *Nuclear Engineering and Design,* 38, 2, Aug. 1976, 357–384. (For an additional source, see Abstract No. 1.2-2.)

In reactor buildings having a separate base mat and a shield-building (outer concrete shell) of large mass, large overturning moments are developed for severe earthquake loading. The standard linear elastic half-space theory is used in the soil-structure interaction model. For a circular base mat, if the overturning moment exceeds the product of the normal force (dead weight minus the effect of the vertical earthquake) and one-third of the radius, then tension will occur in part of the area of contact, assuming distribution of stress as in the static case. For a strip foundation the same arises if the eccentricity of the normal force exceeds a quarter of the total width. As tension is incompatible with the constitutive law of soils, the base mat will become partially separated from the underlying soil.

Assuming that only normal stresses in compression and corresponding shear stresses (friction) can occur in the area of contact, a method of analyzing soil-structure interaction including partial lifting-off is derived, which otherwise is based on the elastic behavior of the soil. A rigorous procedure to determine the nonlinear impedance function of a rigid plate of arbitrary shape, only in partial contact with the elastic halfspace, is developed. Complex dynamic influence coefficients for displacements are used which can either be determined with the finite element method or based on solutions of displacements on the surface of an elastic halfspace at a certain distance from a rigid subdisk. Constant and variable stiffness methods of solving the nonlinear equations of motion are explained which also determine the area of contact. Slipping of the entire mat or of a part thereof can also be taken into consideration.

A simpler approximate method is discussed. For a given force and moment acting on the rigid plate, the area of contact is determined by iteration or based on quadratic programming techniques using the static influence coefficients for displacements. The complex-valued impedance function is estimated by substituting an equivalent circular plate for the actual area of contact. Transforming the equivalent lumped system to the center of the plate, the nonlinear stiffness and damping matrices of the soil are derived. Formulae are given for the partial lifting-off of a disk and a strip. The results of the numerical method are compared to rigorous solutions for full contact. As an example, the dynamic response of the reactor building of a 1000 Mw plant to earthquake motion is calculated using the rigorous and approximate methods. Parametric studies are carred out. The influence of the frequency on the impedance function and on the distribution of stress in the area of contact, which determines the beginning of lift-off, is discussed.

5.5-4 Kennedy, R. P. *et al.*, **Effect on non-linear soil-structure interaction due to base slab uplift on the seismic response of a high-temperature gas-cooled reactor (HTGR)**, *Nuclear Engineering and Design,* 38, 2, Aug. 1976, 323–355. (For an additional source, see Abstract No. 1.2-2.)

The primary purpose of this paper is to evaluate the importance of the nonlinear soil-structure interaction effects resulting from substantial base slab uplift occurring during a seismic excitation. The structure considered for this investigation consisted of the containment building and prestressed concrete reactor vessel for a typical HTGR plant. A simplified dynamic mathematical model was utilized consisting of a conventional lumped mass structure with soil-structure interaction accounted for by translational and rotational springs whose properties are determined by elastic halfspace theory. Three different site soil conditions (a rock site, a moderately stiff soil, and a soft soil

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site) and two levels of horizontal ground motion (0.3 and 0.5 g earthquakes) were considered.

Based upon the parametric cases analyzed in this investigation, it may be concluded that linear analysis (which ignores the nonlinear soil-structure interaction effects of base slab uplift) can be used to conservatively estimate the important behavior of the base slab even under conditions of substantial base slab uplift. For all cases investigated here, linear analysis resulted in higher base overturning moments, greater toe pressures, and greater heel uplift distances than nonlinear analyses. It may also be concluded that the nonlinear effect of uplift does not result in any significant lengthening of the fundamental period of the structure. Also, except in the short period region (period less than half of the fundamental period) only negligible differences exist between in-structure response spectra based on linear analysis and those based on nonlinear analysis.

Finally, it may be concluded that for sites in which soil-structure interaction is not significant, as for the rock site, the peak structural response (shears and moments) at all locations above the base mat are not significantly influenced by the nonlinear effects of base slab uplift. However, for the two soil sites the peak shears and moments are, in a few instances, significantly different between linear and nonlinear analyses. As a result, linear analysis may be used to determine all structural response for rock sites even when there is substantial base slab uplift. However, for soil sites, nonlinear analyses are necessary if substantial base slab uplift occurs.

● 5.5-5 Novak, M. and Nogami, T., **Soil-pile interaction in horizontal vibration,** *Earthquake Engineering and Structural Dynamics,* **5,** *3,* July–Sept. 1977, 263–281.

Interaction between soil and an elastic pile vibrating horizontally is theoretically examined. The soil is modeled as a linear, viscoelastic layer overlying rigid bedrock. The pile is assumed to be vertical and point bearing. A direct solution is developed which yields closed-form formulas for pile displacement, stiffness, and damping. A parametric study clarifies the role of the parameters involved, illustrates the interaction between the soil and the pile, and shows the stiffness and damping properties of the soil-pile system for typical values of the governing parameters.

● 5.5-6 Novak, M. and Howell, J. F., **Torsional vibrations of pile foundations,** *Journal of the Geotechnical Engineering Division, ASCE,* **103,** *GT4,* Proc. Paper 12850, Apr. 1977, 271–285.

Figures are presented to facilitate this application and the response of various representative footings to torsional excitation is also shown. For a pile group, the contribution of pile twisting to the total response in torsion is usually small. For a single pile or pier, torsion can be important and the inclusion of material damping is essential. The following dimensionless parameters govern the soil-pile interaction in torsion: shear wave velocity ratio, slenderness ratio, mass ratio, dimensionless frequency, and material damping ratio. With increasing shear wave velocity ratio, both pile stiffness and damping increase. For soft soils and pile slenderness ratio greater than 25, the response is independent of the slenderness ratio and the tip condition. Pile foundations can have higher resonant frequencies but smaller resonant amplitudes than shallow foundations. The analysis presented compares favorably with the static solution of Poulos.

● 5.5-7 Richardson, G. N. *et al.*, **Seismic testing of reinforced earth walls,** *Journal of the Geotechnical Engineering Division, ASCE,* **103,** *GT1,* Proc. Paper 12696, Jan. 1977, 1–17.

This paper is a summary progress report of ongoing studies at UCLA toward developing a rational seismic design method for reinforced earth retaining walls. A 20 ft (6 m) high reinforced earth test wall was constructed and then subjected to low strain level forced vibration tests using mechanical vibrators and high strain level explosive tests. In addition, four existing reinforced earth walls were also subjected to forced vibration tests. Criteria are presented for determining the design modal frequencies of a reinforced earth wall needed for the design as a function of its effective height and level of seismic excitation. Data are also presented showing the static and dynamic tie force distribution.

● 5.5-8 Singh, J. P., Donovan, N. C. and Jobsis, A. C., **Design of machine foundations on piles,** *Journal of the Geotechnical Engineering Division, ASCE,* **103,** *GT8,* Proc. Paper 13158, Aug. 1977, 863–877.

A practical method for the analysis of pile-supported foundations subjected to dynamic loadings is described. The method uses a single degree-of-freedom mass-spring-dashpot model of a form similar to that used for shallow foundations to analyze the response of the actual system. The method of analysis uses the concept of an equivalent cantilever, which is a technique to simplify the soil-structure interaction problem, and allow the computation of equivalent spring constants in all modes of vibration. The method is demonstrated with an actual case history where computed and observed motions are compared.

● 5.5-9 Novak, M., **Vertical vibration of floating piles,** *Journal of the Engineering Mechanics Division, ASCE,* **103,** *EM1,* Proc. Paper 12747, Feb. 1977, 153–168.

An approximate analytical solution is presented for the vertical dynamic response of a floating pile. The dynamic stiffness and damping of the soil-pile system are also

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obtained. The results are compared with experiments. It appears that the motion of the tip cannot be neglected unless the pile is extremely long or the tip rests on rigid bedrock. The motion of the tip reduces the stiffness and increases the damping. With increasing length, the stiffness of a floating pile increases, while the stiffness of an end bearing pile decreases. The damping of a floating pile is larger than that of an end bearing pile of equal length. Consequently, the dynamic response of a footing can be much smaller with floating piles than with end bearing piles. The relaxation of the tip improves the agreement between the theory and experiments.

● **5.5-10** Saran, S. and Prakash, S., **Effect of wall movement on lateral earth pressure,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2371-2372.

An analytical solution of lateral earth pressure distribution on retaining walls subjected to earthquakes has been developed based on the philosophy of Dubrova. Formulations are presented for an inclined wall with cohesionless backfill. Solutions have been obtained, both in active and passive states, for all three possible modes of wall movement, i.e., (1) rotation about the bottom, (2) translation, and (3) rotation about the top.

● **5.5-11** Jakovlev, P. I., **Coefficients of active and passive earth pressure on retaining walls under seismic conditions,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2356-2362.

On the basis of Coulomb's theory, formulas for determining a slip surface and an active earth pressure coefficient for the case of an arbitrary uneven loading of a horizontal backfill surface are suggested. The solution is derived for the case of a rough inclined wall. Dimensionless coefficients of the active and passive earth pressures upon inclined rough walls under uniformly distributed load, acting on an inclined earth surface, were found on the basis of a safe stress state theory developed by V. V. Sokolovsky, and S. S. Golushkevich.

● **5.5-12** Aliev, G. A., **Influence of properties and stressed state of foundation soils on absorption and diffusion of energy of seismic vibrations,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2904-2905.

The results of an experimental investigation of the free and forced oscillations of full-scale models of a foundation-structure system are stated. The influence of the elastoplastic properties of the foundation soils on the dynamic characteristics and the response of the system to seismic disturbances is analyzed.

● **5.5-13** Margason, E. and Holloway, D. M., **Pile bending during earthquakes,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1690-1696.

There are few, if any, actual measurements of pile bending during earthquakes; only post-seismic or simulated effects can presently be studied. To better understand pile bending, a survey of published field and laboratory case histories has been made and conclusions are drawn. These conclusions are then used as assumptions for available dynamic response analyses of soil profiles to rationally estimate pile curvature. The resulting maximum curvature values are then used to design piles to withstand the induced curvature moment, and ranges of real curvature values for prototype piles are described.

● **5.5-14** Munirudrappa, N., Ramiah, B. K. and Rajanna, B. C., **Response of embedded footings and structures under earthquake motion,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1647-1653.

An approximate solution is presented for the coupled response of footings and structures partially embedded in a semi-infinite medium. The response study has been made with the general equilibrium equation of motion, which includes dynamic interaction coefficients. The foundation is assumed to be flexible and supported on an elastic medium. The approach to the problem is illustrated by the solution of the coupled response, involving horizontal translation, rocking, and vertical translation. The interaction coefficients obtained by Parmelee are used in the analysis of embedded footings and structures. The results are presented for displacements resulting from the translational, horizontal, and rocking motion of structures founded on flexible foundations.

● **5.5-15** Petrovski, J., **Prediction of dynamic response of embedded foundations,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1639-1646.

The dynamic response of foundations and structures, using soil-structure interaction parameters determined from the theory of surface footings, cannot be correctly estimated. In order to establish a suitable procedure for evaluating soil-structure interaction parameters, an intensive experimental study of surface and embedded footings as well as full-scale structures has been performed at IZIIS, Univ. of Skopje. Using experimental results and modified theoretical solutions, a procedure for determination of equivalent linear dynamic properties of soil media and soil-structure interaction parameters has been evaluated. It is found that with performance of dynamic field tests on embedded footings and use of the described procedure,

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dynamic response of embedded foundations as well as structural systems can be predicted accurately.

● 5.5-16 Luco, J. E. and Wong, H. L., **Dynamic response of rectangular foundations for Rayleigh wave excitation,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1542–1548.

The dynamic harmonic response of a rigid massless rectangular foundation welded to an elastic halfspace and subjected to the action of a horizontally propagating Rayleigh wave is analyzed. The results indicate that the response of the foundation contains a pronounced rocking component in addition to the vertical and horizontal components of motion. The horizontal and vertical components of the response exhibit a marked decrease in amplitude for high frequencies. The results for incident Rayleigh waves are completely different from those obtained on the basis of the usual assumption of vertically incident SH waves.

● 5.5-17 Kobori, T., Minai, R. and Kusakabe, K., **Dynamical cross-interaction between two foundations,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1484–1489.

This paper examines the vibrational characteristics of two foundations coupled with each other through a soil medium. Vibrational field tests for two identical foundations under harmonic excitation by a vibration generator were carried out and their responses were measured by velocity-type seismometers. Vibrational characteristics of the two coupled foundations have also been analyzed theoretically by making use of a dynamic ground compliance matrix of the foundations on a viscoelastic halfspace. The theoretical results are compared with the experimental evidence.

5.6 Experimental Investigations

5.6-1 Barkan, D. D. and Rubin, B. I., **Experimental investigations of vibration settlement of sandy soils** (Eksperimentalnye issledovaniya vibrouplotneniya peschanykh gruntov, in Russian), *Svoistva gruntov pri vibratsiyakh,* FAN, Tashkent, 1975, 76–79.

A series of experiments was performed on a shaking table to investigate the effects of vibration frequency and amplitude on the porosity and settlement time of sandy soils. The static load on the sample was held constant. Settlement is found to increase when the vibration frequency is increased. Critical ground acceleration depends on the relation between amplitude and velocity. The duration of vibration settlement is sharply reduced when the frequency increases and it depends on the acceleration at frequencies below 50Hz.

5.6-2 Price, D., **Reinforced earth research,** *Civil Engineering* (London), Dec. 1976, var. pp.

From 1968 to 1972, the Lab. Central des Ponts et Chaussees in Paris investigated the failure of reinforced earth retaining structures by means of two- and three-dimensional models, tests of triaxial specimens and the monitoring of full-scale structures. One of the conclusions was that the single size three-dimensional model investigation gave the most unsafe predictions when compared with measurements on working structures. Another fact was that the tests showed the importance of knowing the true value of the ratio of horizontal to vertical stresses (K) in a reinforced earth structure. The tests also demonstrated the danger of using assumed values, obtained from conventional soil environments, to predict the behavior of working reinforced earth structures.

Work carried out in the United States and Japan has also investigated the basic problems of static design and concluded that the local failure design procedures proposed by the French engineers give satisfactory statistically loaded structures. The tests were then extended to look at the effects of earthquakes, explosions, large sudden variations in temperature (LNG) and vibrations from trains on reinforced earth structures. The results show that the static design gives a stable structure but that the additional loads do cause slightly increased lateral deformation in some cases.

● 5.6-3 Iwasaki, T. and Tatsuoka, F., **Dynamic soil properties with emphasis on comparison of laboratory tests and field measurements,** *Wind and Seismic Effects,* VI-158–VI-178. (For a full bibliographic citation, see Abstract No. 1.2-4.)

In order to evaluate analytically the motion of the ground during earthquakes, it is necessary to obtain the dynamic deformation properties of the soil deposits, especially the strain amplitude-dependent shear moduli and the damping coefficients. At two sites, Iruma, Minami-Isu-cho, and Ohgi-shima, Kawasaki-shi, in-situ seismic surveys were performed. At the former site, a sand embankment was damaged during the Off-Izu peninsula earthquake on May 9, 1974, and at the latter is reclaimed land where borehole accelerometers are installed. Sands sampled from these sites were tested with a Drnevich-type resonant-column apparatus to obtain the shear moduli and damping capacities at small strains. Laboratory test results showed that the two natural sands, which were well graded and included fine particles, had smaller shear moduli than did uniform clean sands such as Toyoura sand and Ottawa sand. A comparison of the shear moduli from shear wave velocities and those from resonant-column tests was performed, giving excellent correlation for sands from both sites.

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● **5.6-4** Marcuson, III, W. F., Cooper, S. S. and Bieganousky, M. A., **Laboratory investigation of undisturbed sampling and standard penetration tests on fine sands,** *Wind and Seismic Effects,* VI-141–VI-147. (For a full bibliographic citation, see Abstract No. 1.2–4.)

Determination of density of sand is presented. Densities determined from using undisturbed samples and from the standard penetration tests are compared. It is shown that the standard penetration test is not sufficiently accurate to be recommended for final evaluation of the density at a site unless site specific correlations are developed.

● **5.6-5** Aizenberg, Ya. M. and Leglina, M. M., **Dynamic testing of earthquake-resistant systems employing disengaging joints and buffers** (Dinamicheskie ispytaniya sistemy seismozashchity s vyklyuchayushchimisya svyazami i uporami-ogranichitelyami, in Russian), *Seismostoikoe stroitelstvo, 1,* 1977, 18–22.

A full-scale model of a pile foundation with reinforced concrete beam foundation mat was tested in order to verify the accepted hypotheses in the design of earthquake-resistant structures. A theoretical analysis of these hypotheses is difficult or impossible at present. Instantaneous brittle fracture of an elastic joint in a vibrating linear system subjected to high intensity dynamic loadings is considered. At the moment of fracture, the potential energy of the system is often assumed to be wholly transformed into kinetic energy, leading to an instantaneous jump of velocity and a consequent shock impulse. The purpose of the experiments was to investigate the accuracy of this assumption. The effects of buffers on the behavior of the system were also investigated.

● **5.6-6** Annaki, M. and Lee, K. L., **Equivalent uniform cycle concept for soil dynamics,** *Journal of the Geotechnical Engineering Division, ASCE,* **103**, *GT6,* Proc. Paper 12991, June 1977, 549–564.

While additional studies of this nature would be appropriate, especially using direction-independent tests such as cyclic simple shear, torsion shear, or shaking table tests, the authors feel justified in concluding that the data presented generally confirm the validity of the equivalent cycle or the cumulative damage method of dealing with irregular loading effects on soil. Some other aspects include a clarification of the relations between damage potential and equivalent number of uniform cycles, certain peculiarities of the cyclic triaxial test on saturated sands that may not be significant for clays, and a perspective comparison of the range of data scatter for cyclic tests on soil as compared with other materials. Thus, while the damage potential or equivalent cycle concept may not be valid for all cases, its continued use in the field of seismic stability analyses of soils seems to be a valid pragmatic approach.

● **5.6-7** Ladd, R. S., **Specimen preparation and cyclic stability of sands,** *Journal of the Geotechnical Engineering Division, ASCE,* 103, *GT6,* Proc. Paper 13014, June 1977, 535–547.

The results of a study on the cyclic structural stability of saturated sands as determined by the cyclic triaxial strength test (liquefaction test) are presented. The results clearly demonstrate that the method of specimen preparation can significantly affect the cyclic behavior of sands, and this effect is most likely directly related to variances in the fabric of the sand. It is also concluded that a better understanding of the strain development characteristics of sands could play an important role in evaluating the results of cyclic triaxial strength tests.

● **5.6-8** Mulilis, J. P. *et al.*, **Effects of sample preparation on sand liquefaction,** *Journal of the Geotechnical Engineering Division, ASCE,* **103**, *GT2,* Proc. Paper 12760, Feb. 1977, 91–108.

The results of undrained stress-controlled cyclic triaxial tests indicated that the dynamic strength of saturated sands, remolded to the same density by different compaction procedures, was significantly different. Variations in the dynamic strength of Monterey No. 0 sand were found to be in the order of 100%; however, tests on other sands indicated that the magnitude of the effect of the method of preparation used may be a function of the type of sand. Fabric studies and electrical conductivity measurements indicated that the orientation and arrangement of the contacts between sand grains were probably the primary reasons for the observed differences in the dynamic strength of Monterey No. 0 sand. Comparisons indicated that specimens prepared by moist tamping or moist vibratory compaction would exhibit dynamic strengths most similar to those of undisturbed samples; however, in most cases, the strength of undisturbed samples ranged from 0% to 45% higher than that of samples prepared by moist tamping.

● **5.6-9** Kiekbusch, M. and Schuppener, B., **Membrane penetration and its effect on pore pressures,** *Journal of the Geotechnical Engineering Division, ASCE,* **103**, *GT11,* Proc. Paper 13331, Nov. 1977, 1267–1279.

An apparatus is described to measure the volume changes due to the membrane penetration into the interstices of a specimen of sand or silt with changing pressures on the membrane. These tests confirm the relation between mean grain size and membrane penetration. It is possible to reduce the membrane penetration by up to 85% by spreading the membrane with a thin layer of liquid rubber. This initially liquid rubber is pressed into the interstices of the surface grains by the cell pressure where it gradually hardens. A comparison of undrained triaxial tests with sand shows that when the membrane is treated with liquid rubber the excess pore-water pressure generated in the

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undrained test is up to 100% higher than in a test with a normal membrane. So the normal use of membranes will underestimate the excess pore-water pressure and overestimate the undrained shear strength.

● **5.6-10** France, J. W. and Sangrey, D. A., **Effects of drainage in repeated loading of clays,** *Journal of the Geotechnical Engineering Division, ASCE,* 103, *GT7,* Proc. Paper 13081, July 1977, 769–785.

An effective stress model for the response of saturated clay soils experiencing cyclic loading is described. Using undrained response as a basis, the influence of periodic drainage during the cyclic loading is illustrated and is quantified within the effective stress model. Behavior is separated for contractive and dilative soils. For contractive soils, drainage results in volume decrease and eventual equilibrium under a particular constant level of cyclic loading. As a result of the drainage, there is significant strength increase. Dilative soils increase in volume as a result of cyclic loading with drainage and there is a resulting decrease in strength. Stress changes, water content changes, and strength after cyclic loading are predictable using this model.

● **5.6-11** Bieganousky, W. A. and Marcuson, III, W. F., **Laboratory standard penetration tests on Reid Bedford model and Ottawa sands,** *Liquefaction Potential of Dams and Foundations, Report 1, Research Report S-76-2,* Soils and Pavements Lab., U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, Oct. 1976, 158.

The standard penetration test (SPT) is critically examined with respect to its ability to estimate relative density in situ. A review of the literature tracing the development of the SPT is presented. Twenty-six test specimens, 4 ft in diam by 6 ft high, were constructed using Reid Bedford model sand and Ottawa sand. A variety of placement techniques were incorporated in the test program. The problems encountered in constructing large homogeneous test specimens are discussed. Using field drilling equipment, the SPT was performed at three overburden pressures on test specimens built to various densities. The results are presented as a family of curves correlating relative density with the SPT N-values at the testing pressures. This correlation is compared with correlations prepared by Gibbs and Holtz at the Bureau of Reclamation and Bazaraa at the Univ. of Illinois. It is concluded that the SPT is fairly repeatable in homogeneous deposits; however, variations in density, structure, or lateral stress within the test medium will produce widely scattered N-values. Thus, estimates of in-situ relative densities from N-values should be considered gross values or trends and should not be interpreted as accurate determinations for any specific case.

● **5.6-12** Bieganousky, W. A. and Marcuson, III, W. F., **Laboratory standard penetration tests on Platte River sand and standard concrete sand,** *Liquefaction Potential of Dams and Foundations, Report 2, Research Report S-76-2,* Soils and Pavements Lab., U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, Feb. 1977, 91.

The standard penetration test (SPT) is critically examined with respect to its ability to estimate relative density in situ. Six test specimens, 4 ft in diameter by 6 ft high, were constructed using Platte River sand and standard concrete sand. SPTs were performed, using field drilling equipment, on specimens under three overburden pressures and constructed to three relative densities. The results are presented as a family of curves correlating relative density with SPT N-values at the three overburden pressures. This research is an extension of a previous test series on Reid Bedford model sand and Ottawa sand. The results from tests of the four sands are compared, and a statistical analysis is presented which produced an empirical equation relating relative density to overburden pressure, SPT N-value, and coefficient of uniformity. Comparisons are also made between this work and that of Gibbs and Holtz at the Bureau of Reclamation and Bazaraa at the Univ. of Illinois. Conclusions are presented based on both series of tests.

● **5.6-13** Yong, R. N., Akiyoshi, T. and Japp, R. D., **Dynamic shear modulus of soil using a random vibration method,** *Soils and Foundations,* 17, *1,* Mar. 1977, 1–12.

The random vibration method is proposed in this study for evaluation of the dynamic shear modulus of soil using the apparatus developed by Hardin, and the results are compared with that obtained by the standard sinusoidal test. The soil specimens, made of two kinds of clay and sand, were placed in a triaxial cell and twisted at the top by sinusoidal or random forces through the magnetic coils under uniform confining pressure and drained condition. The shear modulus of the soil subjected to sinusoidal vibration was computed from the resonant frequency of the soil specimen-top cap coupling system, and that subjected to random vibration was computed from the expected frequency which was deduced from the narrow-band process theory. In this research study, shear strain amplitude was very small (less than 10^{-5}), and the equivalent damping constant of the soil was less than 0.05.

If the test apparatus-specimen coupling system produces an apparent nonsymmetry in the frequency response function with respect to the resonant frequency, variations in the evaluation of the information on the response characteristics of the system over a wide range of frequency result, creating inconsistencies in computation of the shear moduli from the random and sinusoidal vibration tests. Larger differences between the shear moduli using both methods are observed at low confining pressures.

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● **5.6-14** Hill, K. D., Lee, V. W. and Udwadia, F. E., **Non linear soil identification,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2340–2344.

Laboratory tests have been performed on free-standing soil samples in which deformations in the nonlinear range have been studied. The soil samples were tested on a shaking table. The nonlinear response to white noise inputs has been modeled using methods of nonparametric identification. Wiener kernels have been determined for such tests, and these kernels have been further used to model soil responses to sinusoidal excitations. The white noise testing technique for obtaining nonlinear soil characterization has been evaluated.

● **5.6-15** Wolfe, W. E., Annaki, M. and Lee, K. L., **Soil liquefaction in cyclic cubic test apparatus,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2151–2156.

Although many studies have been made of soil liquefaction due to cyclic loading, the laboratory tests have always been limited to two-directional plane or axisymmetric boundary conditions. In 1967-1968, Ko and Scott described a cube type of laboratory device which was capable of subjecting a test specimen to three-directional types of normal static stresses. The study described herein used essentially the same equipment, but was adapted for cyclic loading liquefaction tests on loose saturated sand.

● **5.6-16** Finn, W. D. L. and Vaid, Y. P., **Liquefaction potential from drained constant volume cyclic simple shear tests,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2157–2162.

A constant volume cyclic shear test has been developed for measuring liquefaction potential of saturated sand using either drained saturated specimens or dry sand. The test is free from many of the difficult and time-consuming features of undrained cyclic loading tests. It is quickly performed, gives results with extraordinarily high reproducibility, and leads to a more accurate assessment of liquefaction than current undrained tests.

● **5.6-17** Gupta, M. K. and Prakash, S., **Sand liquefaction during shake table vibration,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2169–2174.

A laboratory study of sands obtained from Ukai, Obra, and Tenughat dam sites and the Solani River was performed on a shaking table. The sands were subjected to steady-state horizontal vibrations. The grain size of sands varied from fine to very coarse. The effects of initial density, and the initial effective surcharge from a number of cycles of motion and acceleration have been studied. Dense sands developed negative pore pressure during vibration. Two types of surcharge devices were used: an air pressure surcharge device and a dead weight surcharge device. Pore pressure developed under a dead weight surcharge was observed to be very high in comparison to that resulting when a surcharge from air pressure was used.

● **5.6-18** Scott, R. F., Liu, H.-P. and Ting, J., **Dynamic pile tests by centrifuge modelling,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1670–1674.

The scaling relations for performing dynamic model tests in a centrifuge are summarized. A description is given of centrifugal experiments (at 100 g) in the lateral vibrations of flexible model piles with attached masses embedded in dry sand. The results of simplified analyses are presented. Model pile lateral frequencies of 115 to 160 Hz for embedment depths of 7.1 cm to 15 cm were observed. These correspond to prototype pile vibration frequencies of 1.15 to 1.6 Hz for depths of 7.1 m to 15 m. Damping was calculated to be in the range of 2% to 7% of critical for the smaller to the greater embedded lengths. Subgrade reaction coefficients for an equivalent Winkler soil model were found to be in the order of 5,000 MN/m^3 in the model (50 MN/m^3 in the prototype).

5.6-19 Mitchell, R. J. and King, R. D., **Cyclic loading of an Ottawa area Champlain Sea clay,** *Canadian Geotechnical Journal,* **14**, *1*, Feb. 1977, 52–63.

Undrained cyclic loading of triaxial samples of a sensitive Champlain Sea clay at deviatoric stress levels in excess of 50% of the static shear strength is shown to produce large deformations and eventual shear failure. Continued deformation of the clay under repeated loadings is believed to result from a progressive destruction of the cemented soil structure. Effective stress failures result from an increase in the excess pore water pressures within the sample.

5.6-20 Yamamoto, K., Kusunose, K. and Hirasawa, T., **Frequency distributions of microfractures in rock samples under cyclic loading tests** (in Japanese), *Zisin, Journal of the Seismological Society of Japan,* **30**, *4*, Dec. 1977, 477–486.

The parameter m in Ishimoto-Iida's frequency-amplitude relation is investigated for acoustic emissions occurring in granite and granodiorite specimens under uniaxial and triaxial compression tests. It is found that the m-value in the case of the uniaxial test decreases with an increase in axial stress, and that the m values in the triaxial test of 200 bars of confining pressure are somewhat larger on the average than those in the former case at the same stress difference. The m-value in the latter seems to fluctuate even while the stress difference is kept nearly constant. In

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the case of cyclic loading of uniaxial stress on granite, the m-value on the second loading is larger than that on the first loading at almost the same stress. These characteristics suggest that the m-value depends not only on the stress difference but also on the mode and history of deformation of the rock sample.

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6. Dynamics of Structures

6.1 General

6.1-1 Rabinovich, I. M., **The continuous beam problem and its effects on the general theory of statically indeterminate systems** (Zadacha o nerazreznoi balke i ee vliyanie na obshchuyu teoriyu staticheski neopredelimykh sistem, in Russian), *Issledovaniya po teorii sooruzhenii*, *22*, 1976, 3–9.

A survey of the ideas and methods originating in the theory of continuous beams, later developed and applied to a wide range of statically indeterminate systems, is presented.

● 6.1-2 Harris, C. M. and Crede, C. E., eds., **Shock and vibration handbook**, 2nd ed., McGraw-Hill Book Co., New York, 1976, 1322.

The publication is the second edition of a previously published handbook and comprises 44 chapters. The 44 chapters were written by 54 authorities from industry, government laboratories, and universities. Chapters dealing with related topics are grouped together. The first group of chapters provides a theoretical basis for shock and vibration. The second group considers instrumentation and measurements. This is followed by a new chapter on vibration standards. The next group of chapters deals with analysis and testing, concepts in the treatment of data obtained from measurements, and procedures for analyzing and testing systems subjected to vibration and shock. To this group has been added a new chapter on the use of digital computers. The subject of methods for controlling shock and vibration is discussed in a group of chapters dealing with isolation, damping, and balancing. This is followed by chapters devoted to equipment design, packaging, and the effects of shock and vibration on man. Duplication of material between chapters is avoided insofar as this is desirable, cross references to other chapters being used frequently. There are extensive references to available technical literature.

6.2 Dynamic Properties of Materials and Structural Components

● 6.2-1 Paulay, T., **Ductility of reinforced concrete shearwalls for seismic areas**, *Reinforced Concrete Structures in Seismic Zones*, *SP-53*, American Concrete Inst., Detroit, 1977, 127–147.

If structural survival of large seismic disturbances is to be assured, brittle failure modes due to shear, anchorage and local instability must be suppressed. Coupled shear wall structures are examined to illustrate the critical areas of behavior and the reasons for possible limited ductility. In particular, the shear strength of shear walls, the role of horizontal construction joints across walls and the behavior of deep and relatively short coupling beams are briefly discussed. The performance of two one-quarter full-size reinforced concrete coupled shear wall models, subjected to several cycles of reversed loading well beyond yield level, is briefly compared. One model contained conventionally reinforced coupling beams while the beams of the second model were reinforced with diagonally placed bars. The latter specimen showed that in coupled shear walls all the features desirable for seismic resistance, particularly ample ductility, can be attained. Great care with detailing and an intelligent distribution of ductile energy absorbing devices over the whole structure are likely to ensure that coupled shear walls will offer adequate seismic protection for reinforced concrete multistory buildings.

● 6.2-2 Artishev, V. P. and Pogosyan, O. K., **Experimental and theoretical analysis of the dynamic characteristics of a**

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150 m high ventilation duct (Eksperimentalnoe-teoreticheskoe opredelemie dinamicheskikh kharakteristik ventilatsionnoi trudy vysotoi 150 m, in Russian), *Seismostoikoe stroitelstvo, 2,* 1977, 33–36.

Experimental investigations of the free vibrations of structures comprising the Armenian nuclear power plant were carried out. One of the objects studied was the plant's 150 m high ventilation duct. Vibration measurement apparatus of various frequency bandwidths located at various heights were employed to record the first three vibration modes of the duct. Natural frequencies and mode shapes were also calculated and compared to experimental data.

● 6.2-3 Mukherjee, P. R. and Coull, A., **Free vibrations of open-section shear walls,** *Earthquake Engineering and Structural Dynamics,* **5**, *1,* Jan.–Mar. 1977, 81–101.

The coupled torsional-flexural vibration of open-section shear walls, braced by connecting beams at each floor level, is analyzed on the basis of Vlasov's theory of thin-walled beams. The basic dynamic equations and boundary conditions are derived from Hamilton's principle, and a numerical solution obtained by the Ritz-Galerkin method. In addition to the primary torsional and flexural inertias, secondary effects due to rotatory and warping inertia forces have also been taken into account. The method is suitable for both rigid and flexible base conditions. A series of numerical examples is presented in which analytical results are compared with available experimental data, and the effects of secondary inertia forces, base flexibility and connecting beams upon the vibration characteristics of such shear walls are examined for two different structural forms.

● 6.2-4 Cheung, Y. K., Hutton, S. G. and Kasemset, C., **Frequency analysis of coupled shear wall assemblies,** *Earthquake Engineering and Structural Dynamics,* **5**, *2,* Apr.–June 1977, 191–201.

The finite strip procedure is used to predict the free vibration response of both planar and nonplanar coupled shear wall assemblies. The solid walls are considered as vertical cantilever strips and a comparison is made between modeling the spandrel beams as discrete beams and as an equivalent continuum with orthotropic plate properties. It is shown that both approaches lead to essentially the same frequencies. The effects of vertical inertial forces and shear deflection are included, and structures considered may have properties that vary with height. The method presented appears to be more versatile than previously published techniques and numerical comparisons with existing methods indicate the predicted results to be accurate.

● 6.2-5 Tso, W. K. and Mahmoud, A. A., **Effective width of coupling slabs in shear wall buildings,** *Journal of the Structural Division, ASCE,* **103**, *ST3,* Proc. Paper 12817, Mar. 1977, 573–586.

The stiffness of a coupling slab between two shear walls is computed by means of the finite element method. A number of common wall configurations are considered. The results are expressed in terms of the effective widths of the slab and are presented in design chart form. Comparisons to available experimental results are made. It is concluded that the theoretical calculations are sufficiently accurate for design purposes, provided a correction for local deformation of the walls is made when the wall opening is small.

● 6.2-6 Henghold, W. M., Russell, J. J. and Morgan, III, J. D., **Free vibrations of cable in three dimensions,** *Journal of the Structural Division, ASCE,* **103**, *ST5,* Proc. Paper 12954, May 1977, 1127–1136.

The results of an extensive analysis of the three-dimensional free vibration of a single-span cable, using nonlinear finite elements, is presented. The method provides excellent results throughout the spectrum of end conditions and geometric nonlinearity. For a homogeneous cable of constant cross section, the nonlinear equilibrium position and the natural frequencies were shown to depend only on the end conditions and the stiffness-to-weight ratio. It has been demonstrated that results of engineering usefulness can be obtained from plots made for a typical value of the stiffness-to-weight ratio since realistic deviations from this value result in only minor changes in the frequencies. An empirical formula based on static calculations, which gives a good approximation to the lowest natural frequency, has been presented.

● 6.2-7 Tso, W. K. and Rutenberg, A., **Seismic spectral response analysis of coupled shear walls,** *Journal of the Structural Division, ASCE,* **103**, *ST1,* Proc. Paper 12671, Jan. 1977, 181–196.

An approximate method for the dynamic earthquake analysis of coupled shear walls within the framework of design response spectra is proposed. The first three modes of vibration are considered and the dynamic loading associated with each is assumed to be represented by the corresponding mode of a vibrating cantilever. Based on this approximation, the natural frequencies, deflection, internal forces, and moments are computed and the design parameters of interest are presented in design chart form. A numerical example is worked out in detail to illustrate the use of the design charts. The contribution of higher modes is examined and the need to consider their effect is demonstrated.

● 6.2-8 Arya, A. S., Gupta, S. P. and Shrivastava, S. K., **In situ measurement of dynamic characteristics of atomic**

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power plant equipment, *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(b), Paper K 8/7, 11. (For a full bibliographic citation, see Abstract No. 1.2–5.)

The seismic response of nuclear power plant equipment is normally based on a theoretical evaluation of the stiffness of the supporting systems and an estimated value of damping. It is always difficult to assess analytically the contribution of the piping connections to these parameters. For the realistic assessment of stiffness and damping, full-scale free vibration tests have been carried out on various pieces of equipment located in plant buildings both during the construction stage and after they were erected. Initial displacement or initial velocity was used to excite the free vibrations. Initial displacement was imparted by means of a steel rope pulled with a chain pulley block. The sudden release was achieved by means of a clutch system. An acceleration transducer with an amplifier and an ink-writing oscillograph was used for recording the vibrations. Frequency and damping were evaluated from the acceleration records. Observed values for some equipment are given.

● **6.2-9** Hegemier, G. A., **Mechanics of reinforced concrete masonry: a literature survey,** *Earthquake Response and Damage Prediction of Reinforced Concrete Masonry Multistory Buildings,* Dept. of Applied Mechanics and Engineering Sciences, Univ. of California, San Diego, La Jolla, Sept. 1975, 133.

This report attempts to survey, and comment upon, the available literature relevant to the mechanics of reinforced concrete masonry assemblages. Although a considerable amount of research on concete masonry has been conducted over the past forty to fifty years, there presently exists little correlation among the various studies conducted by governmental, university, and promotional research organizations. Each study has, of economic necessity, been constrained within extremely narrow bounds. Further, many such studies have never reached the open literature; they lie buried in the form of unpublished laboratory reports or data. Consequently, a complete survey of available data is a difficult and cumbersome task, and no attempt at completeness is claimed herein.

A reading of the masonry literature clearly indicates that much reliance has, in the absence of reliable data on concrete masonry, been placed upon the well-documented literature and experience concerning concrete and reinforced concrete. However, there are important differences in the behavior of concrete and masonry; the latter is a composite material and this must be taken into consideration. Hopefully, recommendations resulting from this research have been conservative.

● **6.2-10** Meinheit, D. F. and Jirsa, J. O., **The shear strength of reinforced concrete beam-column joints,** *CESRL 77-1,* Structures Research Lab., Dept. of Civil Engineering, Univ. of Texas, Austin, Jan. 1977, 271.

Beam-column joints were subjected to deformation to establish basic shear behavior. Fourteen interior type connections with reinforcing bars continuous through the joint were tested. The effects of column reinforcement, column load, joint hoop reinforcement, lateral beams, concrete strength, the ratio of the depth of the column to the depth of the main flexural member, and load reversals on the shearing strength of the beam-column joint were investigated. Specimens were designed using the recommendation of ACI-ASCE Committee 352 and were proportioned so that shear stress in the joint would determine the maximum loads rather than yielding of the flexural members. The failure mode of each specimen was observed. From a statistical analysis of the test data and the mode of failure, new design recommendations for basic shear strength are made that deemphasize column load, concrete shear cracking, and joint hoop reinforcement, and emphasize concrete strength. Based on the load reversal tests, a modification to the basic shear strength is proposed to account for shear strength degradation under cyclic load. The form of the proposed design equation is substantially different from the current ACI-ASCE Committee 352 recommended practice. The proposed design approach represents a significant advance in the understanding of the shear behavior of the beam-column joint.

● **6.2-11** Ichiki, T., Matsumoto, T. and Gunyasu, K., **A seismic analysis of nuclear power plant components subjected to multi-excitations of earthquakes,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(b), Paper K 6/12, 12. (For a full bibliographic citation, see Abstract No. 1.2–5.)

Using a previously developed analytical method, the authors study the seismic response of nuclear reactor components. It is possible that the method also can be applied to the seismic analysis of such three-dimensional systems as piping systems.

● **6.2-12** Morris, N. F., Harstead, G. A. and Soot, O., **Seismic analysis of a dry spent fuel cask handling system,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(b), Paper K 6/6, 8. (For a full bibliographic citation, see Abstract No. 1.2–5.)

An unusual system has been developed for handling the spent fuel cask during loading of spent fuel from the storage pool to decontamination and loading of the cask onto a railroad car or truck. The unique feature of this system is that the outer surface of the cask is kept dry.

However, since the system is so unusual, a seismic analysis of the system was deemed necessary.

The system contains a vehicle which rides on rails and transports the cask from the fuel loading area to the decontamination and shipping area. The cask is supported inside the tank. When the system is connected under the cask loading pool, the water pressure is maintained in such a manner that the pressure head in the tank is determined by the top elevation of the water in the cask loading pool. The cask system is supported, and moves on wheels. Guide shoes prevent the system from moving in the direction transverse to the tracks. When the system is at the loading terminal, motion along the track is prevented by lock bars acting at each side of the transporter. If the system is between the loading terminal and the decontamination terminal, only friction prevents its motion along the track. Because there are many design configurations which may arise in the loading history of the system, a program had to be established to determine the critical load state and the stresses which can arise in that load state. This entailed a large amount of computer analysis for static, earthquake, and impact loadings of similar structures with different support conditions.

The dry cask system was analyzed as a three-dimensional framework carrying a tank. It is an unusual system in that the tank is much more rigid than the supporting system. The actual analyses were carried out with the aid of SAP IV. Even though this meant that computer softwear support would not be available, it was felt the system could be modeled so that it would be satisfactorily analyzed with this program. This proved to be the case although, of course, some difficulties arose in the analysis. In summary, a detailed seismic analysis was carried out for a rather complex mechanical/structural system in an efficient manner utilizing state-of-the-art, public domain programs.

● 6.2-13 Bezler, P. and Curreri, J. R., **Subharmonic excitation in an HTGR core,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(b), Paper K 7/4, 5. (For a full bibliographic citation, see Abstract No. 1.2-5.)

The occurrence of subharmonic resonance in a series of seven blocks with clearance gaps between blocks and with springs and clearance on the outermost ends is the subject of this paper. The clearance between each block could be the same or different. Spring packs are installed between the first and last block and the input motion. The system nonlinearity is due to the severe discontinuity on the interblock elastic forces when adjacent blocks collide and to the clearance that is present between the outermost springs and the core elements. A computer program using a numerical integration scheme was developed to solve for the response of the system to arbitrary inputs.

Sweeping both up and down with sinusoidal excitation on this system shows the nonlinearity in the response behavior. The response in some regions can take on either of two values, depending on the initial conditions and the nature of the transients in the system. The phenomenon occurs even though the system has seven masses with gaps between each. A computer printout of the response of each block is shown on displacement-time plots. When the frequency was increased in the sweeping process, it was found that a second-order subharmonic was easily developed. This was not expected because of the symmetry in the system. This occurred when the forcing frequency was two times the first resonant frequency and very much below the second resonant frequency. Because of the bending of the response curve, the effect of the subharmonic occurs over a broad range. The paper shows the identification of the subharmonic with the seven masses participating.

The magnitude of the response due to the subharmonic is smaller than the primary resonance. Nevertheless, it is an added factor in the peak response of the system. More importantly the occurrence of the subharmonic means that substantial response might be expected of the system regardless of the frequency content of the excitation spectrum. In the nonlinear core of the HTGR, a response peak could occur even though the forcing frequency is above what might be considered as the natural frequency of the system. In addition, for more than one sine wave input, which is generally the case, the occurrence of a subharmonic means that large amplitudes can develop where the response is remarkably greater than the sum of the separate contributions of the components if they acted alone. The possibility of a subharmonic response developing should therefore be examined.

● 6.2-14 Ishizuka, H. *et al.*, **Basic study on seismic response of HTR core,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(b), Paper K 7/9, 12. (For a full bibliographic citation, see Abstract No. 1.2-5.)

Using a projected design of a very high temperature reactor core (VHTR) as a reference model, some basic experimental investigations and computer analyses have been carried out to evaluate the aseismic properties of a block-type HTR graphite fuel assembly. Although VHTR design is still in a preliminary stage, the behavior of a "multiblock" assembly under simple horizontal sinusoidal acceleration has been determined. A simplified theoretical model has been developed and verified with experimental results. Three types of models have been investigated: a one-dimensional array that consists of up to ten cubic graphit blocks with 10 cm sides, a horizontal two-dimensional assembly of sixty-one 1/5-scaled hexagonal graphite

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blocks and vertical elementary single and/or double columns consisting of twelve or thirteen 1/5-scaled hexagonal blocks.

● **6.2-15** Rakowski, J. E. and Olsen, B. E., **Measuring the seismic response of an HTGR core model**, *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(b), Paper K 7/7, 12. (For a full bibliographic citation, see Abstract No. 1.2-5.)

The seismic program for the design verification of an HTGR core and core support structures was previously introduced in two papers. These papers described the General Atomic Co. HTGR core seismic design approach, covering both analytical methods and results, and the results of seismic tests on core models of various sizes. The main objectives of this paper are to present (1) the new 1/5 scale full array core model instrumentation including design requirements, calibration, and data reduction methods; (2) sample test results obtained with the new instrumentation; and (3) a comprehensive error analysis of the data.

● **6.2-16** Crosbie, R. L., **Base isolation for brick masonry shear wall structures**, *77-2,* Dept. of Civil Engineering, Univ. of Canterbury, Christchurch, New Zealand, Feb. 1977, 187.

Base isolation is considered for the protection of reinforced brick masonry shear walls against damage during severe seismic attack. For low-period structures, it is also shown that significant reductions in design forces may be achieved with base isolation. All results reported were obtained using a dynamic inelastic computer program.

● **6.2-17** Wasserman, Y., **The influence of the behaviour of the load on the frequencies and critical loads of arches with flexibly supported ends**, *Journal of Sound and Vibration,* **54**, *4,* Oct. 22, 1977, 515-526.

In this work, exact and approximate formulas for determining the lowest natural frequencies and critical loads for arches with flexibly supported ends have been obtained in three cases of load behavior during the deformation process. The influence of the manner of loading on the frequencies and on the critical load has been shown as a function of the opening angle of the arch and the rigidity of the end supports.

● **6.2-18** Tabba, M. M. and Turkstra, C. J., **Free vibrations of curved box girders**, *Journal of Sound and Vibration,* **54**, *4,* Oct. 22, 1977, 501-514.

The problem of coupled free vibrations of curved thin-walled girders of nondeformable asymmetric cross section is examined in this paper. The general governing differential equations are derived for quadruple coupling between the two flexural, tangential, and torsional vibrations. An approximate solution for the case of triple coupling between the two flexural and the torsional vibrations is given for a simply supported girder, uniform specific gravity of the material of the box being assumed. Section warping is considered but axial forces, rotary inertia, and structural damping are neglected. A parametric study is conducted to investigate the effect of relevant parameters on natural frequencies. Eigenfunctions satisfying the orthogonality condition are given. The solution derived herein for the general case is also shown to cover a variety of special cases of straight and curved girders with doubly symmetric or singly symmetric cross sections.

● **6.2-19** Gianetti, C. E., Diez, L. and Laura, P. A. A., **Transverse vibrations of rectangular plates with elastically restrained edges and subject to in-plane shear forces**, *Journal of Sound and Vibration,* **54**, *3,* Oct. 8, 1977, 409-417.

Simple polynomial expressions and the Galerkin method are used in the present study. The results are in good agreement with values previously published for the range of shear values used in this investigation. Consideration of in-plane, normal forces does not add any complications to the procedure.

● **6.2-20** Ku, A. B., **Upper and lower bounds for fundamental natural frequency of beams**, *Journal of Sound and Vibration,* **54**, *3,* Oct. 8, 1977, 311-316.

In this article, it is pointed out that a better upper bound than the Rayleigh quotient is the Timoshenko quotient, the evaluation of which depends on a pair of compatible admissible moment and displacement functions. Based on both Rayleigh and Timoshenko quotients, a lower bound is readily computed. By means of an iteration procedure, both the upper and lower bound converge to the fundamental natural frequency.

● **6.2-21** Karmakar, B. M., **Non-linear dynamic behaviour of plates on elastic foundations**, *Journal of Sound and Vibration,* **54**, *2,* Sept. 22, 1977, 265-271.

The large amplitude transverse vibration characteristics of a simply supported isotropic equilateral triangular plate on an elastic foundation have been investigated by following Berger's well-known approximate method in conjunction with a special type of coordinates known as trilinear coordinates. A second order nonlinear differential equation for the unknown time function has been obtained and solved, as usual, in terms of Jacobian elliptic functions. Results obtained from numerical calculations are presented graphically.

● **6.2-22** Al-Jumaily, A. M. and Faulkner, L. L., **Vibration of continuous systems with compliant boundaries**, *Journal of Sound and Vibration,* **54**, *2,* Sept. 22, 1977, 203-213.

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A theoretical study of two types of continuous systems with a general form of compliant boundary conditions is presented. The systems considered are elastic beams and circular plates with elastic damped edge constraints. Beam studies are restricted to those with identical boundary conditions at each end. The method of solution consists of formulating the edge condition of the system in terms of the impedance of the compliant boundary material and of using classical solution techniques to solve the equations of motion. The result of matching the boundary conditions of the system with constraining conditions is the system frequency equation in terms of the constraint impedances. The influence of the compliant material on the vibration of the structure is discussed. The models give numerically the effect of elasticity and damping of the supports on the resonant frequencies of the systems. Parameters are obtained which indicate when one may assume simply supported or clamped boundaries for the actual case of elastic damped constraints without introducing large errors in the natural frequencies.

● **6.2-23** Beards, C. F. and Williams, J. L., **The damping of structural vibration by rotational slip in joints,** *Journal of Sound and Vibration,* 53, 3, Aug. 8, 1977, 333–340.

Interfacial slip in joints is the major contributor to the inherent damping of most fabricated structures. By fastening joints tightly enough to prohibit translational slip, but not tightly enough to prohibit rotational slip (thereby making only a small sacrifice in static stiffness), it is shown, both experimentally and theoretically, that a useful increase in the inherent damping in a structure can be achieved, provided an optimum joint load is maintained. The analysis is simplified by using a general dynamic analysis computer program with a subprogram to model the friction joint.

● **6.2-24** Celep, Z., **An analogy between free vibration of a plate and of a particle of mass,** *Journal of Sound and Vibration,* 53, 3, Aug. 8, 1977, 323–331.

In this paper, the free flexural vibration of an elastic circular thin plate with an initial imperfection is investigated. Approximate solution of the problem for the fundamental frequency of vibration, of large amplitude and with the plate imperfection, leads to a nonlinear ordinary differential equation with time as the independent variable. It is shown that this equation also represents the free vibration of a particle of mass on a shallow curve of fourth degree. With this similarity in view, it is possible to draw an analogy between these two vibrations. A numerical analysis is made with particular reference to this analogy and the results are given in figures which represent the vibratory motion and the period of vibration versus the initial amplitude of the plate or of the particle of mass.

● **6.2-25** Neubert, V. H. and Rangaiah, V. P., **The prediction of transient response of beams by transform techniques,** *Journal of Sound and Vibration,* 53, 2, July 22, 1977, 173–181.

Analysis of response of structures to shock loads is one of the important aspects of structural design. In the past, efficient mathematical models which predict accurate dynamic response were devised. A lumped parameter beam model based on mechanical impedance has been used to predict such dynamic responses as the transient response arising when a ground or base motion is applied to the beam. In the work reported here, the transient response (shear force and bending moment) of a beam, when its ends are subjected to an acceleration pulse of a half-sine wave, has been computed by Laplace transform methods. The shear force and bending moment at the ends of the beam as computed by Laplace transform methods compare well with the values obtained by closed-form solutions. The superiority of the Laplace transform solutions over normal mode solutions is demonstrated.

● **6.2-26** Farshad, M., **On lateral-torsional instability of arches subjected to motion-dependent loading,** *Journal of Sound and Vibration,* 53, 2, July 22, 1977, 165–171.

The problem of dynamic lateral-torsional instability of circular arches acted upon by generally nonconservative forces is treated. The influence of motion dependency of loading on the stability characteristics is determined in certain cases. It is also found that a semicircular arch which is only capable of rigid mode buckling according to static analysis may lose its stability through nonrigid modes if the dynamic criterion is utilized.

● **6.2-27** Radhamohan, S. K. and Maiti, M., **Vibrations of initially stressed cylinders of variable thickness,** *Journal of Sound and Vibration,* 53, 2, July 22, 1977, 267–271.

Natural frequencies and buckling loads for cylindrical shells having linearly varying thickness are obtained by using a segmentation technique. The present results for free vibration of a cylinder compare very well with those obtained previously. The effect of the thickness variation on the frequencies of a cylindrical shell is studied. Frequencies are also calculated for a cylinder of variable thickness under axial compression and a relationship between the frequency and axial compression is obtained for a particular wave number.

● **6.2-28** Sakata, T., **Forced vibrations of a rectangular plate with non-uniform thickness,** *Journal of Sound and Vibration,* 53, 1, July 8, 1977, 147–152.

The dependence on frequency of the maximum deflection and surface stresses of a simply supported rectangular plate subjected to a uniformly distributed sinusoidal

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excitation is discussed and simple formulas are proposed for estimating the deflection and surface stresses. The thickness of the plate varies linearly in one direction parallel to a side of the plate.

● **6.2-29** Wood, A. D. and Zaman, F. D., **Free vibrations of randomly inhomogeneous plates,** *Journal of Sound and Vibration,* **52,** *4,* June 22, 1977, 543-552.

This article considers a large collection of elastic rectangular plates with random inhomogeneitics, but otherwise indistinguishable in any overall sense. An expression is obtained for the natural frequency of such plates, vibrating freely under simply supported boundary conditions. The paper concludes with a theorem linking the mean and variance of the natural frequency to the volume concentration and geometry of the inclusions in the plate.

● **6.2-30** Prabhakara, M. K. and Chia, C. Y., **Non-linear flexural vibrations of orthotropic rectangular plates,** *Journal of Sound and Vibration,* **52,** *4,* June 22, 1977, 511-518.

This study is an analytical investigation of free flexural large amplitude vibrations of orthotropic rectangular plates with all clamped and all simply supported stress-free edges. The dynamic von Karman-type equations of the plate are used in the analysis. A solution satisfying the prescribed boundary conditions is expressed in the form of double series with coefficients being functions of time. The modal equations are solved by expanding the time-dependent deflection coefficients into Fourier cosine series. As obtained by taking the first 16 terms in the double series and the first two terms in the time series, numerical results are presented for nonlinear frequencies of various modes of the plates. The analysis shows that, for large values of the amplitude, the effect of coupling of vibrating modes on the nonlinear frequency of the fundamental mode is significant for orthotropic plates, especially for high-modulus composite plates.

● **6.2-31** Rao, D. K., **Vibration of short sandwich beams,** *Journal of Sound and Vibration,* **52,** *2,* May 22, 1977, 253-263.

Flexural vibrations of short unsymmetric sandwich beams are analyzed by formulating the equations of motion and the associated boundary conditions through the use of Hamilton's principle. The equations presented here include all the higher order effects like inertia, extension, and shear of all the layers, which are necessary for accurate prediction of frequency and loss factors of short sandwich beams. Many of the equations of motion available in the literature form special cases of the equations presented herein. Analysis of frequency and loss factors presented for typical short sandwich beams indicates that earlier theories of sandwich beams can lead to inaccurate results which may be in error by as much as 45 percent.

● **6.2-32** Laura, P. A. A., Filipich, C. and Santos, R. D., **Static and dynamic behavior of circular plates of variable thickness elastically restrained along the edges,** *Journal of Sound and Vibration,* **52,** *2,* May 22, 1977, 243-251.

Simple polynomial approximations and a variational approach are used to solve a rather complex elastomechanics problem. It is assumed that the plate is elastically restrained against rotation and translation along the edge. The approach developed in the present paper allows for a unified solution of both free and forced vibration problems, the static situation being a special situation of the dynamic state.

● **6.2-33** Sathyamoorthy, M., **Shear and rotatory inertia effects on large amplitude vibration of skew plates,** *Journal of Sound and Vibration,* **52,** *2,* May 22, 1977, 155-163.

The large amplitude free flexural vibration of elastic, isotropic skew plates is investigated, including the effects of transverse shear and rotary inertia. By use of Galerkin's method and the extended Berger approximation, solutions are obtained on the basis of an assumed vibration mode. The nonlinear period versus amplitude behavior is of the hardening type and the nonlinear period is found to increase when the effects of transverse shear and rotatory inertia are considered in the analysis. The influence of these effects on aspect ratios and skew angles of thin and moderately thick skew plates is investigated at small and large amplitudes.

● **6.2-34** Sundararajan, C., **Relationship between the fundamental frequency and the static response of elastic systems,** *Journal of Sound and Vibration,* **51,** *4,* Apr. 22, 1977, 493-499.

An approximate relationship between the fundamental frequency and the static response of undamped, linearly elastic systems is derived. The relationship is used to calculate the fundamental frequencies of membranes and plates of different geometries and boundary conditions.

● **6.2-35** Wang, T. M. and Stephens, J. E., **Natural frequencies of Timoshenko beams on Pasternak foundations,** *Journal of Sound and Vibration,* **51,** *2,* Mar. 22, 1977, 149-155.

A study of the natural vibrations of a Timoshenko beam on a Pasternak-type foundation is presented. Frequency equations are derived for beams with different end restraints. A specific example is given to show the effects of rotary inertia, shear deformation, and foundation constants on the natural frequencies of the beam.

● **6.2-36** Abbas, B. A. H. and Thomas, J., **The second frequency spectrum of Timoshenko beams,** *Journal of Sound and Vibration,* **51,** *1,* Mar. 8, 1977, 123-137.

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This paper investigates the reported existence of a second spectrum of frequencies of vibration of Timoshenko beams. A concept of coupled vibration is introduced and is used to explain the behavior of the Timoshenko beams with various end conditions. It is shown that except for the special case of a hinged-hinged beam there is no separate second spectrum of frequencies. It is also shown how several previous investigators misinterpreted the frequencies obtained for the Timoshenko beam and hence introduced the notion of a second spectrum of frequencies.

● **6.2-37** Ramachandran, J., **Frequency analysis of plates vibrating at large amplitudes,** *Journal of Sound and Vibration,* **51,** *1,* Mar. 8, 1977, 1-5.

A new method of finding the relationship between nonlinear fundamental frequency of vibration and the amplitude of vibration of plates is explained in this paper. In this method, use is made of an iteration procedure suggested by Schwarz and also of Berger's equations. As an example, a circular plate with clamped, immovable edges is considered.

● **6.2-38** Filipich, C., Laura, P. A. A. and Santos, R. D., **A note on the vibrations of rectangular plates of variable thickness with two opposite simply supported edges and very general boundary conditions on the other two,** *Journal of Sound and Vibration,* **50,** *3,* Feb. 8, 1977, 445-454.

An approximate solution is obtained by using the Galerkin method. The plate displacement function is approximated by means of a sinusoid multiplied by a polynomial. Translational and rotational flexibilities are taken into account. It is shown that the free edge situation (Kirchoff's boundary condition) can be treated as a special case by means of the approach developed herein. A simple algorithm which allows evaluation of the fundamental frequency of vibration is derived.

● **6.2-39** Kanaka Raju, K., **Large amplitude vibrations of circular plates with varying thickness,** *Journal of Sound and Vibration,* **50,** *3,* Feb. 8, 1977, 399-403.

A simple finite element formulation is presented to evaluate the large amplitude vibration frequencies of orthotropic circular plates with linearly varying thicknesses. Period ratios are presented in tables and figures for different values of the orthotropic and taper parameters.

● **6.2-40** Mazumdar, J. and Jones, R., **A simplified approach to the large amplitude vibration of plates and membranes,** *Journal of Sound and Vibration,* **50,** *3,* Feb. 8, 1977, 389-397.

The large amplitude transverse vibration of plates is analyzed by using the method of constant deflection contour lines and the well-known Berger method. By letting the plate stiffness tend to zero, the case of the large amplitude vibration of membranes is treated in a simple and uniform manner. Several illustrative examples are discussed, all details of which are explained by graphs.

● **6.2-41** Lobitz, D. W., Nayfeh, A. H. and Mook, D. T., **Non-linear analysis of vibrations of irregular plates,** *Journal of Sound and Vibration,* **50,** *2,* Jan. 22, 1977, 203-217.

A numerical perturbation method is used to investigate the forced vibrations of irregular plates. Nonlinear terms associated with the midplane stretching are retained in the analysis. The numerical part of the method involves the use of linear finite element techniques to determine the free oscillation mode shapes and frequencies and to obtain the linear midplane stress resultants caused by the midplane stretching. Representing the solution as an expansion in terms of these linear mode shapes, the authors use these modes and the resultants to determine the equations governing the time-dependent coefficients of this expansion. These equations are solved by using the method of multiple scales. Specific solutions are given for the main resonant vibrations of an elliptical plate in the presence of internal resonances. The results indicate that modes other than the driven mode can be drawn into the steady state response. In some cases, one or more of these other modes may dominate the response. Though the excitation is composed of a single harmonic, the response may not be periodic. Moreover, the particular types of responses that can occur are highly dependent on the mode being excited and are sensitive to small geometrical changes.

● **6.2-42** Iyengar, K. T. S. R. and Raman, P. V., **Free vibration of rectangular plates of arbitrary thickness,** *Journal of Sound and Vibration,* **54,** *2,* Sept. 22, 1977, 229-236.

Free vibration of thick rectangular plates is investigated by using the method of initial functions proposed by Vlasov. The governing equations are derived from the three-dimensional elastodynamic equations. They are obtained in the form of series, and theories of any desired order can be constructed by deleting higher terms in the infinite order differential equations. The numerical results are compared with those of classical, Mindlin, and Lee and Reismann solutions.

● **6.2-43** Rajamani, A. and Prabhakaran, R., **Dynamic response of composite plates with cut-outs, part I: simply-supported plates,** *Journal of Sound and Vibration,* **54,** *4,* Oct. 22, 1977, 549-564.

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The effect of square cutouts on the natural frequencies of square, simply supported composite plates is investigated. The forced and free dynamic response of plates with cutouts is formulated. Laminations are assumed to be symmetric about the midplane and the plates are considered analytically as homogeneous anisotropic plates. In the method of solution, it is assumed that the effect of the cutout is equivalent to an external loading on the plate. For free vibration, the method leads to an infinite system of frequency equations. Depending upon the accuracy required, a suitable size of the system of frequency equations is selected. Results are given for square, simply supported composite plates with centrally located square cutouts for different modulus ratios: Results obtained from this method for isotropic plates with cutouts are compared with available literature and excellent agreement is shown.

● **6.2-44** Rajamani, A. and Prabhakaran, R., **Dynamic response of composite plates with cut-outs, part II: clamped-clamped plates,** *Journal of Sound and Vibration,* **54,** *4,* Oct. 22, 1977, 565–576.

The forced and free dynamic response of plates with cutouts formulated in Part I (see Abstract No. 6.2-43) is used to investigate the effect of cutouts on the natural frequencies of clamped-clamped plates. The size, shape, and location of the cutout is expressed as a displacement dependent external loading. The plates considered are homogeneous and anisotropic. Lagrange's equations of motion lead to an infinite system of differential equations in time-dependent generalized coordinates with generalized forces which include the effects of the cutouts. There is an infinite system of frequency equations for free vibrations. The infinite system is truncated to a finite system of equations depending upon the accuracy desired in frequency values. Results are given for square, clamped-clamped plates with centrally located square cutouts for different modulus ratios. Good agreement is obtained when results for isotropic plates with cutouts are compared with available theoretical and experimental results.

● **6.2-45** Martins, R. A. F. and Owen, D. R. J., **Structural instability and natural vibration analysis of thin arbitrary shells by use of the semiloof element,** *International Journal for Numerical Methods in Engineering,* **11,** *3,* 1977, 481–498.

At present the semiloof element is probably one of the most efficient elements available for the solution of thin shells of arbitrary geometry. Experience in static situations indicates that accurate results can be obtained for nontrivial geometric and loading configurations with relatively coarse meshes. Variable thickness shells or discontinuous thicknesses can be accommodated, and no difficulties are encountered in modeling sharp corners or multiple junctions in structures. The element is basically nonconforming, but some measure of continuity is provided by the introduction of normal rotation variables at loof nodes on the element boundary. This paper examines the element behavior when applied to elastic instability and vibration situations. An eigenvalue solution scheme based on Sturm sequences is presented which does not require the usual elimination of a percentage of the total nodal variables by static condensation. Finally, the method is assessed by the solution of several numerical examples.

● **6.2-46** Laura, P. A. A., Luisoni, L. E. and Filipich, C., **A note on the determination of the fundamental frequency of vibration of thin, rectangular plates with edges possessing different rotational flexibility coefficients,** *Journal of Sound and Vibration,* **55,** *3,* Dec. 8, 1977, 327–333.

The title problem is solved by using a simple polynomial expression which identically satisfies the boundary conditions. A variational formulation is then applied and an approximate, but accurate and simple, frequency equation is generated. It appears that the literature does not contain a previous analytical treatment of this theme as general and simple as the one presented.

● **6.2-47** Dasgupta, G. and Sackman, J. L., **An alternative representation of the elastic-viscoelastic correspondence principle for harmonic oscillations,** *Journal of Applied Mechanics,* **44, Series E,** *1,* Mar. 1977, 57–60.

An alternative representation of the elastic-viscoelastic correspondence principle is derived for solids with identical damping characteristics in bulk and shear undergoing steady-state harmonic motion. This form is particularly useful when the elastic solution of the mechanical system is not available in closed form but is known only numerically, say as a tabular list. The analyticity property of the frequency response function is utilized to formulate a Dirichlet problem in the lower half of the complex plane with the elastic solution on the real line supplying the boundary data. The expression for the viscoelastic frequency response function is then obtained as an infinite integral in which the elastic frequency response and the viscoelastic parameters constitute the integrand. This integral may be evaluated numerically by quadrature to any desired degree of accuracy by suitably increasing the range of integration and employing a finer mesh. Any isolated singularity in the elastic response, like poles at the resonant frequencies, can be accurately handled by using exact complex integration in the sense of Cauchy principal value. A simple example is presented to illustrate an application of this alternative formulation.

● **6.2-48** Hashin, Z., **Vibration analysis of viscoelastic bodies with small loss tangents,** *International Journal of Solids and Structures,* **13,** *6,* 1977, 549–559.

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The correspondence principle for vibrations of viscoelastic bodies is specialized for the case of small loss tangents, resulting in considerable simplification. Analytical evaluation of oscillatory fields is greatly simplified; peak frequencies and peak amplitudes under forced vibrations can be simply and directly determined; numerical solution of viscoelastic vibration problems becomes no more complicated than that of elastic problems. Similar simplifications result for computation of real and imaginary parts of effective complex moduli of composite materials.

● **6.2-49** Frisch-Fay, R., **Stability functions for structural masonry,** *International Journal of Solids and Structures,* **13,** *5,* 1977, 381-393.

An analytical method for the calculation of the stiffness and carry-over coefficients for masonry beam-columns is presented. The masonry element is made of a material which can resist little or no tension. It is shown that the stability functions for the conventional frame element represent an upper or lower bound for the functions discussed. It is pointed out that, unlike in the conventional case, specific knowledge of the eccentricity ratio is required for a unique solution.

● **6.2-50** Ramkumar, R. L., Kamat, M. P. and Nayfeh, A. H., **Vibrations of highly prestressed anisotropic plates via a numerical-perturbation technique,** *International Journal of Solids and Structures,* **13,** *11,* 1977, 1037-1044.

The method of matched asymptotic expansions is used to reduce the problem of the transverse vibrations of a highly prestressed anisotropic plate into the simpler problem of the vibration of an anisotropic membrane with modified boundary conditions that account for the bending effects. In the absence of an exact solution, the membrane problem can be solved by any well-known numerical technique. The numerical-perturbation results for a clamped circular plate with rectangular orthotropy and a uniform tensile stress applied on its boundary show an excellent correlation with finite element solutions for the original problem. Furthermore, the solutions obtained for annular plates form the basis for solutions to problems involving near-annular plates.

● **6.2-51** Prathap, G. and Varadan, T. K., **Non-linear vibrations of tapered cantilevers,** *Journal of Sound and Vibration,* **55,** *1,* Nov. 8, 1977, 1-8.

By means of a variable separable assumption, an eigenvalue-like problem is formulated for the nonlinear free flexural vibrations of a cantilever beam. This eigenvalue problem is solved by a simple, relatively straightforward computational technique that provides an exact numerical solution to the problem, without any further assumptions as to the mode shape or periodicity of the time function. In this analysis, the time function is assumed to have certain features at the point of maximum amplitude; and with these assumptions, the mode shape is obtained, together with an eigenvalue consistent with the definitions of the time function. The mode shape is then used to obtain the time equation, which is solved numerically by the Runge-Kutta method to obtain the time period. The accuracy and efficacy of this approach to the problem is demonstrated by studying the nonlinear vibrations of cantilevered tapered beams. The results agree very well with those available in the literature for the linear case (small amplitude vibrations).

● **6.2-52** Reddy, J. N. and Tsay, C. S., **Stability and vibration of thin rectangular plates by simplified mixed finite elements,** *Journal of Sound and Vibration,* **55,** *2,* Nov. 22, 1977, 289-302.

Rectangular finite elements, based on a Reissner-type variational statement for plate bending, are applied to the stability and free vibration analysis of rectangular plates. The finite elements are constructed from the weak statements of the two normal moment-displacement relations and the moment equilibrium equation in terms of the two normal moments. These finite elements are algebraically simple and yield better accuracies for the critical loads and natural frequencies when compared to conventional finite elements. Linear and quadratic rectangular finite elements are used to calculate frequencies and buckling loads of rectangular plates with various edge conditions.

● **6.2-53** Richardson, R. S. H. and Nolle, H., **Energy dissipation in rotary structural joints,** *Journal of Sound and Vibration,* **54,** *4,* Oct. 22, 1977, 577-588.

Many structures and mechanical assemblies are held together by connections containing two friction surfaces in contact under the action of a constant clamping force. When the structure is vibrating, the external load acting on the joint is a time-dependent moment about an axis normal to the contact surfaces. Friction joints of this type constitute a well-defined source of damping in vibrating structures. The joint is analyzed theoretically by means of principles analogous to those used in the Panovko model of the axially loaded lap joint. Complete moment-rotation and energy loss characteristics are obtained. It is shown that the energy dissipated during cyclic loading depends on the peak-to-peak value of the dynamic part of the external moment but is independent of the mean moment. A measure of the efficiency of the joint in dissipating energy is defined, and a comparison drawn between the rotary (moment loaded) joint and linear (axially loaded) lap joints.

● **6.2-54** Downs, B., **Transverse vibrations of cantilever beams having unequal breadth and depth tapers,** *Journal of Applied Mechanics,* 44, **Series E,** *4,* Dec. 1977, 737-742.

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Natural frequencies of doubly symmetric cross-section, isotropic cantilever beams, based on both Euler and Timoshenko theories, are presented for 36 combinations of linear depth and breadth taper. Results obtained by a new dynamic discretization technique include the first eight frequencies for all geometries and the stress distribution patterns for the first four (six) modes in the case of the wedge. Comparisons are drawn wherever possible with exact solutions and with other numerical results appearing in the literature. The results display outstanding accuracy and demonstrate that it is possible to model with high precision the dynamic behavior of continuous systems by discretization onto a strictly limited number of degrees-of-freedom.

● 6.2-55 Gorman, D. J., **Free-vibration analysis of rectangular plates with clamped-simply supported edge conditions by the method of superposition,** *Journal of Applied Mechanics*, **44**, **Series E**, *4*, Dec. 1977, 743–749.

In this paper, attention is focused on the free vibration analysis of rectangular plates with combinations of clamped and simply supported edge conditions. Plates with at least two opposite edges simply supported are not considered as they have been analyzed in a separate paper. It is well known that the family of problems considered here have presented researchers with a formidable challenge over the years. This is because they are not directly amenable to Levy-type solutions. It has been pointed out in the literature that most of the existing solutions are approximate in that they either do not satisfy exactly the governing differential equation or the boundary conditions or both. In a new approach taken by the author, the method of superposition is exploited for handling these dynamic problems. It is found that solutions of any degree of exactitude are easily obtained. The governing differential equation is completely satisfied and the boundary conditions are satisfied to any degree of exactitude by merely increasing the number of terms in the series. Convergence is shown to be remarkably rapid and tabulated results are provided for a large range of parameters. The immediate applicability of the method to problems involving elastic restraint or inertia forces along the plate edges has been discussed in an earlier publication.

● 6.2-56 Shastry, B. P. and Rao, G. V., **Vibrations of thin rectangular plates with arbitrarily oriented stiffeners,** *Computers & Structures*, **7**, *5*, Oct. 1977, 627–629.

Free vibrations of plates with arbitrarily oriented stiffeners are studied using high precision plate bending and stiffener elements. Good convergence of frequency values for coarse mesh is demonstrated. Natural frequencies of square plates with various arrangement of stiffeners are determined for both simply supported and clamped boundary conditions.

● 6.2-57 Mindlin, R. D., **Frequencies of vibration of beams by Schelkunoff iteration with the HP-65 calculator,** *Computers & Structures*, **7**, *5*, Oct. 1977, 639–650.

Three programs are given for computing frequencies of free vibrations of uniform beams with the HP-65 programmable pocket calculator: Bernoulli beams with all 10 combinations of homogeneous end conditions, two-span Bernoulli beams on hinged supports, and Timoshenko beams with 9 of the 10 combinations of homogeneous end conditions. The programs illustrate the capacity and versatility of the calculator and various strengths and weaknesses of Schelkunoff's iterative method for solving trigonometric equations.

● 6.2-58 Banks, D. O. and Kurowski, G. J., **The transverse vibration of a doubly tapered beam,** *Journal of Applied Mechanics*, **44**, **Series E**, *1*, Mar. 1977, 123–126.

This article analyzes the transverse vibrations of a thin homogeneous beam which is symmetric with respect to the x-y and x-z planes. All combinations of clamped, hinged, guided, and free boundary conditions at both ends of the beam are considered.

● 6.2-59 Williams, F. W. and Howson, W. P., **Compact computation of natural frequencies and buckling loads for plane frames,** *International Journal for Numerical Methods in Engineering*, **11**, *7*, 1977, 1067–1081.

Existing theory is assembled to give a method which needs only the core of a minicomputer to calculate the eigenvalues of large rigidly jointed plane frames with certainty, the eigenvalues being natural frequencies and critical load factors in free vibration and buckling problems, respectively. The method is illustrated by annotated listings of vibration and buckling programs, each containing under two hundred Fortran statements and with low-number storage requirements. The use of the programs as "black boxes" is fully explained, with illustrative examples. The member theory used is the "exact" classical Bernoulli-Euler uniform member theory. Possible applications include evaluation of answers from approximate methods; calculation of critical loads for substitution in the modified Merchant-Rankine formula to estimate collapse loads of frames; and calculation of shifts in natural frequencies caused by structural damage, in connection with structural integrity monitoring of inaccessible structures.

● 6.2-60 Achenbach, J. D. and Khetan, R. P., **Elastodynamic response of a wedge to surface pressures,** *International Journal of Solids and Structures*, **13**, *11*, 1977, 1157–1171.

An elastic wedge of interior angle $\kappa\pi$, where $1<\kappa\leq 2$, is subjected to the impact of spatially uniform pressures on its faces. The application of the pressures produces a

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system of longitudinal waves, transverse waves, and head waves. In this paper, the elastodynamic stress singularity in the circumferential stress at the vertex of the wedge is analyzed. The analysis is based on self-similarity of first-order time derivatives of the displacement potentials. By means of appropriate transformations, the statement of the problem is reduced to two Laplace equations, whose solutions in half planes are coupled along the real axes. The solutions to this system are obtained by using elements of analytic function theory, together with summations over Chebyshev polynomials along the real axes.

● **6.2-61** Jones, R., **Approximate methods for the linear and nonlinear analysis of plates and shallow shells,** *Journal of Structural Mechanics,* **5**, *3*, 1977, 233–253.

Berger's equations for the large amplitude deformation of membranes are used to produce a simple approximate expression for the large amplitude deflection of plates. The deformation of shallow shells is also considered and two approximate methods are outlined. Several important problems are discussed, the obtained solution being in good agreement with both experimental data and other approximate results. The main advantage of this technique is its ease of application, as it requires comparatively little computational work. A simple approximate formula for computing the fundamental frequency of a vibrating shallow shell is also presented and is shown to yield very accurate values in the case of a shallow dome and a rectangular panel.

● **6.2-62** Basavanhally, N. and Marangoni, R. D., **Measurement of mechanical vibration damping in orthotropic, composite and isotropic plates based on a continuous system analysis,** *International Journal of Solids and Structures,* **13**, *8*, 1977, 699–707.

The problem of free and forced transverse vibration of orthotropic, composite, and isotropic thin square plates, with uniformly distributed damping and simply supported boundary conditions, has been solved using a modal expansion technique. A load of the type $P_0 \cos\Omega t$ applied at the center of the plate has been considered; and the phase angle between the forcing function and the vibration response at the center, as a function of the forcing frequency and the damping parameter, has been determined. This theoretical relationship, together with the experimentally measured phase angle between the applied mechanical forcing and the resulting vibration response at various forcing frequencies, was used to determine an equivalent viscous damping parameter. This technique has been found to be particularly useful for the measurement and comparison of the relative damping in composite or orthotropic materials. Also, a theoretical relation for the energy loss due to viscous damping in vibrating plates has been developed, and the theoretical energy loss at various frequencies has been compared with the experimentally measured energy loss at the same frequencies. Typical damping results are presented for aluminum, steel, and aluminum/graphite-fiber composite materials.

● **6.2-63** Watanabe, K., **Transient contact shear stress in a layered elastic quarter space subjected to anti-plane shear loads,** *International Journal of Solids and Structures,* **13**, *1*, 1977, 75–78.

In this paper, the transient behavior of a contact shear stress in a layered elastic quarter space subjected to anti-plane shear loads in investigated. The loads are suddenly applied to the upper and side edges of the layer. The effects of the reflected waves, the loaded position, and the material properties to the contact shear stress are shown graphically.

6.2-64 Magrab, E. B., **Natural frequencies of elastically supported orthotropic rectangular plates,** *The Journal of the Acoustical Society of America,* **61**, *1*, Jan. 1977, 79–83.

An expression is derived from which the natural frequencies of a rectangular orthotropic plate, under any combination of simply supported, elastically supported, or clamped boundary conditions, can be obtained. The Mindlin-Timoshenko theory, which includes the effects of transverse shear and rotary inertia, is used to describe the plate motion. The solution is obtained with a previously developed extension of the Galerkin technique. Comparison of results with the limited results of previous investigations is very good. New results are presented for the fundamental frequencies of rectangular and square plates for boundary conditions on all four edges that vary continuously from simply supported to clamped, and for various combinations of length-to-thickness ratios and material constants. Additional results are presented for orthotropic plates simply supported and clamped on all four edges.

6.2-65 Kazarinov, V. M., **Investigation of vibration damping of reinforced concrete beams beyond the limits of elasticity** (Issledouanie zatukhaniya kolebanii zhelezobetonnykh balok pri ikh rabote za predelom uprugosti, in Russian), *Beton i zhelezobeton, 12*, 1977, 26–27.

Results of investigations of the vibration damping properties beyond the limits of elasticity of flexible reinforced concrete structures subjected to seismic loads are presented. The effects of various factors on the damping coefficients of aseismic structures in the elastoplastic stage are investigated.

6.2-66 Rakhmanov, V. A., **Dynamic properties of reinforcing rods subjected to sudden shock loading** (Dinamicheskie svoistva sterzhnevoi armatury pri skorostnom impulsivnom nagruzhenii, in Russian), *Beton i zhelezobeton, 12*, 1977, 21–24.

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The dynamic properties of various classes of reinforcing steel (A-I, A-II, A-III, At-III, At-V, and At-VI) are discussed. Testing was carried out using the MTS electrohydraulic test apparatus operating in rigid mode. Shock loads were applied at four fixed deformation velocities. Test results were analyzed and the relationship between deformation velocity and the strength and deformation parameters of the reinforcement was calculated. The utilization of Campbell's dislocation criterion for the calculation of the behavior of reinforcements is shown to result in a substantial overestimate of their dynamic strength.

6.2-67 Nayfeh, A. H. and Kamat, M. P., **Numerical-perturbation technique for the transverse vibrations of highly prestressed plates,** *The Journal of the Acoustical Society of America,* **61,** *1,* Jan. 1977, 95–100.

Under the usual assumptions of small strains with moderately large rotations, the problem of the transverse vibrations of highly prestressed, nonuniform annular plates is reduced to the solution of the differential equation governing the transverse vibration of the corresponding prestressed membrane subject to modified boundary conditions that account for the effects of bending. The methods of matched asymptotic and/or composite expansions are used to determine these modified boundary conditions. The agreement of the results of both methods with known exact solutions for simple geometries demonstrates the efficiency of this technique when compared with other well-known numerical techniques.

6.2-68 Whitney, J. M. and Sun, C.-T., **Transient response of laminated composite plates subjected to transverse dynamic loading,** *The Journal of the Acoustical Society of America,* **61,** *1,* Jan. 1977, 101–104.

Transient solutions are presented for an infinitely long, simply supported composite plate subjected to either a uniform or line-concentrated dynamic pressure at the upper surface of the plate. A rectangular pulse, triangular pulse, and sinusoidal pulse are considered, and dynamic load factors are determined for maximum values of deflection, bending stress, and interlaminar shear stress as a function of pulse dwell time. The effect of pulse shape on deflection and stresses is also considered. Numerical results are obtained for graphite/epoxy symmetric angle-ply laminates.

6.2-69 Bhat, B. R. and Sinha, P. K., **Forced vibrations of simply supported orthotropic sandwich plates,** *The Journal of the Acoustical Society of America,* **61,** *2,* Feb. 1977, 428–435.

Force transmissibility, driving-point impedance, and transfer impedance of a simply supported rectangular orthotropic sandwich plate are investigated. The plate is driven by a sinusoidally varying point force either at the plate center or at any arbitrary location. The variation of force transmissibility and impedance with frequency, and the manner in which this is influenced by the various sandwich-plate parameters are studied and presented graphically.

● 6.2-70 ACI Committee 439, **Steel reinforcement properties and availability,** *Journal of the American Concrete Institute,* **74,** *10,* Title No. 74-44, Oct. 1977, 481–492.

The material properties of the various types of steel reinforcement produced for use in the U.S. are described. Deformed reinforcing bars, plain bars and wire, welded wire fabric, bar mats, and prestressing reinforcement are the reinforcement types examined. The requirements and restrictions of the pertinent ASTM specifications are reviewed. Included is a discussion of the test requirements of deformed reinforcing bars. The availability of the various types and sizes of reinforcement in the U.S. is also summarized.

● 6.2-71 Yang, T. Y., Kayser, K. W. and Shiau, L. C., **Theoretical and experimental studies of earthquake response of a chimney,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1241–1246.

A tall chimney is modeled using Bernoulli-Euler beam finite elements, and its time-history dynamic responses are analyzed using the method of modal superposition. The chimney includes two reinforced concrete shells, each with two flue openings. The full-scale measurement provides three natural frequencies which agree well with the computed values. The horizontal and vertical components of the 1940 El Centro earthquake are used as input in the analysis. Various damping coefficients are considered. The stresses around the flue openings at the most critical instant are computed using 3-D quadrilateral plate elements. Results are discussed.

6.2-72 Nagaya, K., **Vibrations of cross-supported viscoelastic plates,** *The Journal of the Acoustical Society of America,* **61,** *5,* May 1977, 1191–1197.

This paper discusses a free- and a forced-vibration problem of an elastically cross-supported rectangular viscoelastic plate with various edge conditions. A three-element viscoelastic model is adopted in the analysis. The result for the viscoelastic plate is obtained from the correspondence principle by applying the Laplace transform to the constitutive equation for the viscoelastic materials and to the equation of motion in terms of unknown forces, which are equivalent to reaction forces and resisting moments of the supports. Some numerical results are shown for steady-state and transient response problems.

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6.2-73 Sakata, T. and Sakata, Y., **Approximate formulas for natural frequencies of rectangular plates with linearly varying thickness,** *The Journal of the Acoustical Society of America,* **61**, *4*, Apr. 1977, 982–985.

A characteristic equation is derived analytically for a rectangular plate with thickness varying linearly in one direction by using the trigonometric series. Boundary conditions of the plate are simply supported along two opposite sides, free along one side and simply supported or free along the remaining side. By using the results computed numerically from the characteristic equation, approximate formulas are derived for estimating accurately the fundamental natural frequency of the plate.

● **6.2-74** Priestley, M. J. N. *et al.*, **Ductility of reinforced concrete bridge piers,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3150–3155.

The design, manufacture, and testing of five reinforced concrete bridge pier models under simulated seismic loading are described. Results from slow cyclic testing of four units show that adequate ductility can be obtained, but that American Concrete Inst. recommendations for confining steel may be inadequate to prevent buckling of vertical compression steel. Dynamic testing of a fifth model produced results which agree well with the static test results and with predictions of an inelastic time-history computer analysis.

● **6.2-75** Oesterle, R. G., Fiorato, A. E. and Corley, W. G., **Free vibration tests of structural walls,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3239–3240.

As part of an experimental and analytical investigation of structural walls for earthquake-resistant buildings, large isolated reinforced concrete specimens have been tested under reversing in-plane lateral loads. Free vibration tests were carried out to determine the frequency and damping characteristics of isolated walls. These tests were conducted at selected stages as the number and magnitude of the reversed lateral load cycles applied to the specimen were increased. Details of the test specimens are described in a paper under Topic 5 of this conference. Results of the free vibration tests are described in this paper.

● **6.2-76** Guha, S. K., Wedpathak, A. V. and Desai, P. J., **Aseismic tests on some electro-mechanical systems,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3328–3333.

Natural frequencies of some electromechanical systems estimated from aseismic tests in the laboratory are within the earthquake frequency range of 2 to 15 Hz, while some natural frequencies have increased beyond this range because of influences of supporting and foundation conditions at a site as revealed by in-situ tests. These systems behaved satisfactorily during an earthquake of Richter magnitude 7.0 and when subjected to simulated earthquake forces of 0.50 g (horizontal) and 0.25 g (vertical) on shaking tables.

● **6.2-77** Kaar, P. H. and Corley, W. G., **Properties of confined concrete for design of earthquake resistant structures,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 2098–2099.

As part of an experimental and analytical investigation of structural walls for earthquake-resistant buildings, elements of walls were tested to determine the effectiveness of rectangular confinement reinforcement.

● **6.2-78** Edgar, L. and Bertero, V. V., **Evaluation of contribution of floor system to dynamic characteristics of moment-resisting space frames,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3006–3012.

Parametric, finite element analyses are used to study the contribution of beam-slab floor systems to the overall stiffness of moment-resisting frames, and to establish graphs of the influence of different floor parameters on the floor stiffness. The graphs are the basis of a practical and sufficiently accurate method for computing the contribution of the floor system to the lateral stiffness of a moment-resisting space frame. The proposed method was found to be more accurate than other methods currently used.

● **6.2-79** Basak, A. K. and Gupta, Y. P., **Experimental determination of lateral stiffness characteristics of space framed three dimensional steel structures,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2907–2909.

The objective of this paper is to discuss the experimentally determined dynamic characteristics and the discrepancies in stiffness contributions of steel plane and space frame structures while subjected to lateral forces. The experimental investigations are carried out on two models, single-story and three-story single-bay plane and space frames. This preliminary study of force-deflection characteristics shows that the lateral stiffness of space frame structures is not a summation of plane frame stiffnesses. An attempt is being made to find a multiplying factor which relates the plane and space frame stiffnesses.

● **6.2-80** Kaar, P. H. and Corley, W. G., **Properties of confined concrete for design of earthquake resistant**

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structures, *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2902–2903.

As part of an experimental and analytical investigation of structural walls for earthquake-resistant buildings, elements of walls are being tested to determine the effectiveness of rectangular confinement reinforcement. Variables in the program include spacing and size of confinement reinforcement, amount of longitudinal reinforcement, concrete strength, and size of specimen. Analysis of the test results showed that all arrangements of rectangular hoops significantly increased the limiting concrete strain. For specimens with hoop reinforcement meeting the requirements of Appendix A of the ACI building code, limiting concrete strains exceeded 0.015. The spacing and amount of transverse reinforcement were the primary variables affecting the stress versus strain relationship of the concrete.

● 6.2-81 Barua, H. K. and Mallick, S. K., **Behaviour of one storey reinforced concrete frame infilled with brickwork under lateral loads,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3214–3220.

A predominantly experimental approach has been adopted for summarizing the results of twenty-one lateral tests of reinforced concrete frames infilled with brickwork. The test data furnish new information on the influence of the quality of brickwork on the modes of failure, stiffness, strength, and the sharing of the load between the frame and the infilled material. Simple, nondimensional expressions for lateral stiffness, strength, and the sharing of the load between the frame and the infilled material are proposed.

● 6.2-82 Huang, C. L. D. and Al-Khattat, I. M., **Finite amplitude vibrations of a circular plate,** *International Journal of Non-Linear Mechanics,* 12, *5,* 1977, 297–306.

The problem of finite-amplitude, axisymmetric free and forced vibration of a circular plate with various boundary conditions is examined. The nonlinear boundary-value problem is converted into the corresponding eigenvalue problem by elimination of the time variable. Then, using a Newton–Raphson iteration scheme and the concept of analytical continuation, the solution to the nonlinear eigenvalue problem for the vibrations is obtained in a discrete form. It is seen that the removal of radial restraint causes severe changes in the plate responses and the patterns of membrane stresses. Comparison with solutions based on the Berger assumption reveals the unsuitability of the assumption when the plate is not radially restrained.

● 6.2-83 Vendham, C. P., **An investigation into nonlinear vibrations of thin plates,** *International Journal of Non-Linear Mechanics,* 12, *4,* 1977, 209–222.

The variational and modified forms of the von Karman-type nonlinear plate equations are considered in the context of the Rayleigh–Ritz and Galerkin methods. An approximate analysis of the nonlinear vibrations of thin elastic plates, including inplane inertia, is presented. The quantitative study confirms that the inplane inertia effects are negligible for thin plates provided the nonlinearity is not too large. It is observed that the nonlinear inertia terms in the transverse equation of motion should be retained in any such study. The analysis is simplified by neglecting the inplane inertia and applied to constrained and unconstrained plates. A different type of inplane boundary condition, termed the partially constrained condition, is studied; and the inadequacy of replacing the unconstrained condition by means of an average zero stress condition is clearly demonstrated. It is observed that in most of the cases considered the Galerkin method yields lower bounds for the nonlinear coefficient of the modal equation. In all cases, the Galerkin results yield less stiff models than the Rayleigh–Ritz method. The general significance of the convergence of the two methods beyond the scope of the title problem is highlighted.

● 6.2-84 Liauw, T. C. and Lee, S. W., **On the behaviour and the analysis of multi-storey infilled frames subject to lateral loading,** *Proceedings, The Institution of Civil Engineers,* Part 2, 63, Paper 8052, Sept. 1977, 641–656.

Multistory infilled frames, consisting of four-story steel frames with reinforced concrete infills with and without openings, and also with and without connectors between the frames and the infills, are experimentally investigated and analytically examined in terms of their strength and stiffness. Two analytical methods are presented: one for infilled frames without connectors and the other for infilled frames with connectors. The analytical and experimental results are compared and the important role of the connectors in improving the strength, stiffness, and reliability of the infilled frames is shown.

● 6.2-85 Vallenas, J., Bertero, V. V. and Popov, E. P., **Concrete confined by rectangular hoops and subjected to axial loads,** *UCB/EERC-77/13,* Earthquake Engineering Research Center, Univ. of California, Berkeley, Aug. 1977, 128. (NTIS Accession No. PB 275 165)

Results of an experimental investigation carried out on concrete confined by rectangular hoops subjected to axial loads are reported. The behavior of the confined concrete is studied regarding the effect of three parameters: (1) concrete cover, (2) lateral reinforcement (confinement), and (3) longitudinal reinforcement. Fourteen prismatic specimens were tested. These were subjected to monotonically increasing axial compressive loads applied at low strain rates. The experimental results obtained are then evaluated and subsequently compared with the results predicted by other researchers.

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Current methods of predicting behavior for axially loaded confined concrete are shown to be inaccurate insofar as estimating the maximum stress levels, the strain values corresponding to the maximum stress levels, and the strain values in the descending branch of the longitudinal stress-strain curve. An analytical stress-strain relationship is suggested which considers the increase in strength and ductility caused by the lateral and longitudinal reinforcement and which gives better agreement with the experimental results obtained than those predicted using current stress-strain relationships.

● **6.2-86** Omote, Y. *et al.*, **A literature survey-transverse strength of masonry walls,** *UCB/EERC-77/07*, Earthquake Engineering Research Center, Univ. of California, Berkeley, Mar. 1977, 145. (NTIS Accession No. PB 277 933)

This literature survey collates most of the available relevant information on the transverse, or out-of-plane, strength of masonry walls. The report discusses several of the test techniques used and summarizes the most significant available test results. Formulations for predicting the capacity of walls subjected to transverse loads are presented, together with their correlation with experimental results. Also included is a section relating test results to present design practices and code requirements.

● **6.2-87** Cameron, G. R., **Stiffness coefficients of conductors,** *ME-77-2*, Bonneville Power Admin., U.S. Dept. of the Interior, n.p., Feb. 16, 1977, 14.

The purpose of this study was to obtain the stiffness coefficients of the conductors and conductor bundles at tower number 4 of the Moro UHV Mechanical Test Line. The structural dynamic analysis of that tower, after the conductors have been strung, requires the use of those coefficients. An approximate analytical procedure was developed, and a computer program used for icing loads on towers was adapted to predict the stiffness coefficients. Comparison of the computed and measured values are presented.

● **6.2-88** Neille, D. S., **Behaviour of headed stud connections for precast concrete panels under monotonic and cycled shear loading,** *Structural Research Series Report No. 20*, Dept. of Civil Engineering, Univ. of British Columbia, Vancouver, Oct. 1977, 224.

The research on headed stud connections described in this report forms a part of an overall objective of predicting the behavior of precast concrete panel buildings under earthquake loads. Existing laboratory test data and current design procedures of headed stud connections are briefly reviewed.

It is postulated that shear loads are transmitted via a connection to the surrounding concrete by three distinct mechanisms: (1) friction between faceplate and concrete, (2) bearing of end of faceplate on concrete, and (3) interaction between studs and concrete. Tests on laboratory models designed to isolate individual aspects of these mechanisms confirm that all three exist. Friction forces between faceplate and concrete are small in comparison with the remaining forces acting in a connection, particularly under cycled loading. Bearing of the end of the faceplate on concrete, and interaction between studs and surrounding concrete are shown to be the main contributions to the load carried by a connection. A simple analytical model is presented for the prediction of the ultimate shear load capacity of a connection, and a computer algorithm is proposed for the prediction of the load versus deflection behavior of a connection under both monotonic and cyclic conditions.

Existence of the three mechanisms whereby a connection transfers applied shear forces to the surrounding concrete contradicts the shear friction concept which is currently used in the design of connections. The analytical equations developed in the investigation indicate that the strength of a connection is directly dependent upon the strength of the surrounding concrete, as opposed to the expression for shear friction, which does not contain concrete strength as a variable.

● **6.2-89** Shimazu, T. and Hirai, M., **Confinement effects of web reinforcements on reinforced concrete columns,** *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. III, 1977, 3120–3126.

In this study, the axial stress-strain relationships of columns are established, taking into consideration the confinement effects of certain variables in web reinforcement, such as amount, diameter, spacing, and enclosing shape (circular or square). These relationships are based on test results for concentric axial loading. Next, the results of tests performed on reinforced concrete square columns subjected to eccentric axial loading with different amounts of web reinforcement are discussed. The moment-curvature relationships of the test results are examined, using the previously established axial stress-strain relationships and using the strain distribution of a hoop bar.

● **6.2-90** Cismigiu, A. I. and Dogaru, L. C., **Brittle matter ductilization in structures,** *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. III, 1977, 3056–3061.

The division of the properties of materials into ductile and brittle are a matter of convention, when, in fact, the response of materials essentially depends upon the conditions to which they are subjected. Detailed lab tests show

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that, under normal conditions, concrete behaves like a brittle body, but under triaxial compression shows high ductility and a significant increase in compressive and shear strength. In contrast, normally ductile steel shows splitting and brittleness under high-strength triaxial tension. It is suggested that testing of stress states to determine the point at which brittle material becomes strong and ductile has significant technical implications.

● **6.2-91** Zhunusov, T. Zh. and Bespayev, A. A., **Effect of amount and type of reinforcement on behaviour of reinforced concrete elements under seismic impulse excitation,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3233–3238.

Experimental data on the impulsive dynamic loading behavior of ordinary and prestressed reinforced concrete beams with reinforcement bars of class A-III hot-rolled steel; class A-IIIB hot-rolled, strengthened-by-drawing steel; and class AT-VI hot-rolled, thermally strengthened steel are presented. Stiffness, dynamic behavior, energy capacity, and vibration damping values are compared. Advantages of high-strength steels in earthquake engineering are shown, and inadmissibility of heavily reinforced sections is emphasized. Formulas for evaluation of the reinforcement limit are presented, and recommendations for calculation of stiffness and crack resistance under dynamic loading are given.

● **6.2-92** Bleich, H. H., **Strain energy expressions of rings of rectangular, T- and I-section, suitable for the dynamic analysis of ring-stiffened cylindrical shells,** Dept. of Civil Engineering and Engineering Mechanics, Columbia Univ., New York, New York, Oct. 1976, 32.

Strain energy expressions are obtained for rings of rectangular, T-, and I-section. The expressions are intended for use in the dynamic analysis of ring-stiffened cylindrical shells. The approach is essentially a generalization of the conventional, approximate analysis of straight beams, i.e., the influence of shear stresses, and of direct stresses at right angles to the axis of the beams is neglected.

● **6.2-93** Chen, W.-F. and Atsuta, T., **Theory of beam-columns: in-plane behavior and design,** Vol. 1, McGraw-Hill Book Co., New York, 1976, 530.

The book presents principles and methods for analysis of beam-columns and shows how beam-column theory may be used in the solution of practical design problems. The following topics are covered: stress-strain relations; elastic stability of columns; special topics in elastic stability of columns; moment-curvature relations of column segments; plastic buckling of columns; analysis of elastic and plastic beam-columns; deflection method for plastic beam-columns; curvature method for plastic beam-columns; curvature method–column curvature curves; curvature method–examples; moment method and others for plastic beam-columns; approximate methods for ultimate strength of beam-columns; design of columns and beam-columns; and probabilistic analysis of column strength. Answers to some selected problems are given.

● **6.2-94** Bonaldi, P. *et al.,* **Concrete dam problems: an outline of the role, potential and limitations of numerical analysis,** Ist. Sperimentale Modelli e Strutture, Bergamo, Italy, 1976, 53.

● **6.2-95** Sekhniashvili, E. A., **On the effective use of light concrete and reinforced concrete in construction in seismic regions,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 2034–2040.

The advantages of using lightweight and reinforced concrete in structures include less weight, better thermotechnical and acoustical properties, and improved fire resistance. In industrial construction, the use of larger mounted parts is made possible since the built-up members weigh less. These advantages are greater in seismic regions, since the smaller the structure's weight, the lower the seismic loads acting upon it. In the seismic districts of the U.S.S.R., lightweight and reinforced concrete are widely used for enclosing structures, floors, and roofing of residential and public buildings; however, they are rarely used for the supporting structures designed for horizontal seismic loads.

Significant economic effects can be achieved by using lightweight and reinforced concrete in supporting and enclosing structures. These concretes ensure higher structural flexibility, as a result of the lightweight concrete's elastic modulus, which is lower than that of common types of concrete, and its higher oscillation energy. An equation is given to justify these conclusions.

● **6.2-96** Bostjancic, J., **Model materials suitable for dynamic tests of models in the plastic range,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2745–2752.

Dynamic model tests in the plastic range can be carried out at the Inst. for Research and Testing in Materials and Structures, Ljubljana, with existing equipment. However, the laws of model similarity must be known, and suitable model materials which can fulfill the necessary conditions, ideally, the conditions of complete model similarity, must be found. This is being investigated. In this

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report, suggestions are given as to how the dynamic characteristics of materials can be determined and how reinforcement can be effected with materials which have the same properties as the reinforcement materials for the prototype.

6.2-97 Farah, A., Ibrahim, I. M. and Green, R., **Damping of floor vibrations by constrained viscoelastic layers,** *Canadian Journal of Civil Engineering,* **4,** *4,* Dec. 1977, 405–411.

Formulations are presented for enhancing the serviceability of one-way floor systems subjected to dynamic loading through the use of constrained viscoelastic layers. The constrained layers are combined with the floors and the resulting systems are analyzed as sandwich structures using a double Fourier series approach. Results indicate that the damping of the resulting sandwich beams is governed by factors related to the elastic and geometric properties of the constrained layer and the constraining system (i.e., cover plates and beams) and the loss modulus of the viscoelastic material, and is highly influenced by the location of the viscoelastic layer in the sandwich beams. Optimum designs of the sandwich beams are obtained using the box algorithm optimization technique.

6.2-98 Rosman, R., **Approximate analysis of regular spatial frameworks,** *Building and Environment,* **12,** *2,* 1977, 97–110.

Simple design formulas and coefficient tables are given for regular spatial frames. The cross sections of the columns are either constant along their height or they increase toward the bottom. The design aids cover all the governing factors: the extreme values of the bending moments of the columns and of the lintels, the extreme values of the lateral deflection of the frame and of the relative lateral deflection between two adjacent floors, the lowest buckling loads, and the fundamental vibration periods. The formulas and coefficient tables enable simple design and preliminary analyses to be made in early and advanced planning stages.

6.2-99 Gibson, R. F. and Plunkett, R., **A forced-vibration technique for measurement of material damping,** *Experimental Mechanics,* **17,** *8,* Aug. 1977, 297–302.

This paper describes a technique for measuring material damping in specimens under forced flexural vibration. Although the method was developed for testing fiber-reinforced composite materials, it can be used for any structural material. The test specimen is a double-cantilever beam clamped at its midpoint and excited in resonant flexural vibration by an electromagnetic shaker. Under steady-state conditions, material damping is defined in terms of the ratio of input energy to strain energy stored in the specimen. If external losses are negligible, the input energy must equal the energy dissipated in the specimen. Input energy and strain energy are found from measured specimen dimensions, resonant frequency, input acceleration, and bending strain. Problems associated with minimization of external energy losses in the apparatus and verification of measurements are discussed in detail. Measured damping of aluminum-alloy calibration specimens shows good agreement with calculated thermoelastic damping. Examples of measured damping showing amplitude and frequency dependence in fiber-reinforced plastic materials are presented.

6.2-100 Dafalias, Y. F. and Popov, E. P., **Cyclic loading for materials with a vanishing elastic region,** *Nuclear Engineering and Design,* **41,** *2,* Apr. 1977, 293–302.

Within a plastic internal variables formalism of rate independent plasticity, a recently developed constitutive law for plastic material response under stress reversals is shown to apply to materials with a vanishing elastic region. The concept of the bounding surface introduced earlier replaces the vanished yield and loading surfaces in defining loading-unloading criteria. Good comparison of the model with experimental data is obtained.

● 6.2-101 Cameron, G. R., **Dynamic testing—tower number four—strung,** *ME-77-6,* Bonneville Power Admin., U.S. Dept. of the Interior, Portland, Oregon, 1977, 37.

6.3 Dynamic Properties of Linear Structures

● 6.3-1 Chlaidze, N. Sh., **Space stresses in the arch dam of the Inguri hydroelectric plant under a special combination of loads** (Raschet prostranstvennogo napryazhennogo sostoyaniya arochnoi plotiny Inguri GES dlya osobogo sochetaniya nagruzok, in Russian), *Seismostoikoe stroitelstvo, 6,* 1977, 8–13.

Stresses arising in the arch dam of the Inguri hydroelectric power plant due to special load combinations (in particular, seismic loads) are investigated. The stresses are calculated from test load data using a computer algorithm. The loads taken into account include hydrostatic pressure, the weight of the dam, thermal stresses due to average temperature, horizontal seismic inertial loads on the downstream side of the dam, and seismic pressure.

● 6.3-2 Kuranishi, S. and Takahashi, T., **Vibrational characteristics of bridges with damping devices,** *Proceedings of U.S.-Japan Seminar on Earthquake Engineering Research with Emphasis on Lifeline Systems,* Japan Society for the Promotion of Earthquake Engineering, Tokyo, Nov. 1976, 119–139.

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The effect and utility of damping devices installed in bridges are discussed. The damping capacity and characteristics increased by damping devices are studied theoretically for simplified models, namely a cantilever and cable structure. Using these results, numerical and experimental studies have been conducted for a suspension bridge with dampers.

● 6.3-3 Lin, C.-W. and Liu, T. H., **A discussion of coupling and resonance effects for integrated systems consisting of subsystems,** *International Seminar on Extreme Load Conditions and Limit Analysis Procedures for Structural Reactor Safeguards and Containment Structures,* U 3/3, 11. (For a full bibliographic citation, see Abstract No. 1.2-2.)

It is customary to conduct a seismic analysis of a nuclear power plant by dividing systems according to design responsibilities and classifications. To account for the possible interaction effects between the systems, many analysts have adopted the approach of either taking into account the stiffness or the weight of the interacting systems which are not considered an integral part of the model.

In this paper, three representative cases have been studied to evaluate the interaction effect and to establish the need to include both stiffness and mass of the interacting systems in the system model. The first case is a system supported by a two degree-of-freedom system. The second case represents two single degree-of-freedom systems, each supported by itself but interconnected by a spring. The third case represents a single degree-of-freedom system supported by another single degree-of-freedom system. In each of the three cases studied, the interaction effect is first measured by the errors incurred in the natural frequencies, for both the supported and the supporting systems. The conclusions are presented.

● 6.3-4 Tsushima, Y., Jido, J. and Mizuno, N., **Analysis of equations of motion with complex stiffness mode superposition method applied to systems with many degrees of freedom,** *International Seminar on Extreme Load Conditions and Limit Analysis Procedures for Structural Reactor Safeguards and Containment Structures,* T 1/3, 18. (For a full bibliographic citation, see Abstract No. 1.2-2.)

In this paper, the authors propose a simple method for expressing the damping effects of a structure using complex numbers, although viscous damping mechanics is generally used as the conventional mathematical method. The concept of complex damping is described comparing it with the Voigt model for a system with a single degree-of-freedom, and it is concluded that both solutions are identical under the conditions of free and forced vibration when both systems have equivalent natural periods and damping ratios. In addition, the authors apply the above complex stiffness to systems with many degrees-of-freedom and investigate the mathematical and dynamic characteristics of the multisystems. Based upon studies, the method presented is reasonable for estimating the damping effects of multidegree-of-freedom structures.

● 6.3-5 Gerdes, L. D., **Dynamic structural analysis of uncoupled subsystems,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(b), Paper K 6/18, 15. (For a full bibliographic citation, see Abstract No. 1.2-5.)

Analytical techniques and results are presented for dynamic seismic analysis of uncoupled structures. It is concluded that an uncoupled subsystem dynamic analysis can produce results essentially identical to those from a coupled analysis. The techniques utilized and conclusions obtained are applicable in general for any system/subsystem configuration. A nuclear power plant primary structure and a pressurized water reactor coolant system are used as the system/subsystem example. To develop general and realistic results, soil-structure interaction, nonuniform damping, and representative structural stiffnesses were considered in developing the results. Standard time-history modal analysis techniques were used. However, the conclusions are not dependent on the solution techniques.

● 6.3-6 Lin, Y. J. and Hadjian, A. H., **The effect of rotatory inertia on the dynamic response of cantilever structures,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(a), Paper K 3/13, 10. (For a full bibliographic citation, see Abstract No. 1.2-5.)

For the dynamic response of cantilever beams, the error introduced by bending theory becomes significant as the ratio of the radius of gyration to the beam length exceeds 0.1. In this case, the use of Timoshenko's beam equation becomes more appropriate. This equation includes, in addition to the bending effects, both shear deformation and rotatory inertia effects. In the discrete modeling of beam elements, both the shear deformation and rotatory inertia terms play roles in the mass matrix, while only the shear deformation terms appear in the stiffness matrix. The effect of rotatory inertia on the frequencies and dynamic response of cantilever structures subjected to lateral earthquake excitation is thoroughly studied. This is done by using both the consistent and lumped mass matrices and an analytical solution. The beam support is treated either as fixed or elastically restrained to consider soil-structure interaction effects. Since containment structures can be treated as hollow beams, a cantilever beam of uniform cross section is examined first. For those cases where the ratio of the radius of gyration to beam length lies within the range of interest, all the solutions show that rotatory inertia has an important impact on both the frequencies (other than that of the fundamental mode) and the vertical component of the response. However, as the soil-structure interaction effects become significant, rotatory inertia effects become secondary. For shear wall structures used in nuclear power plants, the floors may be treated as rigid diaphragms and the shear walls between floors are usually considered to be beam elements of uniform cross section. To examine the effect of the rotatory inertia, studies are made on cantilever beams with concentrated masses and mass moment of inertias acting at floor levels. The results from all the solutions indicate that the inclusion of the floor inertias significantly affects the frequencies and the vertical component of the response. Both types of models suggest that closer attention should be given to the modeling of rotatory inertia effects in some classes of structural systems. These are graphically identified in the paper.

● **6.3-7** Kelly, T. E., **Uniaxial dynamic analysis of a six storey reinforced concrete framed structure,** *Research and Development Report 77-1,* New Zealand Ministry of Works and Development, Wellington, 1977, 30.

A six-story reinforced concrete frame was analyzed inelastically using a two-dimensional dynamic analysis computer program. The program utilizes the step-by-step integration method for the solution of the equations of motion. Two natural and three artificial acceleration records were used. Harmonic motions of differing frequencies and each followed by a relatively long acceleration pulse were studied using artificial records created for the purpose. An approximate allowance for the effects of concurrent earthquake excitation in the orthogonal direction was made by reducing the available column capacities. Parameters studied were ductility requirements, deformations, and column forces.

● **6.3-8** Warburton, G. B. and Soni, S. R., **Resonant response of orthotropic cylindrical shells,** *Journal of Sound and Vibration,* **53,** *1,* July 8, 1977, 1-23.

The resonant response of simply supported, thin and thick, orthotropic cylindrical shells is determined by using modal analysis for three levels of hysteretic damping. This response is significantly changed if the larger of the elastic constants associated with normal stresses refers to the meridional, instead of the circumferential, direction. These changes may be related to changes in the relevant natural frequency. The causes of the latter are studied by considering the modal amplitudes and the contributions to the strain energy function associated with the various stress resultants.

● **6.3-9** Saito, H. and Wada, H., **Forced vibrations of a mass connected to an elastic half-space by an elastic rod or a spring,** *Journal of Sound and Vibration,* **50,** *4,* Feb. 22, 1977, 519-532.

An analysis of the forced vibrations of a rigid mass connected to an elastic halfspace by an elastic rod or a spring and subjected to a harmonic disturbing force in the direction of the rod or the spring axis is presented. The response curves of a rigid mass are obtained, for both uniform normal stress distribution and uniform normal displacement at the interface between a rod and a halfspace. Numerical results are presented for three kinds of rod and halfspace material combinations, and the effects of stiffness, mass, and damping of an elastic halfspace on the response curves of the system are shown. The force transmissibility curves are also given.

● **6.3-10** Fawzy, I., **A theorem on the free vibration of damped systems,** *Journal of Applied Mechanics,* **44,** Series E, *1,* Mar. 1977, 132-134.

A damped linear system may oscillate freely (with a decaying motion) in modes. Certain sufficient conditions have been found under which those modes are the principal modes defined in the absence of damping. A simple condition is established in this paper that is both necessary and sufficient for this to be the case. It admits other, hitherto undisclosed, conditions under which this simplification can validly be adopted.

● **6.3-11** Darwin, D. and Pecknold, D. A., **Analysis of cyclic loading of plane *R/C* structures,** *Computers & Structures,* **7,** *1,* Feb. 1977, 137-147.

A numerical procedure for cyclic loading response of planar reinforced concrete structures is presented. A nonlinear orthotropic stress-strain law for biaxially loaded plain concrete is developed and compared with experimental results for monotonic biaxial loading and uniaxial cyclic loading. The stress-strain law recognizes strength and ductility changes due to biaxial stress, and strength and stiffness degradation with cycles of loading. The stress-strain law is incorporated into a finite element computer program which utilizes isoparametric quadrilaterals with extra nonconforming deformation modes. Numerical and experimental results are presented for a monotonically loaded shear wall-frame system and a cyclically loaded shear wall.

● **6.3-12** Rasmussen, M. L., **On the damping decrement for non-linear oscillations,** *International Journal of Non-Linear Mechanics,* **12,** 2, 1977, 81-90.

The logarithmic damping decrement is obtained as a function of arbitrary nonlinear restoring forces and arbitrary, but small, nonlinear damping forces. General expressions are obtained for both amplitude-dependent and speed-dependent damping. The special case of a cubic restoring force with quadratic amplitude-dependent damping and the special case of a cubic restoring force with

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quadratic speed-dependent damping are considered in detail. The results of the analysis suggest how experimental data can be utilized to identify and evaluate the damping parameters for a given nonlinear oscillator.

● **6.3-13** Kelly, T. E., **Uniaxial dynamic analysis of a six storey reinforced concrete framed structure,** *Bulletin of the New Zealand National Society for Earthquake Engineering,* **10**, *1*, Mar. 1977, 37-51.

A six-story reinforced concrete frame was analyzed inelastically using a two-dimensional dynamic analysis computer program. The program utilizes the step-by-step integration method for the solution of the equations of motion. Two natural and three artificial acceleration records were used. Harmonic motions of differing frequencies, each followed by a relatively long acceleration pulse, were studied using artificial records created for the purpose. An approximate allowance for the effects of concurrent earthquake excitation in the orthogonal direction was made by reducing the available column capacities. Parameters studied were ductility requirements, deformations, and column forces.

● **6.3-14** Tyler, R. G., **Damping in building structures by means of PTFE sliding joints,** *Bulletin of the New Zealand National Society for Earthquake Engineering,* **10**, *3*, Sept. 1977, 139-142.

The characteristics of PTFE (polytetrafluoroethylene, a fluorocarbon resin, known under the trade name Teflon) sliding joints, when used to separate secondary components from the main structure of a building, are described. Their use enables damping of earthquake and wind motions to be obtained within the normal elastic range of the structure, and beyond, by virtue of the frictional forces generated, while temperature movements are at the same time accommodated. In addition, a positive fixing is obtained, which is a useful alternative to the hung-type of fixing that is at present recommended where separation is required. It is concluded that further testing is needed to obtain more friction data on filled PTFE materials, and suitable joints need to be developed.

6.3-15 Nayfeh, A. H. and Mook, D. T., **Parametric excitations of linear systems having many degrees of freedom,** *The Journal of the Acoustical Society of America,* **62**, *2*, Aug. 1977, 375-381.

The method of multiple scales is used to analyze parametrically excited linear systems having many degrees-of-freedom and distinct natural frequencies. Explicit second-order expressions are obtained for the characteristic exponents which yield the transition curves. Various combination resonances are treated. The results are applied to the buckling of free-clamped columns under the influence of sinusoidally varying axial loads.

● **6.3-16** Udwadia, F. E., **Uniqueness problems in structural identification from strong motion records,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1010-1015.

Problems in the identification of the structural parameters of a building system from strong-motion records have been investigated. Concepts of local and global nonuniqueness have been introduced. Through an analysis of a linear N-degree-of-freedom system used to model an N-story structure, conditions under which global uniqueness follow have been found. The studies indicate that global nonuniqueness may be obtained by proper instrument location and point out that the roof and basement records do not have sufficient information to uniquely determine the estimates of stiffness and damping. It has been shown that nonuniqueness may occur locally even when the initial guess is close to the true parameter values.

● **6.3-17** Joseph, M. G. and Radhakrishnan, R., **A damping model for response analysis of multistoreyed buildings,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1124-1129.

An experimental study of the damping pattern of building models during free and forced vibrations and an analytical study of a flexible mathematical method for incorporating any chosen intermodal damping in response analysis are discussed. A computerized case study of the influence of the damping matrix on the response of multistoryed buildings with and without joint rotations, on rigid and flexible foundations, is also discussed.

● **6.3-18** Ohno, S., Watari, A. and Sano, I., **Optimum tuning of the dynamic damper to control response of structures to earthquake ground motion,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1130-1134.

A method is presented for tuning a dynamic damper so that the mean square acceleration of an aseismic structure to which the damper is attached can be kept at a minimum. The acceleration power spectral density of earthquake ground motion at the base under the ground layer is assumed to be constant for a certain frequency range. The natural frequency and the damping of the dynamic damper which give optimum tuning are shown in a chart.

● **6.3-19** Arya, A. S., Chandra, B. and Gupta, S. P., **Seismic analysis of a complex turbo-generator building including torsion,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2639-2644.

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A turbogenerator building with complex frames has been studied for seismic forces, using three analytical models. The effects of joint rotations and axial deformations on the dynamic characteristics and the seismic response are studied. Suitability of block model and plane frame models for seismic analysis of such buildings is examined.

● 6.3-20 Abdel-Ghaffar, A. M. and Housner, G. W., **Vibrations in suspension bridges**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. II, 1977, 1288-1293.

A method of dynamic analysis is developed for the vertical, torsional, and lateral free vibrations of suspension bridges. The method is based on a linearized theory, a finite element approach, and use of the digital computer; and it incorporates certain simplifying features. The objective of the study is to determine a sufficient number of natural frequencies and mode shapes to enable an accurate analysis for practical purposes. The reliability of the analysis is illustrated by computing modes and natural frequencies of a real bridge, and comparing them with the measured natural frequencies for the vertical and torsional modes of vibrations.

● 6.3-21 Arya, A. S. and Kumar, K., **Effect of non-conventional bearings on the earthquake response of bridges**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. II, 1977, 1273-1279.

The rocker-roller arrangement conventionally used for supporting bridge spans imposes a large horizontal inertial force on the superstructure due to the rigid link between the superstructure and the substructure at the rocker bearing. This force acts eccentrically on the superstructure and may produce excessive vertical accelerations. The possibility of reduction in the inertial force has been investigated by providing end bearings of a sliding or rolling type together with external dampers and springs. A simple mathematical model has been developed for girder bridges. The results show that substantial reduction in inertial loads is possible using a suitable combination of rollers and dampers.

● 6.3-22 Norton, R. L. and Weingarten, V. I., **The effect of asymmetric imperfections on the earthquake response of hyperbolic cooling towers**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. II, 1977, 1343-1349.

Linear shell theory predicts that the response of axisymmetric cooling towers to base translation will be only of the beam-bending mode type, while experimental evidence indicates that higher-order shell modes have significant response. A linear computer program was used to evaluate several towers with various asymmetric imperfections. The results show that the bending stresses produced by the imperfections can be a substantial fraction of the conventional membrane stresses.

● 6.3-23 Zayed, M. M. and Lord, J., **Dynamic analysis of cable structures using large deflection theory**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. II, 1977, 1260-1266.

The application of a finite element approach in analyzing the dynamic response of geometrically nonlinear cable structures during periods of strong ground shaking is described. Cable structures referred to in this paper are those in which the overall structural stiffness is primarily governed by cable elements, i.e., elements which are highly sensitive to changes in geometry. Elements of this type violate the basic assumptions set forth in small deflection theory. Thus, large deflection theory must be used to formulate element stiffnesses in a structure of this type. A discussion of cable, truss, and beam-column element formulation is included. In large deflection theory, an element stiffness is a function of its deformation. Given the geometry of a cable structure under gravity loading condition, a "reference state" is obtained by means of an iterative procedure. An outline of the algorithm for this procedure is included. A "state" is defined when all the forces and deformations in the various elements of the structure are known. Once the gravity load state is established, the structure can be solved for any time-varying load function, e.g., a ground acceleration due to an earthquake. The time-dependent load function can be approximated as a series of step functions. A description of two analytical, step-by-step methods follow in which a reference state is established at the end of each time step and used to obtain a solution for the next time step. An approximate method using modal analysis after establishing the gravity load state is compared with the step-by-step analysis. The paper concludes with reference to a three-dimensional seismic dynamic analysis of a pipeline suspension bridge using the methods outlined.

● 6.3-24 Kelkar, V. S., Mazumdar, S. and Buch, J. D., **Earthquake analysis of a TV tower structure**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. II, 1977, 1388-1390.

Results of modal earthquake analysis of a television tower structure consisting of a stiff concrete lower shaft and a slender upper steel portion are presented. It can be seen from the mode shapes that for such composite structures the addition of a light slender top portion degenerates each mode of vibration of the stiff shaft into two modes. This fact should be recognized before one uses only the first few modes for calculating design forces.

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● **6.3-25** Arya, A. S., Thakkar, S. K. and Rani, P., **Approximate vibration analysis of a cable stayed bridge**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. III, 1977, 2868–2873.

An approximate method for vibration analysis of a cable-stayed bridge in the longitudinal direction has been presented, the results of which have been compared with those of a small-scale model test and an exact analysis. It has been found that the fundamental frequency of the structure as determined by the approximate method shows good agreement with the results of experimental and exact analyses.

● **6.3-26** Ishac, M. and Heidebrecht, A., **Dynamic response of asymmetric shear wall-frame building structures**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. II, 1977, 1363–1368.

This paper describes a method for the computation of the dynamic response of uniform tall buildings, comprising frames and shear walls. The example given in the paper shows that a dynamic analysis should be conducted because dynamic coupling amplifies the torsional response so that a static analysis will not adequately determine stresses and deformations.

● **6.3-27** Yang, T. Y., Gran, C. S. and Lo, H., **Theoretical study of earthquake response of a cooling tower**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. II, 1977, 1307–1312.

A cooling tower in the 1200-Mw fossil fuel steam generating plant at Paradise, Kentucky, (Tennessee Valley Authority) is studied. The hyperbolic paraboloidal reinforced concrete shell is modeled using 3-D orthotropic quadrilateral flat plate finite elements. The top ring beam and the bottom supporting columns are modeled by 3-D beam finite elements. Natural frequencies and normal modes are first found. Time history responses of 30 sec to the N-S component of the 1940 El Centro earthquake are then computed by the method of modal superposition. Only the modes with one circumferential wave are excitable by horizontal motion. The effect of viscous damping is considered. Results are discussed.

● **6.3-28** Yang, T. Y., Baig, M. I. and Bogdanoff, J. L., **Vibration of a large boiler supporting structure**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. II, 1977, 1313–1318.

Free vibration of the steam generator and its supporting structure of the Tennessee Valley Authority 1200 Mw fossil plant at Paradise, Kentucky, is studied. The steam generator is modeled by lumped masses connected by rigid bars. The hanger rods and ties are modeled by bar finite elements. The framing and floors are modeled by beam and plate finite elements, respectively. Fundamental frequencies and modes are first found for various versions of subsystems to check the computer program and the consistency of results. Three frequencies and modes are then found for the total system. Mode shapes are plotted. The torsional motion is found in the first mode.

● **6.3-29** Cheung, Y. K. and Delcourt, C., **Buckling and vibration of thin, flat-walled structures continuous over several spans**, *Proceedings, The Institution of Civil Engineers*, Part 2, 63, Paper 7959, Mar. 1977, 93–103.

The finite strip method is applied to the stability and free vibration analysis of thin, flat-walled structures continuous over several spans. The accuracy of the method is checked against known continuous beam results and the method is then applied to the analysis of a real structure. For the type of structures under consideration, the finite strip method is very economical in comparison with the standard finite element analysis.

6.3-30 Nobile, M. A. and Snowdon, J. C., **Viscously damped dynamic absorbers of conventional and novel design**, *The Journal of the Acoustical Society of America*, **61**, *5*, May 1977, 1198–1208.

The behavior of dynamic vibration absorbers of conventional and novel design has been investigated experimentally and found to compare closely with prediction. The dynamic absorbers were employed to suppress the transmissibility at resonance across a 1-df primary system. Initially considered was a dynamic absorber with a conventional mass-spring-dashpot configuration; the primary system was undamped. Subsequently considered were (1) so-called dual dynamic absorbers, and (2) a single, nominally undamped absorber, or two such absorbers, attached to the primary system after it had been damped heavily. The dual absorbers—a conventional viscously damped absorber used in parallel with a less massive undamped absorber—introduced a pronounced transmissibility trough without the appearance of unwanted "compensating" peaks at lower and higher frequencies. The attachment of a nominally undamped absorber to the heavily damped primary system also introduced a pronounced trough without giving rise to compensating peaks. Further, the attachment of two such absorbers introduced pronounced troughs at two frequencies that could be varied independently of one another. Thus, the novel absorber systems considered here behaved as mechanical "notch" filters, providing at specific "low" frequencies a high degree of isolation that other passive systems cannot duplicate without exhibiting a marked loss in isolation at neighboring frequencies.

● **6.3-31** Keightley, W. O., **Building damping by Coulomb friction**, *Proceedings, Sixth World Conference on*

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Earthquake Engineering, Sarita Prakashan, Meerut, India, Vol. III, 1977, 3043–3048.

Infilled wall panels made of concrete segments prestressed together are presented as a nondestructive means of consuming vibrational energy in structures. The panels conform to building frame distortion similar to a deck of cards distorting in shear, consuming energy through Coulomb friction along the slip surfaces. Interfloor Coulomb forces are determined for two buildings to approximate maximum response during the El Centro earthquake of the same buildings with 20% viscous damping. Coulomb forces chosen for the lowest stories are 6%-8% of a structure's weight. Residual displacements after the earthquake are small, but extra loading on frame members at joints is significant.

● 6.3-32 Guerra, O. R. and Esteva, L., **Equivalent properties and ductility requirements in seismic dynamic analysis of nonlinear systems**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. II, 1977, 2005–2012.

Ductility demands of a number of shear systems subjected to earthquake ground motion are studied. Variables covered include natural period and distribution of stiffness and strength throughout the building height. Stories are assumed to show elastoplastic behavior in shear, and the response is obtained by step-by-step integration of the equations of motion. An approximate equivalent linearization criterion is proposed, and its results are compared with those obtained by the step-by-step analysis.

● 6.3-33 Jerath, N., Udwadia, F. E. and Kapsarov, H., **Experimental and theoretical studies of time variations of structural properties during strong ground shaking**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. II, 1977, 2085–2092.

Studies of the variation of structural properties recorded in structures during actual ground shaking have been carried out. After modeling a structure as a linear time-variant single degree-of-freedom system, a systematic methodology for the analysis of such behavior from a knowledge of the basement and roof records has been developed. Utilizing such records and incorporating data from pre- and post-earthquake vibration tests, the time-varying coefficients of the differential equation, used to model the structure, have been identified using a minimum variance sequential filter. Polynomial forms of stiffness and damping variations have been investigated.

The technique has been used to analyze the response of a nine-story reinforced concrete structure during the San Fernando, California, earthquake of Feb. 9, 1971. The results indicate that significant changes in the stiffness and damping characteristics from those obtained during vibration tests could ensue during actual strong ground shaking, and that a proper knowledge of the pre-earthquake characteristics may be useful in identification of structural properties using strong ground motion records.

● 6.3-34 Ibanez, J., **A modal, energy reserve method, of calculation for earthquake-proof structures**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. II, 1977, 1854–1857.

An energy reserve method, based on the modal analysis of a reinforced concrete structure, is presented. The ductility of each member is defined. The 1940 El Centro, California earthquake N-S component, with 10% critical damping and ductility factor 2, is used. An example demonstrates that the method does not necessarily add empirical reduction factors to the instrumentally obtained data as does the conventional maximum force method. The process is entirely analytic.

6.3-35 Rebora, B. *et al.*, **Computational experience with non-linear dynamic analysis of thin reinforced concrete structures with the initial-stress approach**, *Nuclear Engineering and Design*, **42**, 2, July 1977, 381–390.

Computational experience gained in the area of the dynamic nonlinear analysis of thin reinforced concrete structures is described. Parasitic damping is shown to be related to the residual stress. Using only one modified three-dimensional isoparametric element across the thickness of a thin plate, capable of representing a bilinear stress distribution, the ultimate moment can be accurately calculated. The difficulties in reaching convergence when the stress resultants are redistributed throughout a structure are discussed. Formulas for initial stress when multiple cracks occur are derived.

● 6.3-36 Juhasova, E., **Seismic vibration of the multistoried structures with the stiff core**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. II, 1977, 1417–1418.

● 6.3-37 Joseph, M. G. and Radhakrishnan, R., **Analogies in earthquake and wind response of structures**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. II, 1977, 1760–1766.

The effects of gusts of wind on tall structures are analyzed deterministically, as is done in earthquake analysis. Numerical integration techniques and modal analysis are performed on single degree structures. A specific force response spectrum for wind effects is given. Results of an analytical case study of structures, using numerical integration techniques, are presented.

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● **6.3-38** Bukharbaev, T. Kh., **Theoretical and experimental investigations of seismic-resistant box-unit structures,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 2052–2057.

New results of theoretical and experimental investigations of the earthquake resistance of thin-wall box-unit spatial structures are reported. A technique is proposed for determining the stress-strain condition of box-units, and the spatial mode of their behavior is considered. The technique shows good agreement with experimental data. Disagreement is observed between calculation results obtained from the spatial design method and those from the traditional plane design method. The full-scale tests of a three-story box-unit structure under vertical, lateral, static, and dynamic loads are described. The analysis and experimental data reveal qualitative and quantitative distribution of stress and strain in box-unit walls under lateral and vertical loads. It is concluded that the use of box-unit systems in earthquake engineering would be sound practice.

● **6.3-39** Cardona, R. and Esteva, L., **Static analysis of asymmetric multistory structures,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1999–2004.

A simplified criterion for the computation of dynamic amplification factors for twisting moments in multistory systems is proposed. By means of a generalized-coordinates approach and proper approximations to the first translational, torsional, and combined modes, applicability of results valid for single-story systems is extended to multistory structures. The accuracy of the approximate procedure is verified by comparing its results with those produced by standard modal analysis.

● **6.3-40** Belogorodsky, B. A., Malyshev, L. K. and Panteleev, A. A., **The solution of seismic stability problems on small-scale models by optic methods,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2851–2855.

Determination of the stress state of hydraulic structures under seismic impact, which is especially important for high or wide structures, involves very complex mathematical models. However, methods of determining the hydraulic structural stress state by testing small-scale, three-dimensional models with polarization optics and holographic interferometry have been developed. These methods were applied in the investigation of nonstationary stress waves in a high arch dam under seismic impact. The methods were also used to estimate natural vibration modes and frequencies of an arch-gravity dam and its aggregate block and to determine the distribution of hydrodynamic pressure in the vicinity of the pressurized edges.

● **6.3-41** Calciati, F. *et al.*, **In situ tests for the determination of the dynamic characteristics of some Italian dams,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2823–2828.

The Italian National Board for Electric Power (ENEL) is sponsoring research to determine the dynamic behavior of major Italian dams in order to reduce the hazards from events such as earthquakes. Experimental studies on a number of dams have been conducted by ISMES of Bergamo, Italy. Test techniques and results are summarized.

● **6.3-42** Buckens, F., **Analysis of the damped response of a multistorey building, considered as a heavy Timoshenko beam, to a bidimensional shaking motion of the ground,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1382–1387.

The author points out that vertical motions of the ground are usually neglected in earthquake engineering and that vertical accelerations are usually small compared to g. However, it is known that natural frequencies and modes are functions of the distributed weight, the effect of which is of the same order of magnitude as that of rotating inertia and shear forces. The effect of two-dimensional ground motions, possibly with vertical accelerations, is analyzed in a plane, first without internal damping in the structure and then with viscous damping. Horizontal and vertical components and tilt of such a motion at the base of a structure are given.

● **6.3-43** Salvadori, M. G. and Singhal, A., **Strength characteristics of jointed water pipelines,** *IR-3,* Weidlinger Assoc., New York, July 1977, 84.

The report has a threefold purpose: (a) to give a general description of a typical pipeline network, with particular emphasis on its geometric configuration and physical characteristics; (b) to describe the strength properties of typical cast-iron pipeline materials and the materials used in their joints, to give a summary of test data on pipeline joints and to try to correlate the test data with the results of elementary theory; (c) to describe standard cast-iron pipeline design methods and to derive the needed typical entries for a static "failure matrix," which establishes failure and leakage characteristics for cast-iron pipelines, depending on their geometric configuration, their joints, and their support, operating, and loading conditions.

This report is the result of a thorough survey of the literature on underground water pipes, but aims at establishing a static "failure matrix methodology" rather than at presenting the application of such methodology to a complete set of pipeline types. Therefore, most of the data and

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examples are limited to cast-iron pipes of diameters between 4 in. and 36 in. under static conditions. The report also contains recommendations for additional tests on pipeline joints to complement the scant data available at present.

6.4 Deterministic Dynamic Behavior of Linear Structures

6.4-1 Goluber, A. A., **Aseismic design and the possibility of "non-oscillating" beam systems** (Seismicheskii raschet i vozmozhnost postroeniya "neostsillatsionnykh" sterzhnevykh sistem, in Russian), *Raschet i proektirovanie prostranstvennykh konstruktsii grazhdanskikh zdanii i sooruzhenii*, NAUKA, Leningrad, 1975, 47–51.

In problems of earthquake engineering the necessity arises for calculating the so-called dynamic coefficient, which depends in an essential way on the vibration mode shape. It is generally assumed that small vibrations of the majority of beams and systems are oscillations. This concept presupposes a fully determinate relationship between natural mode shapes and natural frequencies. The possibility of constructing "nonoscillating" beam systems is demonstrated on a concrete example. A structure of this type may be useful in achieving a substantial reduction of seismic loads.

6.4-2 Shiraki, K. *et al.*, **Bell-ring vibration response of nuclear containment vessel with attached masses under earthquake motion,** *Nuclear Engineering and Design*, **38**, *3*, Sept. 1976, 475–493. (For an additional source, see Abstract No. 1.2-2.)

There are two types of vibrations, designated as "beam-type" and "bell-ring type" occurring with axisymmetric thin shell nuclear containment vessels. Up to this time, the seismic analysis for such thin axisymmetric shells has mostly been carried out only for the beam-type vibration because the response participation factor for the bell-ring type vibration under seismic motion is zero when the shell structure is perfectly axisymmetric. However, as with nuclear containment vessels, when the thin axisymmetric shell has several attached heavy masses such as the equipment hatch or the manholes, the resulting seismic response of bell-ring type vibration is unexpectedly large and becomes remarkably more important than the beam-type vibration. For the seismic analysis of bell-ring type vibration, an approximate uncoupled analysis using the natural mode shapes of an unweighted perfect axisymmetric shell has been advocated on the assumption that the effect of the attached mass on the natural modes might be very small. However, application of this method to some models showed that the calculated response of bell-ring type vibration was noticeably smaller than the experimental results.

In this paper the authors show the seismic response analysis of the bell-ring type vibration coupled with the beam-type vibration through the attached masses with the new consideration. These results show good agreement between the theoretical calculations and the experiment.

● 6.4-3 Ramakrishnan, R. and Kunukkasseril, V. X., **Response of circular bridge decks to moving vehicles,** *Earthquake Engineering and Structural Dynamics*, **5**, *4*, Oct.–Dec. 1977, 377–394.

This paper discusses the dynamic response of a curved bridge deck to a moving vehicle. The bridge deck is idealized as a set of annular sector plates and circular rings rigidly jointed together. On the basis of classical plate and ring theories, a method has been developed to obtain the response to a moving vehicle idealized as a spring-mass system. After obtaining the normal modes and frequencies and establishing the orthogonality conditions, the problem of the forced motion of the deck is solved by the method of spectral representation. Numerical results have been presented to illustrate the effect of several vehicle and bridge parameters on the response.

● 6.4-4 Kan, C. L. and Chopra, A. K., **Effects of torsional coupling on earthquake forces in buildings,** *Journal of the Structural Division, ASCE*, **103**, *ST4*, Proc. Paper 12876, Apr. 1977, 805–819.

The elastic response of torsionally coupled one-story buildings to earthquake ground motion, characterized by idealized shapes for the response spectrum, is studied. Influence of the basic system parameters on the response is investigated. The relationship between the forces—base shears and torque—in a torsionally coupled system and the base shear in a corresponding torsionally uncoupled system is established, and the effects of torsional coupling on earthquake forces are identified. Useful upper and lower bounds are presented for the base shears and torque due to simultaneous action of two horizontal components of ground motion of equal intensity.

● 6.4-5 Kan, C. L. and Chopra, A. K., **Elastic earthquake analysis of a class of torsionally coupled buildings,** *Journal of the Structural Division, ASCE*, **103**, *ST4*, Proc. Paper 12877, Apr. 1977, 821–838.

A simple procedure is developed for analysis of the elastic response of a particular class of torsionally coupled multistory buildings to earthquake ground motion, characterized by smooth response spectra. In this procedure, the response of an N-story torsionally coupled building—a system with 3N degrees-of-freedom (DOF) is determined by analyzing two systems: (1) an N-story torsionally uncoupled

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counterpart of the actual building—a system with N DOF; and (2) an associated one-story torsionally coupled system—a system with 3 DOF. The simpler analysis procedure leads to "exact" results if the variation of earthquake spectral acceleration with vibration period is idealized as flat or hyperbolic. For arbitrary but smooth shapes of the response spectrum, it is shown through a numerical example that the results of the simpler analysis are sufficiently accurate for design purposes.

● 6.4-6 Lee, T. H. and Wesley, D. A., **Seismic response of a stacked HTGR fuel column interacting with a control rod,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(b), Paper K 7/6, 8. (For a full bibliographic citation, see Abstract No. 1.2-5.)

A theoretical investigation has been conducted to study the dynamic interaction between an HTGR fuel column and an inserted control rod when the system is excited by boundary motion due to earthquake disturbances. The mathematical model developed simulates a control rod structure inserted at varying depths into the channel of a fuel column. The control rod is modeled as a series of rigid masses with flexible contact points representing the canisters which are interconnected by spherical ball joints. The fuel column consists of a series of stacked graphite blocks constrained by dowel pins. The dynamic response of stacked columns has already been extensively studied by the present authors. Therefore, the interaction model was established by coupling an existing stacked column model with a control rod model which was separately developed.

Before combining the two structures into a coupled system, the dynamic characteristics of the rod by itself were verified by a harmonic vibration analysis. The control rod modeling incorporates the dead weight of each canister mass. When the rod is hanging vertically, the gravity effect produces a system in tension which gradually increases with elevation. The use of ball-joint connectors ensures the characteristics of a control rod closely resembling that of a chain-type structure. Therefore, the dynamic behavior of a rod was compared with that of a continuous hanging chain. This comparison was made by first obtaining the forced vibration response of a rod excited sinusoidally through its top support at various driving frequencies and then comparing these response configurations with the mode shapes of a continuous chain of the same length. It was found that the harmonic response shapes of a control rod can be identified with the vibration modes of a vertical chain having the same boundary conditions. The chain mode shapes were computed from analytical expressions in terms of Bessel functions. The column-rod interaction analysis was performed by coupling the chain-type model of the control rod with the mathematic model of a stacked column through the phenomenon of impact. The geometrical length of the control rod canister is designed in such a way that the canisters are always staggered with respect to the fuel blocks, irrespective of rod insertion position. A feature of the present analysis is that the coordinates for the contact nodes where impact forces are generated are not fixed but rather internally assigned by the computer according to the insertion position specified and, consequently, an arbitrary insertion depth can be considered. The numerical results show variations of seismic response of the system with respect to the insertion depth and other parameters. An inserted control rod was found to act as an efficient impact damper which attenuates the column response.

● 6.4-7 Murakami, H. and Luco, J. E., **Seismic response of a periodic array of structures,** *Journal of the Engineering Mechanics Division, ASCE,* **103,** *EM5,* Proc. Paper 13278, Oct. 1977, 965–977.

A simplified two-dimensional model of the dynamic interaction through the soil among adjacent structures in a densely built area is presented. The model consists of an infinite number of identical, parallel infinitely long shear walls placed on equally spaced, rigid semicylindrical foundations. The steady-state response of the shear walls to obliquely incident plane SH waves is evaluated and compared with the response of an isolated structure. The results obtained indicate that the presence of adjacent structures has a significant effect on the seismic response, particularly at low frequencies and in the vicinity of the Rayleigh frequencies for the array. For high frequencies, only the immediately adjacent structures affect the response of the central shear wall.

● 6.4-8 Kapur, A. D., Nakra, B. C. and Chawla, D. R., **Shock response of viscoelastically damped beams,** *Journal of Sound and Vibration,* **55,** *3,* Dec. 8, 1977, 351–362.

Analysis of the dynamic response of two-layered and three-layered viscoelastically damped beams subjected to half-sine shock excitation has been carried out. Effects of rotary and longitudinal inertias have been included in the analysis, in addition to the effects of transverse inertia. The properties of the viscoelastic material have been represented by those of a four-element viscoelastic model. The constants of the model have been determined for a viscoelastic material for which properties are known for maintained harmonic excitation. Experimental verification of the analytical results for strain response due to shock excitation is reported.

● 6.4-9 Masri, S. F. and Udwadia, F., **Transient response of a shear beam to correlated random boundary excitation,** *Journal of Applied Mechanics,* **44,** Series E, *3,* Sept. 1977, 487–491.

The *transient* mean-square displacement, slope, and relative motion of a viscously damped shear beam subjected to correlated random boundary excitation is presented. The effects of various system parameters including the spectral characteristics of the excitation, the delay time between the beam support motion, and the beam damping have been investigated. Marked amplifications in the mean-square response are shown to occur for certain dimensionless time delays.

● **6.4-10** Goel, S. C., **Seismic behavior of multistory K-braced frames under combined horizontal and vertical ground motion,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1172-1177.

The effect of the vertical component of ground motion on the earthquake response of two 6-story, braced steel frames is studied. One frame has K-pattern bracing and the other an X-pattern in 2-story modules. The accelerograms are 1.5 times the intensity of the El Centro 1940 earthquake. The results show that inclusion of the vertical component of ground motion can cause significant increases in the ductility requirements of frame members, among which the columns are most affected, particularly in terms of axial forces.

● **6.4-11** Iyengar, R. N. and Saha, T. K., **Effect of vertical ground motion on the response of cantilever structures,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1166-1171.

The lateral response of cantilevers under the combined action of horizontal and vertical ground accelerations is studied. The vertical acceleration mainly alters the self-weight at any section. This necessitates the consideration of the self-weight of the structure also. The governing differential equation is solved for several past ground excitations. The envelopes of the maximum bending moment and shear force have been obtained along the height of the cantilever with and without the vertical ground motion.

● **6.4-12** Yoshimura, K. and Inoue, M., **Dynamic analysis of reinforced concrete frames with framed shear walls,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1178-1184.

Linear static and dynamic analyses of reinforced concrete frames with framed shear walls are carried out for various arrangements of framed shear walls. Analytical results indicate that dynamic response of such frames during an earthquake can be evaluated approximately by the static analysis.

● **6.4-13** Masri, S. F. and Weingarten, V. I., **Transient response of cooling towers to propagating boundary excitation,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1235-1240.

Many axisymmetric shell structures such as cooling towers and nuclear power plant containment structures can be subjected to traveling seismic waves. The characteristic times defining the passage of the wave front across the structures may have significant effects on the response of the cooling towers. At present, these shell structures are analyzed by subjecting the structure to seismic base motion. Using linear theory, this type of motion induces a beam-type response of the cooling tower and does not permit other types of response in the structure. A traveling wave solution physically more closely represents the actual problem and allows the structure to respond in all of its natural modes. The cooling tower problem is considered in the present investigation. The cooling tower is analyzed as a shell of revolution by the finite element method. The ground motion is assumed to be a traveling wave and is decomposed into Fourier components for each time interval. The normal mode method in conjunction with the Wilson-theta integration technique is used to determine the response of the cooling tower.

● **6.4-14** Arya, A. S. *et al.*, **Seismic analysis of reactor internals of Narora Atomic Power Plant, India,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2659-2664.

This study describes the structural system that has been developed for reactor internals. The internal structure of a heavy water reactor plant located in a zone with a low degree of seismicity is usually kept independent of the cylindrical containment structure and is quite flexible under lateral vibrations. If such a system is adopted in a moderate or severe seismic zone, the absolute as well as the interfloor seismic displacements are found to be too large to be withstood by the equipment and piping systems; therefore, the structural system in the latter case has to be specially designed. This paper presents the results of a preliminary earthquake analysis of the original and the redesigned structural systems. Suitable assumptions have been made to represent the internal structure as a multimass system and its dynamic behavior has been obtained for a prescribed raft motion.

● **6.4-15** Hegemier, G. A. *et al.*, **Earthquake response and damage prediction of reinforced concrete masonry multistory buildings, Part I: program definition,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3013-3018.

● *See* ***Preface****, page v, for availability of publications marked with dot.*

This paper outlines portions of an extensive research program on reinforced concrete masonry. Sample results are discussed in Part II (Abstract No. 6.11-43).

● 6.4-16 Bairactaris, D. and Roussopoulos, A., **Theoretical and observational investigation of structural response to instantaneous peak accelerations during strong earthquakes,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 762-765.

A sinusoidal earthquake motion is considered with a sudden acceleration increment at its n-th period and the response of a one degree-of-freedom structure is theoretically investigated by successive application of the final conditions of one stage to the initial conditions of the following stage. The results are compared with recorded strong-motion accelerograms of this type and the corresponding acceleration and displacement response spectra. Some conclusions are derived concerning the validity of the theoretical results as well as the structural behavior for transient forced vibration of this type.

● 6.4-17 de Aleman, M. A., Meinheit, D. F. and Jirsa, J. O., **Influence of lateral beams on the behavior of beam-column joints,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3089-3094.

One of the most difficult problems associated with the design of framed structures for seismic loads is the design of the beam-column joint for shear. Current design procedures are based on tests of planar frames and little consideration is given to the influence of lateral beams (beams which frame into the joint normal to the direction of the primary shear force or deformation). The object of this paper is to examine the behavior of beam-column joints with and without lateral beams.

● 6.4-18 Iguchi, M., **A basic study on the behaviour of long dimensional size buildings during earthquakes,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1490-1495.

A theoretical, approximate, three-dimensional analysis of the vertical behavior of long structures subjected to obliquely incident earthquake motion is presented. A long elastic base plate with a rectangular plane shape is used as a model. Integral equations are formulated using Galerkin's method.

● 6.4-19 Baron, F., Arikan, M. and Hamati, R. E., **The effects of seismic disturbances on the Golden Gate Bridge,** *EERC 76-31,* Earthquake Engineering Research Center, Univ. of California, Berkeley, Nov. 1976, 172. (NTIS Accession No. PB 272 279)

This report deals with the effects of seismic disturbances on the Golden Gate Bridge. It is a supplement to the studies made during the design phase of the bridge. The results that are obtained in this study are based on the tangent-stiffness properties of a bridge at its dead load state. Both static and dynamic properties of the bridge are considered. A two-dimensional analytical model is used to study the longitudinal and vertical vibrations of the bridge; a three-dimensional model is used for the transverse and torsional vibrations.

Studies are made of the response of the Golden Gate Bridge to uniform ground motion and to ground motion propagating along the longitudinal axis of the bridge. To determine the severities of the maximum forces and moments caused by various excitations, the maximum values of stresses are calculated for selected elements of the bridge. The studies reported show that the effects of a moderate earthquake on the Golden Gate Bridge are essentially negligible; and that a strong earthquake of Richter magnitude 8.+ can produce large values of stresses in certain elements of the bridge. Studies of other suspension bridges located in areas of high seismicity need to take into account the influences of earthquake excitations in the transverse directions as well as in the longitudinal directions of the structures. Multiple-support excitations tend to increase the participation of the higher modes in the total response, leading to larger values of maximum response than those obtained for uniform-support excitations.

6.4-20 Hawkins, N. M., **State-of-the-art report on seismic resistance of prestressed and precast concrete structures,** *Journal of the Prestressed Concrete Institute,* 22, 6, Nov.-Dec. 1977, 80-110.

The results of analytical and experimental studies concerning the seismic resistance of prestressed and precast concrete structures and their subassemblages are reviewed. The significance of the studies is assessed, and research needs for structural design are identified. Separate sections are presented for prestressed and precast concrete. Prestressed concrete is covered in this presentation. The report updates a state-of-the-art paper developed for a Workshop on Earthquake Resistant Reinforced Concrete Building Construction (ERCBC) held at the Univ. of California at Berkeley in July 1977. Eight other papers dealing with prestressed and precast concrete buildings were presented at that workshop. Information from these eight papers is incorporated in the report along with research recommendations developed at the workshop by a working group concerned with prestressed and precast concrete.

● 6.4-21 Matthiesen, R. B. *et al.*, **Earthquake-induced dynamic response of bridges and bridge measurements,** *TRB/TRR-579,* Transportation Research Board, National Research Council, Washington, D.C., 1976, 106. (NTIS Accession No. PB 256 523)

● *See* ***Preface,*** *page v, for availability of publications marked with dot.*

The nine papers included in this publication were prepared for the 54th Annual Meeting of the Transportation Research Board. The papers include (1) a description of the strong-motion instrumentation program for state highway bridges; (2) a description of retrofit measures to improve the seismic performance of existing highway bridges and a discussion of a numerical seismic method of analyzing their effectiveness; (3) a discussion of research on a model relating to the seismic resistance of large multispan curved overcrossings; (4) a description of new seismic design criteria for bridges that consider both fault proximity and local soil conditions; (5) a description of applications of bridge measurement data to fatigue design, including impact, girder load distribution, fatigue life stress cycles, predicted and measured stresses, and the potential for weighing trucks in motion on bridges; (6) a description of field testing of the Aguasabon River bridge in Ontario; (7) a discussion of an analytical method for determining the response of horizontally curved bridges to loads; (8) a presentation of a seven-step procedure for using stress history data to predict stress in bridge girders; and (9) a presentation of a rational approach for determining remaining fatigue life of a bridge.

6.5 Nondeterministic Dynamic Behavior of Linear Structures

● 6.5-1 Duff, C. G. and Asmis, G. J. K., **Seismic design for steam generators—a multiple system approach,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(b), Paper K 6/16, 12. (For a full bibliographic citation, see Abstract No. 1.2-5.)

Certain classes of nuclear power stations such as CANDU-PHW plants have multiple reactor coolant systems. The typical CANDU 600MW(e) plant has four such systems functioning within each reactor building, with the major components in each coolant loop being the steam generators. These steam generators are clustered about the reactor and are supported independently, but in similar fashion by the boiler room floor. Although the component steam generators were designed as identical units, it was found that differences in operating conditions and variations in lateral supports caused a detuning effect between the four generators, with a consequent decrease in the effective mass ratio and an increase in response. It is the purpose of this paper to extend the concept of mass ratio to multiple system installations.

Guided by the results of the parametric study, it was concluded that the coupled building/steam generator analysis, carried out on only one system, with the weight of the remaining three systems lumped into the building structure, would provide conservative but reasonable results; whereas, an analysis modeling all four systems would not only lead to a more extensive and complicated analysis but would also underestimate the response. A description of the coupled analysis and representative results of the steam generator accelerations and displacement are presented in the paper.

● 6.5-2 Masri, S. F. and Aryafar, A., **Response of beam to stochastic boundary excitation,** *Journal of the Engineering Mechanics Division, ASCE,* **103,** *EM5,* Proc. Paper 13274, Oct. 1977, 807-822.

A closed-form solution is presented for the covariance kernel of the transient response of a damped Bernoulli-Euler beam with arbitrary boundary conditions to correlated stochastic excitation applied at the boundaries. The analytical results are applied to the case where the autocorrelation function of the excitation resembles that of a wide class of input functions including earthquake excitations. The mean-square transient response at arbitrary locations along the beam is evaluated, and the effects of various system parameters are determined. It is shown that, depending on the beam characteristics, the spectral width of the input, and the delay time involved in the wave propagation between supports, the mean-square response of the beam can be dramatically influenced.

6.6 Deterministic Dynamic Behavior of Nonlinear Structures

● 6.6-1 Berzhinskii, Yu. A. and Shpyneva, N. V., **Analysis of the response of nonlinear systems subjected to seismic excitation using decomposition of deformation diagrams** (Analiz reaktsii nelineinykh sistem pri seismicheskom vozdeistvii s ispolsovaniem razlozheniya diagramm deformirovaniya, in Russian), *Seismostoikoe stroitelstvo, 3,* 1977, 48-52.

The behavior of nonlinear single degree-of-freedom systems subjected to real seismic loads was investigated. Prandtl diagrams were employed to approximate deformation diagrams by means of linear segments. The technique presented is applicable to all deformation diagrams and is suitable for computer algorithms. It must be pointed out, however, that the method of decomposition used is applicable only in those situations where the principle of superposition holds true.

● 6.6-2 Frank, R. A. *et al.,* **Variability of inelastic structural response due to real and artificial ground motions—Report No. 4 in series: Evaluation of seismic safety of buildings,** *R76-6,* Dept. of Civil Engineering, Massachusetts Inst. of Technology, Cambridge, Jan. 1976, 95.

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The variation of nonlinear dynamic structural response due to real and artificial ground motions is studied. Three four degree-of-freedom systems were designed using Newmark's inelastic response spectrum, and statistics of ductility, accelerations, and displacements were calculated. The effects of stiffness variation with height and change of the motion intensity are also examined.

● 6.6-3 Korenev, B. G. and Polyakov, V. S., **Optional parameters of dynamic vibration damper for seismic type excitations** (Optimalnye parametry dinamicheskogo gasitelya kolebanii pri vozdeistvii tipa seismicheskogo, in Russian), *Seismostoikoe stroitelstvo, 3,* 1977, 37–42.

The study of the response of structures equipped with dynamic dampers under seismic excitations of various kinds involves many unsolved questions. In this article, the authors continue their investigations of the behavior of single degree-of-freedom systems with dynamic dampers subjected to damped sinusoidal excitations. Analog computer methods were employed to find optimal parameters for the dynamic damper.

● 6.6-4 Misra, N. and Ramakrishnan, V., **Behavior of isolated shear walls subjected to lateral loads,** *SDSM&T-CNSF 7502,* Civil Engineering Dept., South Dakota School of Mines and Technology, Rapid City, May 1975, 29. (NTIS Accession No. PB 243 945)

The project deals with lateral load distribution in nonlinear concrete shear wall systems. The work is being performed along two lines: theoretical analyses using the finite element method and other more approximate techniques and experimental studies using fairly large microconcrete models. The objective of the investigation is to recommend practical guidelines for the economical design of shear walls to resist lateral loads.

Behavior of an isolated shear wall subjected to lateral loads is analyzed in this report. Two concentrated lateral loads were applied at alternate floor levels in a four-story shear wall. Deflections, stresses and strains were measured and compared with the theoretical results. The differences between the theoretical and experimental values are shown and discussed.

● 6.6-5 Pakman, B., **Earthquake analysis of Halic Bridge** (Halic koprusu deprem analizi, in Turkish), *Deprem Arastirma Enstitusu bulteni,* 3, *9,* Apr. 1975, 1–11.

In this paper, using information concerning the structure of the Istanbul Golden Horn Bridge, the maximum response method used in the earthquake analysis by Consortium Halic Bridge and the application of this method to the Istanbul bridge are explained, along with the assumptions made. Consideration of the results obtained and the probable behavior of the structure during an earthquake are discussed.

● 6.6-6 Prathuangsit, D., **Inelastic hysteresis behavior of axially loaded steel members with rotational end restraints,** *UMEE 76R5,* Dept. of Civil Engineering, Univ. of Michigan, Ann Arbor, Apr. 1976, 180.

Past studies of the cyclic inelastic behavior of predominantly axial-force-carrying structural elements, such as bracing members, assumed the members to be ideally pin-connected or fully fixed. A real member always has some rotational restraints at its ends due to the flexural rigidity of the connections. The effects of the rotational end restraints need to be investigated.

This report provides an analytical study of the inelastic hysteresis behavior of axially loaded steel members with symmetrical rotational end springs (connections). The springs are intended to be a simulation of the connections of a bracing member or a simulation of the end restraints of an axially loaded member due to other connected members in an actual structure.

● 6.6-7 Tezcan, S., **Analysis of buildings with regard to torsion** (Binalarin burulmaya gore hesabi, in Turkish), *Deprem Arastirma Enstitusu bulteni,* 3, *12,* Jan. 1976, 16–26.

In this study, the analysis of a floor with regard to torsion caused by earthquake loadings is presented in tabular form and a numerical example is given. In the analysis, it is assumed that the inertia moments, the geometry of the floor, and the column loads are known.

6.6-8 Belytschko, T. B. and Kennedy, J. M., **A fluid-structure finite element method for the analysis of reactor safety problems,** *Nuclear Engineering and Design,* 38, *1,* July 1976, 71–81.

A method is presented for the safety analysis of reactor containment structures by means of finite elements. The finite element equations of both fluid and structural elements for arbitrarily large, nonlinear response are developed and the way in which they are combined is indicated. Both explicit and implicit integration of the equations in time is considered. Three examples of the application of these methods to the analyses of reactor safety problems are described.

6.6-9 Liebe, R., **Experimental verification of structural models to analyze the nonlinear dynamics of LMFBR fuel elements,** *Nuclear Engineering and Design,* 38, *1,* July 1976, 29–41.

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Local fault situations in LMFBR cores may produce severe pressure pulses within one fuel element. The fact cannot be ignored that these pressures can have peaks and impulses that may expand and rupture the wrapper around the element. This will impulsively load the surrounding subassemblies and possibly the control rods due to extreme coolant pressure gradients and/or subassembly collision forces. Fast reactor safety requires this mechanical propagation process through the core to be analyzed, and therefore appropriate models and solution methods are needed to simulate the nonlinear structural dynamics of one typical hexagonal fuel element. The aim of this paper is to outline one- and two-dimensional structural models and discuss their capabilities and suitability for multirow core calculations. For this purpose static and impulsive single subassembly loading experiments are described and typical results are reported and compared with numerical predictions.

A short description is given of relevant physical effects, for instance the interaction of two distinct deformation modes of the externally loaded fuel element. The role of the fuel-pin bundle inside the wrapper is briefly mentioned, and it is shown that the strongly nonlinear transient response is due to large elastoplastic deformation of the wrapper combined with a frictional distortion of the pin bundle. Special discrete models were developed which are characterized by lumped point masses connected by elastoplastic beams or nonlinear springs.

A brief discussion of an experimental program is then given which is designed to verify theoretical models and underlying hypotheses. A drop-weight facility is used for transverse impulsive loading of natural size SNR-type fuel element models. Honeycomb crushing material is placed between the falling mass and the specimen to limit the peak force of the loading pulse. A simplified computer model has been set up describing the elastoplastic impact between the falling mass, the crushing material and the subassembly model, which is resting on a heavy, spring-suspended table.

Finally, numerical prediction and experimental response data are presented for large, elastoplastic deformations of a specific reference case under two loading conditions: (1) a constant and low rate of deformation producing a quasi-static yielding response of an empty hexcan on a rigid plate; and (2) a controlled time-dependent impact force pulse (drop mass) producing typically dynamic responses.

6.6-10 Takemori, T., Sotomura, K. and Yamada, M., **Nonlinear dynamic response of reactor containment,** *Nuclear Engineering and Design,* 38, *3,* Sept. 1976, 463-474. (For an additional source, see Abstract No. 1.2-2.)

In the seismic analysis of nuclear power plants, the time history method is used as one of the techniques of dynamic analysis. Many problems such as idealization of a dynamic model, assessment of damping coefficients and soil-structure interaction are involved in this method. Since the time history method is usually carried out in the elastic range, the ultimate behavior of a structure to some extent still remains unknown after these problems are solved; thus, a computer program was developed to investigate the elasto-plastic behavior of structures. The purpose of this paper is to present the outline of this program and to discuss the problems of the nonlinear response of structures. There are two theoretical methods in the time history approach; that is, the mode superposition method and the direct integration method. Since the mode superposition method is only valid in an elastic analysis, the direct integration method was adopted here.

As the sample model, an actual reactor containment (reactor building) of a PWR plant was adopted. This building consists of three components: a concrete internal structure, a steel containment vessel and a concrete outer shield wall. These components rest on a rigid foundation mat. Therefore, they were modeled with a lumped mass model and coupled on the foundation. The following assumptions and procedures were employed to establish the properties of the dynamic model and to carry out the response analysis: (1) Rocking and swaying springs of soil can be obtained from an elastic halfspace solution. The nonlinear rocking spring can be considered when a foundation mat detaches from the soil. For its hysteretic characteristic, both bilinear and spring types are studied and employed in the response analysis. (2) Springs connecting each mass are dealt with as shear beams so that both bending and shear deflections can be included. The hysteretic characteristics of each spring of the containment vessel and the outer shield wall are determined to be either bilinear or trilinear according to its intensity and the springs of the internal structure are linear. (3) Two kinds of hysteretic loops are studied and used in the response analysis. (4) Generally, a particular damping coefficient is given for each mode in modal superposition. However, a damping matrix must be made directly in a nonlinear response. Therefore, the damping matrix of the model was made by combining the damping matrices of each component obtained by Cauchy's method and a damping value of the rocking and swaying by the halfspace solution. (5) Two earthquake waves of El Centro 40 N-S and Golden Gate E-W are used with modifying 300, 450 and 600 gal. On the basis of above conditions, the nonlinear response of the structure was obtained and the difference between elastic and elasto-plastic analysis is presented.

● **6.6-11** Shimogo, T. and Fujimoto, S., **Response analysis of 500KV circuit breaker with nonlinear damping devices under seismic excitation,** *Proceedings of U.S.-Japan Seminar on Earthquake Engineering Research with Emphasis on*

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Lifeline Systems, Japan Society for the Promotion of Earthquake Engineering, Tokyo, Nov. 1976, 257–269.

In this paper, a mathematical model of a 500 kV air circuit breaker structure is proposed to analyze the responses, and the validity of this model is verified by an experiment using a physical model of the circuit breaker, and further through a test of a full-size circuit breaker. Three cycles of a sinusoidal wave at resonant frequency are used as an input for this verification.

● **6.6-12** Bertero, V. V. and Popov, E. P., **Seismic behavior of ductile moment-resisting reinforced concrete frames,** *Reinforced Concrete Structures in Seismic Zones, SP-53,* American Concrete Inst., Detroit, 1977, 247–291.

Results obtained from investigating the hysteretic behavior of flexural critical regions subjected to high and low shears, and of interior beam-column subassemblages of reinforced concrete ductile moment-resisting frames, are summarized and evaluated.

When the average shear stress at the critical region of a flexural member exceeds a value of $3.5\sqrt{f'_c}$ (psi), with a load reversal exceeding the flexural yielding value, a significant degradation in the energy absorption and energy dissipation capacities occurs. By using an inclined web reinforcement in combination with vertical ties, stable hysteretic behavior was achieved of inelastic rotations up to 0.035 rads in one case, and 0.067 rads during a full load reversal. The hysteretic behavior of interior beam-column subassemblages after a reversal of loads inducing a displacement ductility ratio of 2.5 shows (1) a significant P-δ effect; and (2) a drastic drop in strength, and especially in stiffness. This drop is caused by the slippage of the main bars of the beams through the interior joint. To avoid these effects, it is recommended that the reinforcement of the beams be designed and detailed to induce plastic hinge formation away from the column faces.

The aseismic design implications of the results obtained are assessed and recommendations for improving present aseismic code provisions are offered.

● **6.6-13** Gulkan, P. and Sozen, M. A., **Inelastic responses of reinforced concrete structures to earthquake motions,** *Reinforced Concrete Structures in Seismic Zones, SP-53,* American Concrete Inst., Detroit, 1977, 109–115. (Reprinted from *Journal of the American Concrete Institute,* **71,** *12,* Dec. 1974, 604–610.)

Two basic characteristics of reinforced concrete structures play an important role in determining response to strong ground motions. They are the changes in stiffness and energy dissipation capacity. Both can be related to the maximum displacement. Results of dynamic tests of reinforced concrete frames are used to illustrate the effects on dynamic response of changes in stiffness and energy dissipation capacity. It is shown that maximum inelastic response can be interpreted in terms of linearly elastic analysis by reference to a fictitious linear structure whose stiffness and damping characteristics are determined as a function of the assumed or known maximum displacement. This leads to a simplified method for estimating the design base shear taking account of inelastic response.

The object of this paper is to describe basic phenomena of energy dissipation in reinforced concrete structures subjected to strong ground motion and to present a simplified method for estimating the design base shear corresponding to inelastic response. Tests of a series of reinforced concrete frames are reported in detail. Individual frames were subjected to steady-state dynamic base motion, simulated earthquake motion, or static lateral loading. Data from that study are used to illustrate fundamental response characteristics.

● **6.6-14** Selna, L. G. and Lawder, J. H., **Biaxial inelastic frame seismic behavior,** *Reinforced Concrete Structures in Seismic Zones, SP-53,* American Concrete Inst., Detroit, 1977, 439–461.

The importance of overturning forces and biaxial response is studied for 3D reinforced concrete frame structures subjected to biaxial inputs. A biaxial yield rule is used for the moment-curvature relation. Incremental static and dynamic analyses are performed. From the study it is concluded that: (1) in biaxial response the total lateral load and sequence of load application collectively influence the response; (2) overturning forces have an effect on the flexural shear capacity of individual columns; (3) the computed lateral frame capacity tends to decrease when overturning is recognized; and (4) in earthquake response the biaxial nature of ground motion input and frame resistance is evident.

● **6.6-15** Paramzin, A. M. and Itskov, I. E., **Behavior of thin walls and ceilings of reinforced concrete blocks under seismic type loadings** (Povedenie tonkikh sten i potolkov zhelezobetonnykh obemnykh blokov pri nagruzkakh tipa seismicheskikh, in Russian), *Seismostoikoe stroitelstvo, 2,* 1977, 19–23.

During dynamic testing of full-scale models of room-sized reinforced concrete blocks, high normal accelerations were recorded in the ceiling and the walls. Natural frequencies and mode shapes were investigated and the effect of external loadings on the dissipative properties of the structure analyzed. The testing methods and apparatus are described. Special design features required for earthquake-resistant precast panel buildings are discussed. The need for continued research into these problems is pointed out.

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● 6.6-16 Stepanyan, V. A., **Exact calculation of the limiting state parameters of a reinforced concrete frame from results of testing under horizontal loads** (Utochnenie parametrov predelmogo sostoyaniya zhelezobetonnogo karkasa po resultatam ispytaniya ram na gorizontalnye nagruzki, in Russian), *Seismostoikoe stroitelstvo*, *6*, 1977, 16-19.

Full-scale models of reinforced concrete frames were tested under horizontal dynamic and alternating static loads. The effects of crack formation on the elastoplastic properties and ultimate strength of reinforced concrete buildings are discussed.

● 6.6-17 Wolf, J. P., **Seismic response due to travelling shear wave including soil-structure interaction with basemat uplift,** *Earthquake Engineering and Structural Dynamics*, **5**, *4*, Oct.-Dec. 1977, 337-363.

The seismic response due to a traveling shear wave is investigated. The resulting input consists of a translational- and a torsional-acceleration time history, which depends on the ratio of the wavelength to the dimension of the footing. A nuclear reactor building is used for illustration. The combined result of the translational and torsional elastic response (the latter arises even in an axisymmetric structure) will not, in general, be larger than that encountered in the case of a spatially uniform earthquake. If the footing slips or becomes partially separated from the soil, a nonlinear dynamic analysis has to be performed to determine the response. Substantial motions in all three directions will take place. The peak structural responses and the floor response spectra are found to be highly nonlinear for high acceleration input values.

● 6.6-18 Kan, C. L. and Chopra, A. K., **Elastic earthquake analysis of torsionally coupled multistorey buildings,** *Earthquake Engineering and Structural Dynamics*, **5**, *4*, Oct.-Dec. 1977, 395-412.

With the aid of perturbation analysis of vibration frequencies and mode shapes, it is shown that any lower vibration mode of a torsionally coupled building may be approximated as a linear combination of three vibration modes of the corresponding torsionally uncoupled system (a system with coincident centers of mass and resistance but all other properties are identical to the actual system): one translational mode along each of the two principal axes of resistance and one mode in torsional vibration. This result provides the motivation for a simpler (relative to the standard) procedure for analyzing the response of torsionally coupled multistory buildings to earthquake ground motion. To illustrate the application and accuracy of this procedure, two numerical examples are presented.

● 6.6-19 Humar, J. L. and Wright, E. W., **Earthquake response of steel-framed multistory buildings with setbacks,** *Earthquake Engineering and Structural Dynamics*, **5**, *1*, Jan.-Mar. 1977, 15-39.

A study is made of the dynamic behavior of multistory steel rigid-frame buildings with setback towers. The effects of setbacks upon the building frequencies and mode shapes are examined. Then, the effects of setbacks on seismic response are investigated by analyzing the response of a series of setback building frame models to the El Centro ground motion. Finally, the computed responses to the El Centro earthquake are compared with some code provisions dealing with the seismic design of setback buildings.

The conclusions derived from the study include the following: (1) the higher modes of vibration of a setback building can make a very substantial contribution to its total seismic response; this contribution increases with the slenderness of the tower; (2) some of the important response parameters for the tower portion of a setback building are substantially larger than for a related uniform building; and (3) for very slender towers, the transition region between the tower and the base may be subjected to very large story shears.

● 6.6-20 Fintel, M., **Investigation of earthquake resistance of structural (shear) wall buildings carried out at Portland Cement Association,** *Wind and Seismic Effects*, VI-199-VI-219. (For a full bibliographic citation, see Abstract No. 1.2-4.)

In July of 1974, the Portland Cement Assn. began a comprehensive analytical and experimental study to investigate the response to earthquake excitation of reinforced concrete structures containing structural (shear) walls. The aim of the study is to develop design procedures and reinforcing details for earthquake-resistant multistory reinforced concrete structures containing structural walls.

● 6.6-21 Penzien, J. *et al.*, **Seismic response of reinforced concrete highway bridges,** *Wind and Seismic Effects*, VI-1-VI-9. (For a full bibliographic citation, see Abstract No. 1.2-4.)

Presented is a brief progress report of an investigation entitled "An Investigation of the Effectiveness of Existing Bridge Design Methodology in Providing Adequate Structural Resistance to Seismic Disturbances," which was initiated in 1971 within the Earthquake Engineering Research Center, Univ. of California, Berkeley, under the sponsorship of the U.S. Dept. of Transportation, Federal Highway Admin.

● 6.6-22 Birnbaum, N. K. *et al.*, **Calculation of transient, non-linear phenomena in nuclear reactors using the**

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PISCES computer codes, *International Seminar on Extreme Load Conditions and Limit Analysis Procedures for Structural Reactor Safeguards and Containment Structures,* T 2/1, 11. (For a full bibliographic citation, see Abstract No. 1.2-2.)

PISCES is a system of explicit finite difference computer codes that solve the fundamental partial differential equations of continuum mechanics using the difference formulations of Wilkins and Noh. The PISCES system is composed of seven codes. PISCES 1DL, PISCES 2DL and PISCES 3DL are written in a Lagrangian reference frame. PISCES 1DL and PISCES 2DL are used for geometries with one or two independent space variables. PISCES 3DL is for asymmetric geometries. PISCES 1DE, 2DE and 3DE are analogous codes written in a Eulerian reference frame. PISCES 2DELK is a two-dimensional code utilizing dynamically coupled Eulerian and Lagrangian grids. All the PISCES codes are capable of modeling complex geometries, time- and space-dependent initial conditions. There are no restrictions with respect to equations of state or constitutive model.

The numerical techniques used in PISCES are discussed. The use of PISCES to compute transient nonlinear response in a nuclear reactor is illustrated by computation of the response of a pressurized water reactor (PWR) to longitudinal and circumferential breaks in the steel pressure vessel wall.

● **6.6-23** Jones, N. and Okawa, D. M., **Dynamic plastic buckling of cylindrical and spherical shells,** *International Seminar on Extreme Load Conditions and Limit Analysis Procedures for Structural Reactor Safeguards and Containment Structures,* T 3/4, 30. (For a full bibliographic citation, see Abstract No. 1.2-2.)

A theoretical analysis is developed to predict the dynamic plastic buckling of a long, impulsively loaded cylindrical shell. This theoretical work is used to examine various features of plastic buckling and to assess the importance of several approximations which previous authors have introduced in dynamic plastic buckling studies. In particular, the influence of a time-dependent circumferential membrane force, the sharpness of the peaks in the displacement and velocity amplification functions, the restrictions which are implicit when employing the Prandtl-Reuss equations in this class of problems, and the limitations due to elastic unloading are examined in some detail. An experimental investigation was also conducted of the dynamic plastic buckling of circular rings subjected to uniformly distributed external impulsive velocities. In order to provide perspective, a brief account is given of some previous studies on the dynamic plastic buckling of complete spherical shells which were subjected to an almost axisymmetric external impulsive velocity field. It appears that the threshold velocities for elastic, elastic-plastic and rigid-plastic spherical shells are larger than the corresponding values associated with cylindrical shells having the same R/H ratios and material parameters.

● **6.6-24** Bazant, Z. P. and Bhat, P. D., **Prediction of hysteresis of reinforced concrete members,** *Journal of the Structural Division, ASCE,* **103,** *ST1,* Proc. Paper 12662, Jan. 1977, 153-167.

The endochronic theory for inelasticity and failure, previously established, is used to predict the response of reinforced concrete beams in cyclic bending at large strains. Cross sections are assumed to remain plain and are subdivided in slices. Existing bending theories must be enhanced by inclusion of transverse normal strain as a third variable, in addition to curvature and transverse shear angle. The forces in stirrups bring concrete under confining hydrostatic pressure, and according to endochronic theory, this greatly increases ductility and strength and suppresses strain-softening. The theory is applicable to any history of bending moment, shear force, and axial force, and allows the necessary cross-sectional area of stirrups to be calculated. It is most remarkable that a number of test data have been correctly predicted without having to adjust any of the material parameters determined previously from tests of plain concrete. The endochronic theory represents not merely a descriptive model but a prediction method.

● **6.6-25** Anderson, J. C. and Townsend, W. H., **Models for RC frames with degrading stiffness,** *Journal of the Structural Division, ASCE,* **103,** *ST12,* Proc. Paper 13441, Dec. 1977, 2361-2376.

Analytical models for representing the hysteretic behavior of reinforced concrete members are reviewed. Based on results of exterior connection tests, two degrading trilinear models are suggested, one of which considers the effect of the connection. The inelastic dynamic response of a 10-story single-bay frame to earthquake excitation is evaluated using the two proposed models. For purposes of comparison, the response using a bilinear model and degrading bilinear model is also considered. Results show that inclusion of the degrading concrete stiffness and reduced connection stiffness has a significant effect on the response.

● **6.6-26** Curreri, J. R. and Bezler, P., **Effect of damping on the response of a non-linear system with multiple sine wave excitation,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(b), Paper K 8/9, 7. (For a full bibliographic citation, see Abstract No. 1.2-5.)

Sine wave sweep tests are commonly used to evaluate the dynamic characteristics of, or to simulate the shock testing to, a response spectrum of linear and nonlinear systems. This paper shows that large differences can occur in the dynamic response of a cubic nonlinear system if even

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a small amount of a second sinusoidal term appears in the excitation; that is, small responses are obtained if the system is excited with either term applied separately. However, when the combination is applied, large responses can develop. The cubic nonlinearity is used as a first analytical form of a system with clearance gaps between masses, such as an HTGR core. The paper discusses the effect of damping on the critical lock-on value of the excitation for a subharmonic oscillation to develop.

● **6.6-27** Aziz, T. S., **Inelastic dynamic analysis of building frames,** *R76-37,* Dept. of Civil Engineering, Massachusetts Inst. of Technology, Cambridge, Aug. 1976, 226.

The nonlinear dynamic behavior of building frames is considered. The study covers basic points associated with inelastic dynamic analysis procedures. A formulation for the inelastic analysis of a building frame is presented. The formulation is general enough to take into consideration different nonlinearities which might occur in a building frame on a selective basis. Two types of nonlinearities are studied: those due to material behavior and those due to geometry changes. The importance of each nonlinear effect is studied separately and those effects which might be more important than others are pointed out. Among the different effects studied are the P-Δ and stability effects, the presence of gravity loads, axial deformations in the columns, joint size and nonlinear joint behavior, damping, and nonlinear soil-structure interaction.

Comparisons are made between different complex, intermediate, and simple models for inelastic dynamic analysis. From the studies performed, it would appear that for typical building frames a generalized bending model including all the nonlinear effects will yield results which are realistic and physically reasonable. Complex models can be theoretically more exact, but they are also more sensitive to numerical errors. For the cases studied, use of these complex and expensive models does not seem justified.

● **6.6-28** Mark, K. M. S., **Nonlinear dynamic response of reinforced concrete frames,** *R76-38,* Dept. of Civil Engineering, Massachusetts Inst. of Technology, Cambridge, Aug. 1976, 263.

The purpose of this study is to investigate the applicability of a fiber model to the determination of the nonlinear dynamic response of frames. In this model, the stress and strain are monitored through time at each fiber of several cross sections along each member. The tangent moduli for the steel and concrete resulting from the assumed nonlinear stress-strain relationships are then used to assemble a tangent stiffness matrix for the structure at each step. The dynamic analysis is carried using a central difference formula to advance the solution in time.

Several models for the concrete and the steel are first reviewed and compared. The effect of these models on the moment curvature relationship for a cross section is investigated and results are compared to experimental data. The process is repeated for simple members under static cyclic loading. Finally, the fiber model is used to study the dynamic response of a single-bay one-story frame under sinusoidal and earthquake excitation. Results are again compared to experimental data and to those of simpler models. The fiber model reproduces well the qualitative behavior of a reinforced concrete section or member and explains the effect of constant or variable axial loads. A point-by-point agreement with experimental data cannot be obtained, however. Results for the frame are somewhat better than those provided by other, simpler models, but the model is mainly of academic interest because of the cost of computations. Several refinements are suggested.

● **6.6-29** Sehayek, S., **Effect of ductility on response spectra for elasto-plastic systems,** *R76-42,* Dept. of Materials Science and Engineering, Massachusetts Inst. of Technology, Cambridge, Sept. 1976, 92.

The purpose of this work is to evaluate two procedures used to estimate response spectra for inelastic systems subjected to earthquakes. The first, which is a set of rules suggested by Newmark, derives the inelastic response spectrum for the elastic case. The second is a procedure which replaces the nonlinear spring of the system by an equivalent linear one with stiffness and damping obtained as functions of a characteristic strain. It is concluded that Newmark's procedure, although approximate, performs quite satisfactorily. The second procedure, although not as satisfactory as Newmark's, reproduces the general trends of spectra quite well also.

● **6.6-30** Popov, E. P., Bertero, V. V. and Ma, S. M., **Model of cyclic inelastic flexural behavior of reinforced concrete members,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(a), Paper K 3/14, 12. (For a full bibliographic citation, see Abstract No. 1.2-5.)

Reinforced concrete (RC) structures are commonly used in nuclear power plants to provide housing, shielding, and support of essential equipment. Increased concern about the safety of such plants in the event of a strong-motion earthquake has directed greater attention to studying the hysteretic behavior of these structures under cyclic loading. This paper presents a practical mathematical model for predicting the hysteretic behavior of critical regions (plastic hinges) in RC flexural members designed by using current seismic code provisions (Uniform Building Code or American Concrete Inst.).

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Prediction of such behavior is based on formulating a hysteretic moment-average curvature (M–ϕ) model for critical regions, which in turn is based on stress-strain hysteretic models of the reinforcing steel and the concrete of such regions. The steel stress-strain model uses a Ramberg-Osgood equation with a set of rules describing the strain-hardening behavior under cycles of reversal of stress and strain. The basic parameters are expressed as a function of accumulated plastic strain. The concrete stress-strain relationship under compression is established using the curve for monotonic loading as an envelope. Unloading and reloading before reaching the envelope is assumed to be elastic. The loss of concrete strength due to cracking under tension is accounted for by assigning zero stiffness to the cracked concrete. In the M–ϕ model, a linear strain distribution is assumed across the section. The internal moment is computed from integrating steel and concrete stresses corresponding to the given strain distribution.

The accuracy of the proposed M–ϕ is checked by comparing predicted and measured moment-rotations of a series of half-size "ductile" RC beams designed by the American Concrete Inst. code. These beams were tested under cyclic loading reversals with small shear. Comparison with tests showed that the proposed model offers a reasonable prediction of the M–ϕ hysteretic loops. Analytical results showed that the M–ϕ relationship is mainly controlled by the hysteretic behavior of the top and bottom reinforcement. During most of the cyclic response, cracks that were formed when moment was applied in one sense remained open when subjected to moment of opposite sense due to residual inelastic tensile strains in the new compressed steel. These residual strains resulted from the previous loading which strained the steel well into the plastic range. Since concrete is ineffective in carrying compressive load, the stiffness and strength of a member are controlled by just the top and bottom steel. A study of energy dissipated, i.e., the area of hysteretic loops, showed that a RC section with small amounts of steel at the bottom forces more energy to be dissipated in the bottom steel. From a seismic design point of view, in order to increase energy dissipated through flexural deformation in a plastic hinge under fully reversed deformations, the ratio between the top and bottom steel should be kept close to unity, and closely spaced stirrups should be used at locations of potential plastic hinges. The stirrups provide confinement of concrete and prevent premature buckling of the main beam reinforcement.

● 6.6-31 Hsiu, F. J. and Hanson, R. D., **Inelastic seismic response of turbine buildings,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(a), Paper K 4/3, 12. (For a full bibliographic citation, see Abstract No. 1.2-5.)

Inelastic design of turbine buildings to withstand a safe shutdown earthquake (SSE) is one of the most cost-effective ways to assure that they will not collapse onto adjacent safety-related Category I structures. Application of the Newmark-Hall inelastic response spectra technique to turbine building seismic design is examined analytically for various cases of design. Emphasis is placed on the qualification of the structure to sustain SSE loading at a nuclear power plant site without collapse.

This investigation of the inelastic design and analysis of turbine buildings due to gravity and seismic loading used three common turbine building framing configurations. These structures were designed for various design ductilities using Newmark-Hall inelastic spectrum modification procedure with the Nuclear Regulatory Commission guide 1.60 design response spectra as a base. The DRAIN-2D computer program was used for nonlinear dynamic response computations. Typical hysteresis models for the beam and column behavior were used with nearly elastoplastic nonlinear behavior. The bracing behavior followed the newly developed model in which energy dissipation occurs during tension yielding as well as during post-buckling compression. Nonlinear dynamic responses to a synthetic ground input motion developed to envelop the basic SSE response spectra, to the component of the 1940 El Centro earthquake, and to the N21E component of the 1952 Taft earthquake accelerograms were determined. Limits on the maximum displacement response to avoid structural instability, and limits on member ductilities are major parameters used to qualify a structure as capable of withstanding SSE loading. Displacements, member forces, and member ductilities for the inelastic spectra procedure and nonlinear response computations are compared. The yielded structural displacement response time histories and yield mechanism revealed important characteristics and parameters for turbine building structural design.

This study showed that the inelastic spectral design procedure must be carefully applied for this type of structural system. It is apparent that if the inelastic response of a building can be accurately described by its elastic response characteristics, then this technique will give good results. The turbine building, and possibly many braced frames, do not meet the above criteria of having their nonlinear response accurately represented by an elastic response for significant nonlinearities. This means that the selected design ductility for this system must be smaller than that usually suggested for single-mass systems. Based on this study, a design ductility of three is recommended for this type framing. This is a compromise between the economic advantage of using large ductilities and the factor of safety against collapse.

● 6.6-32 Whitley, J. R. *et al.*, **Base response arising from free-field motions,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,*

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Vol. K(a), Paper K 2/15, 10. (For a full bibliographic citation, see Abstract No. 1.2-5.)

A procedure is illustrated in this paper for deriving from a free-field record the horizontal base motions of a building, including horizontal rotation and translation. More specifically, the goal was to compare results of response calculations based on derived accelerations with the results of calculations based on recorded accelerations. The motions are determined by assuming that an actual recorded ground wave transits a rigid base of a given dimension. Calculations given in the paper were made employing the earthquake acceleration time histories of the Hollywood storage building and the adjacent lot for the Kern County (1952) and San Fernando (1971) earthquakes.

For the Kern County earthquake, the derived base corner accelerations, including the effect of rotation, show generally fair agreement with the spectra computed from the Hollywood storage corner record. For the San Fernando earthquake, the agreement between the spectra computed from derived base corner accelerations and that computed from the actual basement corner record is not as good as that for the Kern County earthquake. These limited studies admittedly are hardly a sufficient basis on which to form a judgement, but these differences noted probably can be attributed in part to foundation distortion, building feedback, distance between measurement points, and soil-structure interaction; it was not possible to take any of these factors into account in these particular calculations.

The original concept of "accidental" torsion arising from seismic wave transmission effects assumes that the irregular nature and spatial variation of the ground waves leads to torsional motion, and it is not clear that treatment as a systematic plane wave, as was done herein, was entirely justifiable. In fact, careful study of the calculations could lead one to conclude that there may be no significant torsional effects present in the Hollywood storage building arising from the two earthquakes studied. Moreover, review of the results presented in the paper suggests that torsional effects in buildings of large plan dimension computed by the technique given may be unreasonably large. Accordingly, based on experience and on earthquake observations, for the present, the authors recommend that current code procedures for accidental torsion be employed in design.

● **6.6-33** Reich, M. and Koplik, B., **Effect of clearance and distribution of mass on the dynamic response of an HTGR core,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(b), Paper K 7/3, 8. (For a full bibliographic citation, see Abstract No. 1.2-5.)

One of the primary factors in determining the structural integrity and, consequently, the safety of an HTGR is the dynamic response of the core when subjected to a seismic excitation. The HTGR core can be modeled as a multimass system with clearance gaps between adjacent elements and surrounded on its outer periphery by reflector blocks, restraining spring packs, and finally the PCRV vessel. Under operational conditions, the magnitude of the clearances change. In addition, the total clearance can change substantially during the active lifetime of the fuel elements due to radiation effects. Earthquake input motions to this type of core arrangement will result in multiple impacts between adjacent elements as well as between the reflector blocks and the restraining spring packs. The highly complex nonlinear response associated with the multiple collisions across the clearance gaps as well as with the spring packs is the subject matter of this paper. Of particular interest is the effect that variations in clearance and the choice of number of masses in the model have on the dynamic response of the HTGR core.

To simulate the response of this system, a model using a single row, or slice, of horizontal elements was adopted for the dynamic investigations. Actual clearances as well as the total mass of a typical horizontal slice were used. The input motion was supplied by the horizontal displacement of the core vessel which in this case was chosen as a sine wave of constant amplitude. However, the frequency of the input was changed for each analysis. Interelemental elastic and damping coefficients were derived from the element geometry and material characteristics. Coefficient of restitution data was obtained from a model test using graphite. The viscous interelemental damping coefficients in the analytical model were adjusted to duplicate the energy dissipated during a collision in the model test. The viscous damping coefficients of the ground-motion dampers were evaluated on the basis that they would dissipate the same energy per cycle as ground friction.

A computer program OSCIL was developed to solve for the response of the system. Using the program, the resonant frequency versus clearance characteristics of the system were determined by noting the frequency at which the maximum force was developed across the boundary springs for a given clearance. By varying the clearance from 0 to 130% of nominal (100% clearance corresponds to 4.6 in. for the full core), the dependence of resonant frequency on total clearance is clearly established. Results for the model chosen, with a fixed input amplitude, show that the resonant frequency varies from 11.4 rad/sec for zero clearance (the natural frequency of the linear system) to 2.5 rad/sec at 100% clearance. Thus, the magnitude of clearance in an HTGR core is shown to have a significant effect on its dynamic response.

In order to investigate the effect of mass lumping, core response outputs for models of 5, 7, 9, and 30 masses were investigated with the clearance fixed at 100% of nominal. Results show that when the mass systems are excited at their resonant frequency, all of the systems have similar

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response characteristics. Indeed, under the conditions stated above, the five-mass system has the same natural frequency as the thirty-mass system and develops the same maximum forces. Therefore, for the existing conditions, a reduced-mass model can be used to represent the dynamic response of the more complex system.

● **6.6-34** Tzung, F.-K., **Study of fuel block collision in HTGR core,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(b), Paper K 7/5, 11. (For a full bibliographic citation, see Abstract No. 1.2-5.)

Analytical and experimental studies have been performed to determine the effects of the collision of fuel blocks in the HTGR core during seismic events. In this paper, the study of contact phenomena, such as stress wave propagation, contact time and rebound velocity, as well as the numerical method suitable for this study, is presented.

The difficulties of treating nonlinear dynamic contact problems have been discussed by many authors. Numerical treatments have been hampered either by the stability of an explicit time integration scheme or the complexity of an implicit scheme. In order to avoid stability problems with the solution scheme, a matrix-vector modification technique, or the Sherman-Morrison method, in conjunction with the finite element method and Newmark's integration scheme, is used. In using finite element methods for structural analysis, the hexagonal graphite block containing fuel holes and coolant holes is idealized by thousands of elements and nodes. The corresponding matrix-vector equation, derived from the variational principle, likewise includes many thousands of components. The decomposition of the matrix for solutions is indeed a time-consuming process, but one which need only be performed once with the method described in this paper. As the surface of the structure comes in contact with another structure, the boundary condition changes are represented by a localized modification to the matrix-vector equation. In considering local plastic deformation, computations are minimized by expressing the relevant modification to the element stiffness matrix in terms of the product of a vector and its transpose. The modification is incorporated into the solution in a manner such that the frequent contact-release phenomenon can be treated with ease. Convergence of the procedure is demonstrated by solving Hertzian problems. Agreement with the Hertz theorem is obtained with a relatively crude mesh representation and time-step size.

Numerical results of corner and flat face impact of similar and dissimilar blocks are presented. A longer contact time and higher peak stress are found as expected for the corner impact. The rebound velocity appears to be lower for blocks with holes than for blocks without holes. Results for a 7-hole block are compared with experiments. The significance of this comparison is discussed.

● **6.6-35** Parker, J. V., Ahmed, K. M. and Ranshi, A. S., **Dynamic response of nuclear power plant due to earthquake ground motion and aircraft impact,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(b), Paper K 9/5, 9. (For a full bibliographic citation, see Abstract No. 1.2-5.)

The reactor primary circuit of a nuclear power station is, in the event of an aircraft crashing onto the facility, protected from direct damage from impacting missiles by the primary containment. The potential for indirect damage does exist however due to vibrations being transmitted through the foundations, the primary circuit components thereby being subjected to base excitations as in the case of ground motion due to earthquakes. This indirect effect of an aircraft crash can be assessed pessimistically by assuming that the aircraft hits the primary containment directly and horizontally near the top of the structure. Furthermore, this pessimism is increased by ignoring the energy absorbed in causing the local damage experienced by the primary containment. This paper examines both the indirect effect of this form of aircraft crash and the effect of earthquake ground motions on the dynamic response of a single reactor nuclear island.

The nuclear island components are idealized using beam elements and rigid elements, the foundation raft being assumed to be rigid. The ground itself is idealized as elastic spring-viscous damper elements. The effect of ground properties on the dynamic response is investigated by varying the ground stiffness and damping over a range defined by the shear wave velocities 500 to 2000 m/sec.

The time history dynamic response is calculated using modal transformations for aircraft impact forcing functions and for earthquake ground motions corresponding to the 1940 El Centro N-S component and the 1966 Parkfield N5W and S25W components. The effect of both the aircraft crash and the earthquake on the reactor plant can be compared directly by computing floor response spectra from the time history response. The precise shape of the forcing function does significantly affect the response and consequently the floor response spectra. However, it is concluded that where nuclear facilities are being designed to ensure a safe shutdown against earthquakes, then provided the primary containment is designed to protect the primary reactor circuit against direct damage from a multirole combat aircraft, the reactor plant within the primary containment will have an acceptable response. In the event of a large aircraft such as the Boeing 707 crashing onto the facility, then the design of the reactor plant could be affected depending upon the amount of energy absorbed locally through direct damage.

● **6.6-36** Holzer, S. M. and Somers, A. E., **Nonlinear model solution process: energy approach,** *Journal of the*

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Engineering Mechanics Division, ASCE, **103**, *EM4,* Proc. Paper 13139, Aug. 1977, 629–647.

The paper provides a comprehensive treatment of the mathematical model and solution process of the computer code SINGER. The function of SINGER is to predict the geometrically and physically nonlinear response, including element failures and structural collapse, of skeletal reinforced concrete and steel structures to static and dynamic loads. The model of SINGER represents an assemblage of one-dimensional finite elements, which is expressed in the form of an energy function. The solution process is based on function minimization. A selection of test problems is presented to provide an indication of the capability of SINGER.

● **6.6–37** Chon, C. T. and Symonds, P. S., **Large dynamic plastic deflection of plates by mode method,** *Journal of the Engineering Mechanics Division, ASCE,* **103**, *EM1,* Proc. Paper 12757, Feb. 1977, 169–187.

The extension of the modal approximation technique to finite deflections is illustrated in this paper by application to a fully clamped circular plate that is impulsively loaded and whose material exhibits rigid-perfectly plastic or viscoplastic behavior. Deflections up to about 10 plate thicknesses are treated. The approximation technique required finding instantaneous mode form solutions at a sequence of times, satisfying the current equations of material behavior, edge constraint, and dynamics including finite deflection terms. The modal form is defined as having velocity and acceleration fields differing by a scalar factor; thus, ordinary rather than partial differential equations are solved. Iterative methods are described which furnish these solutions efficiently in the present problem. The general approach furnishes some useful information about errors of the final deflection magnitudes. Comparisons are made with other solutions and experimental results.

● **6.6–38** Lybas, J. M. and Sozen, M. A., **Effect of beam strength and stiffness on dynamic behavior of reinforced concrete coupled walls,** *UILU-ENG-77-2016, Structural Research Series 444,* Univ. of Illinois, Urbana, July 1977, 2 vols., 569.

This report describes a study aimed at developing an understanding of the response of reinforced concrete coupled wall systems to seismic loading. The study had analytical and experimental phases. Five test structures (approximately one-twelfth scale) were subjected to one component of the earthquake base motion measured at El Centro, California (1940). The base motions were strong enough to cause yielding of the test structures. A sixth test structure was subjected to slowly applied cyclic lateral loading. The experimental program is outlined, and the results are presented. The details of experimental procedures, along with the characteristics of the test specimens and materials, are given. An analytical study of the static hysteretic response of the test structures was undertaken. The effect of the hysteresis relations of the members on the overall hysteresis relation of the structure was studied. Equivalent viscous damping factors, consistent with the calculated overall structure hysteresis relation, were determined. The variation of damping factor with response mode and response amplitude was studied. The study of static hysteretic response is presented. The feasibility of simulating the observed dynamic responses with a linear viscously damped analytical model was investigated. Both response spectrum analyses and response history analyses were performed. Finally, the experimental results were compared with the results of the analytical studies.

● **6.6–39** Hsu, C. S., Cheng, W. H. and Yee, H. C., **Steady-state response of a non-linear system under impulsive periodic parametric excitation,** *Journal of Sound and Vibration,* **50**, *1,* Jan. 8, 1977, 95–116.

The nonlinear problem of an elastically restrained and damped bar subjected to a periodic impact load is studied in this paper. The possible steady-state response of such a system is examined in a systematical manner. The approach used is to formulate the problem first in terms of nonlinear difference equations and then to investigate the periodic solutions of these equations and their stability character. In this manner, the problem is thus linked to the idea of diffeomorphisms in the theory of differentiable dynamics. It is believed that the application of this approach given in this paper is a novel and effective one. For the particular problem investigated, many important nonlinear features, such as limit cycle response and bifurcation phenomenon, can be studied in a rather complete fashion. The exact treatment given in this paper and the results thus obtained could provide a useful guide to studies of other much more complex problems of similar nature.

● **6.6–40** Hitchings, D. and Ward, P., **The steady state response of geometrically non-linear shallow arches,** *International Journal for Numerical Methods in Engineering,* **11**, *8,* 1977, 1261–1269.

The nonlinear steady state response of structures with curvature is investigated using a shallow circular arch as an example. A consistent mass finite element formulation is employed to derive the governing nonlinear differential equations. These equations are solved by assuming a single mode expansion reducing the governing equations to the single degree-of-freedom Duffing's equation with a quadratic term. The nonsymmetric amplitude-frequency curve is derived and compared with results previously obtained by direct integration of the equations of motion.

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● **6.6-41** Belytschko, T., Schwer, L. and Klein, M. J., **Large displacement, transient analysis of space frames,** *International Journal for Numerical Methods in Engineering,* **11,** *1,* 1977, 65–84.

A formulation is presented for the transient analysis of space frames in large displacement, small strain problems. For purposes of treating arbitrarily large rotations, node orientations are described by unit vectors; and deformable elements are treated by a corotational (rigid-convected) description. Deformable elements may be connected either to nodes directly or through rigid bodies. The equations of motion are integrated by an explicit procedure. Sample results are presented on the snap-through of an arch-type structure.

● **6.6-42** Drenick, R. F., **The critical excitation of nonlinear systems,** *Journal of Applied Mechanics,* **44,** Series E, *2,* June 1977, 333–336.

The critical excitation of a mechanical system, in the terminology of this paper, is one that drives the system to a larger response peak than any other in some class of allowed excitations. The critical excitation is of interest in questions related to reliability and safety because the magnitude of the response peak is frequently an indicator of the survivability of the system. The problem of finding the critical excitation was solved for linear systems some time ago. This paper deals with the generalization of the problem to nonlinear systems. It is shown that its solution is in many ways analogous to its earlier counterpart.

● **6.6-43** Morris, N. F., **The use of modal superposition in nonlinear dynamics,** *Computers & Structures,* **7,** *1,* Feb. 1977, 65–72.

This paper decribes the application of the modal superposition method to the calculation of the nonlinear dynamic response of structures. Although this method has often been used in the analysis of linear response, it has seldom been applied in nonlinear problems. The reasons for this are obvious and seem to be quite valid, but the examples considered herein indicate that modal superposition may be a more useful computational tool than has heretofore been imagined. Four different problems are considered in this paper: a three-dimensional cable-stayed bridge, a double-layered three-dimensional cable network, an unstiffened suspension bridge, and a three-dimensional elastic-plastic frame. The problems are chosen to exemplify both the strengths and weaknesses of the modal superposition method.

● **6.6-44** Whitman, R. V. and Protonotarios, J. N., **Inelastic response to site-modified ground motions,** *Journal of the Geotechnical Engineering Division, ASCE,* **103,** *GT10,* Proc. Paper 13269, Oct. 1977, 1037–1053.

A building with a period equal to that of a site may be more susceptible to yielding during a moderate earthquake, but the larger yielding during a major earthquake is much the same as for a building having a different period. This conclusion results from analyzing one degree-of-freedom, elastoplastic structures using ground motions (both real and calculated) whose elastic response spectra have peaks attributable to site conditions. Inelastic response spectra for site-modified motions do not show pronounced peaks at the period of the site; instead, they are as "smooth" as inelastic spectra computed from motions unaffected by site conditions. Inelastic spectra for design may be based upon the same ratios of spectral acceleration to peak acceleration and spectral velocity to peak velocity as for normal motions. Thus, the amount by which a site modifies peak acceleration and peak velocity is important, and the period of a site is not significant by itself.

● **6.6-45** Florence, A. L., **Response of circular plates to central pulse loading,** *International Journal of Solids and Structures,* **13,** *11,* 1977, 1091–1102.

An analysis is presented for the response of a clamped circular plate subjected to a rectangular pulse uniformly distributed over a central circular area. The plate is rigid-perfectly plastic with yielding according to the Johansen criterion and the associated flow rule. Bending is assumed to be the predominant response. Simple formulas are obtained for the permanent central deflection for all pressures and loaded areas.

● **6.6-46** Nonaka, T., **Approximation of yield condition for the hysteretic behavior of a bar under repeated axial loading,** *International Journal of Solids and Structures,* **13,** *7,* 1977, 637–643.

This paper is concerned with the hysteretic behavior of a prismatic bar subjected to repeated axial loading. Elastic-perfectly plastic behavior is assumed in the analysis under the combined action of axial force and bending moment. The fully plastic states in pure bending and in pure tension and compression are bilinearly interpolated to serve as the yield condition; linearly interacted regimes are combined with interactionless regimes of pure bending to form the yield hexagon, which is a modification of a previously assumed yield quadrangle. Basic equations are derived through the analysis, expressed in a simple analytic closed form, and can be used to determine the load-deformation relationship of the bar for any specified history of axial loading.

As a result of the analysis it follows that, due to the load cycle of tension and compression, an initially straight and plastically bent bar undergoes a plastic extension upon the recovery of the straight configuration. This is the balanced axial deformation at a yield hinge between extension and contraction, taking place during the cycle of hinge rotation. Another characteristic feature found is that cyclic alternate displacement loading with large amplitude leads to a steady state after some repeated deterioration of processes. Comparison is made of the load-axial displacement relation between the analytical and experimental results to show reasonable agreement. Discussions extend to the appropriateness of the form of the yield condition.

● **6.6-47** Derecho, A. T., Freskakis, G. N. and Fintel, M., **Inelastic seismic response of isolated structural walls,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1148–1153.

Observations of building damage due to earthquakes over the last two decades have gradually led engineers to recognize the need for buildings which not only survive strong ground motions without collapse but survive these motions with a minimum of damage to both structural and nonstructural components. Thus, it is felt that, in addition to the primary requirement of life safety, a requirement for damage control under strong ground motion should be included in the performance criteria for buildings in seismic areas. The superior performance of reinforced concrete multistory structures stiffened by properly proportioned structural walls (i.e., shear walls), particularly with respect to damage control, has focused attention on the need for more information on this type structure. The results reported here represent part of the data obtained during the first phase of an analytical investigation aimed at determining the force and deformation requirements in structural walls and wall systems when subjected to strong ground motions. The analytical study forms part of a combined analytical and experimental program undertaken at the Portland Cement Assn.

● **6.6-48** Iwan, W. D., **The response of simple stiffness degrading structures,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1094–1099.

The response of six different yielding and stiffness-degrading systems to an ensemble of six earthquakes is examined. Average response spectra are given for each system for periods ranging from 0.4 to 4.0 sec and ductilities ranging from 0.6 to 8.0. The response spectra are used to define equivalent linear system parameters for both the simple yielding and stiffness-degrading systems.

● **6.6-49** Takizawa, H. and Jennings, P. C., **Ultimate capacity of lowrise R/C buildings subjected to intense earthquake motion,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1052–1057.

The ultimate capacity of lowrise R/C buildings to resist intense ground motions is examined extensively by use of planar structural models. The effects of cracking, yielding, crushing and spalling, and stiffness degradation are included in the analysis along with the nonlinear geometrical effect associated with the overturning action of gravity. The study is intended to provide a realistic description of the process of ultimate failure in R/C flexural (flexural-shear) frames and to clarify their margin of safety when subjected to severe earthquake motion.

● **6.6-50** McGuire, R. K., **Response of simple elastoplastic systems to earthquakes of various magnitudes and distances,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1016–1021.

The responses of one degree-of-freedom elastoplastic structures to earthquake motions is examined by least-squares regression using magnitude and distance as independent variables. The distribution of peak elastoplastic displacement at a site subjected to a range of seismic motions from a nearby fault is well-approximated by the distribution of displacement of an equivalent elastic system, substantiating results reported by others based on comparison of displacements during individual motions. Total hysteretic energy absorbed by the elastoplastic models is used as a measure of damage; it is found that, contrary to an often-expressed notion, two earthquakes (one a small, close event; the other a large, distant event) which produce the same expected elastic displacements are also expected to produce approximately the same amount of hysteretic energy in elastoplastic systems. For events with the same peak ground acceleration, the large, distant event generally produces more damage.

● **6.6-51** Freeman, S. A., **Approximating inelastic response of structures to ground shaking,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1142–1147.

Inelastic response of structures to ground shaking can be approximated by a practical analytical alternative that is more sophisticated than a typical elastic analysis but less complex than an inelastic time-history procedure. Data from two real structures and actual recorded motion are used to illustrate the procedure. Elastic and inelastic capacities of the structures are estimated and are reconciled by graphical methods with the demand response spectra of ground shaking events. The solution results in values for peak structural response, peak ductility demands, equivalent response periods of vibration and percentages of critical damping, and reserve capacities.

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● **6.6-52** Kan, C. L. and Chopra, A. K., **Earthquake response of a class of torsionally coupled buildings,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1211-1216.

Results of an investigation of earthquake response of buildings for which the lateral motion is coupled with the torsional motion are summarized. The relationships between the story forces in a torsionally coupled system and those in a corresponding torsionally uncoupled system are presented, and the effects of torsional coupling on earthquake forces are identified.

● **6.6-53** Cheng, F. Y. and Oster, K. B., **Ductility studies of parametrically excited systems,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1118-1123.

The method proposed for studying ductility and excursion ratios of inelastic frameworks subjected to parametric earthquake motions of one horizontal and vertical components employs a mathematical formulation based on elastic and dissipated strain energy. The numerical results derived for various structures when compared with those obtained by using other conventional methods illustrate the advantages of the method and show the significant effect that coupling earthquake motions have on ductility requirements.

● **6.6-54** Paulay, T. and Santhakumar, A. R., **Ductile behaviour of shear walls subjected to reversed cyclic loading,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1201-1210.

Possible failure modes in cantilever shear walls and their suitability as energy-dissipating mechanisms are reviewed. From the analogy of a ductile shear fiber, the earthquake-resistant potential of coupled shear walls is discussed. The development of ductile coupling beams and their beneficial influence on the overall elastoplastic response of two large experimental reinforced concrete models are discussed.

● **6.6-55** Aneja, I. K., **Seismic response of floating nuclear power plant equipment (turbine-generator on steel foundation),** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2665-2672.

Seismic response (maximum accelerations, displacements at critical points), due to a sample shock spectrum in the vertical direction, of turbogenerator steel foundation equipment designed for a 1200 Mw floating nuclear power plant was obtained. The required dynamic characteristics of the complex total structural system were obtained by using a large general purpose finite element computer program. These characteristics were then used in a seismic response program in order to obtain the upper bound values of responses to a sample shock spectrum using the modal superposition technique.

● **6.6-56** Brankov, G. and Sachanski, S., **Response of large-panel buildings for earthquake excitation in nonelastic stage,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1330-1342.

The response of a six-story large-panel building is investigated, taking into account inelastic deformations in the wall-panel connections. The 1971 San Fernando earthquake S16°E component is used. The equations of motion are solved using a step-by-step integration procedure, changing the stiffness in each successive interval. Inelastic strength and deformation characteristics of the wall-panel connections under reversed loads are determined by theoretical and experimental investigations. Skeleton curves, hysteresis loops, ductility factors, and absorption of energy in terms of the vertical and horizontal load history are determined. The influence of the inelastic deformations on the redistribution of forces among the vertical diaphragms is investigated.

● **6.6-57** Dalal, J. S. and Perumalswami, P. R., **Overturning behavior of nuclear power plant structures during earthquakes,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2620-2626.

Nuclear power plant structures are designed to withstand severe postulated seismic forces. Structures subjected to such forces may be found to overturn if the factor of safety is computed in the traditional way, treating these forces as static. This study considers the transient nature of the problem and draws a distinction between rocking, tipping, and overturning. Responses of typical nuclear power plant structures to earthquake motions are used to assess their overturning potential more realistically. Structures founded on rock and soil are considered. It is demonstrated that the traditional factor of safety when smaller than unity indicates only minimal base rotations and not necessarily overturning.

● **6.6-58** Okada, T. and Seki, M., **A simulation of earthquake response of reinforced concrete buildings,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2723-2728.

Nonlinear earthquake response of reinforced concrete one-story building frames to a recorded ground motion was simulated by a computer-actuator online system and the results were compared with the results of a computer-simulated analytical model (OS-model), using a hysteretic loop developed by the authors. The OS-model considers the

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nonlinear stress-strain relationships of concrete and steel. Four frames of the column-yielding type having different natural periods were tested using the online system and one was tested under cyclic loading. The earthquake response obtained using the online test showed fairly good agreement with that using the proposed computerized analytical model.

● **6.6-59** Gulkan, P., **The inelastic response of repaired reinforced concrete beam-column connections,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2473–2479.

Results obtained from two 3/4-size beam-column test specimens, subjected to alternated loads applied at the beam ends before and after damaged columns were repaired, are described. The repair technique in which the original shell was chipped off and replaced with additional reinforcement by a thicker shell improved the strength of the subassemblage considerably. It was observed that the joint core where the repair process remains ineffective may then become the critical location. There appears to be no reduction of the ductility and energy absorption capacity of the specimens after repair.

● **6.6-60** Prasad, B. K. R. and Jagadish, K. S., **The inelastic torsional response of structures to earthquake ground motions,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1395–1396.

The torsional response of a single-story framed structure with hysteretic behavior is studied. The displacement ductility of various frames subjected to the El Centro records is obtained by numerical integration of the equations of motion.

● **6.6-61** Irwin, A. W. and Andrew, N., **Performance of model reinforced concrete cores for tall buildings cycled in torsion,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3241–3242.

6.6-62 Hashmi, M. S. J. and Al-Hassani, S. T. S., **Large-deflection response of square frames to concentrated impulsive loads,** *The Journal of Mechanical Engineering Science,* **19,** *6,* Dec. 1977, 243–250.

A study is made of the elastic-plastic responses of portal and free-square frames subjected to concentrated explosive impulses. A finite-difference method, which reduces the structure to small masses connected by light links, is used to solve the equations of motion of the deforming frames. The links are assumed to have the same strength properties as the material of the structure. Dynamic strain is measured experimentally, instantaneous profiles are obtained from high-speed photographs, and the final deflections substantiate the predictions of the analytical method.

● **6.6-63** Hadjian, A. H., **On the decoupling of secondary systems for seismic analysis,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3286–3291.

The coupled analysis of primary (supporting) and secondary (supported) systems may not always be feasible or desirable. Decoupling of secondary systems from primary systems from the viewpoint of acceptable errors in frequency and response is studied. It is concluded that the presently used criteria for decoupling are arbitrary and conservative. Due to the difficulties of interpreting the results of studies of simple systems for use in multidegree-of-freedom models, the concept of mass ratio is generalized to reflect the modal masses of the subsystems. The impact of the location of the secondary system is also reviewed. A more general and rational criteria for decoupling is suggested.

● **6.6-64** Iwan, W. D., **Predicting the earthquake response of resiliently mounted equipment with motion limiting constraints,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3292–3297.

A method is presented whereby the response spectrum may be used to predict the response of an isolation system with nonlinear motion limiting constraints. The results of the approximate method are compared with the results obtained from direct numerical integration. Observations are made on the role of various system parameters in determining the response.

● **6.6-65** Kobori, T., Inoue, Y. and Kawano, M., **Elasto-plastic earthquake response of frames with shear wall,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3037–3042.

A wall-frame system is modeled as a single-story, single-bay frame infilled with a wall. The finite element method is used to model the elements, which are applicable to von Mises-type yielding. The dynamic responses of a system having masses concentrated at nodal points and a stiffness matrix are evaluated. The results are compared for the wall-to-frame stiffness and strength ratios.

● **6.6-66** Minami, K. and Wakabayashi, M., **Seismic resistance of reinforced concrete beam-and-column assemblages with emphasis on shear failure of column,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3101–3106.

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The main objective of this investigation was to determine the elastic-plastic behavior of reinforced concrete assemblages with emphasis on shear failure of columns subjected to monotonically increasing and well-defined alternately repeated horizontal loads. The principal variables of the experimental program were (1) the shear span ratio of beam and column, (2) the web reinforcement ratio of column, and (3) the axial load ratio. The effects of experimental parameters on the strength, stiffness, ductility, failure mechanism, and energy absorbing capacity were carefully studied. An analytical model, based on modified shear-friction theory, was formulated to simulate the behavior of the column and comparisons were made between the calculated and observed behavior.

● **6.6-67** Wakabayashi, M., Matsui, C. and Mitani, I., **Cyclic behavior of a restrained steel brace under axial loading,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3181–3187.

In order to clarify the effects of end restraints on the cyclic behavior of braces, steel bars restrained against rotation at both ends were tested under alternating axial load, the slenderness ratio and stiffness of the end restraints being varied. The approximate analysis sufficiently predicts the experimental behavior. It is concluded that important parameters representing the resistance of braces against earthquake motion can be estimated by the proposed empirical formulas when the effective slenderness ratio of the braces is given.

● **6.6-68** Mitani, I., Makino, M. and Matsui, C., **Influence of local buckling on cyclic behavior of steel beam-columns,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3175–3180.

In order to clarify the influences of local buckling on the hysteretic restoring force characteristics of steel beam-columns, cantilever steel columns of H-shape cross section were tested under constant axial load and alternating horizontal load. Three parameters studied in the tests are the width-to-thickness ratios of the flange and the web, and the axial load ratios. Test results clearly show the deterioration of the restoring force due to local buckling, particularly due to the web buckling occurring after the flange buckling. A set of empirical formulas are proposed to relate the plastic deformation capacity of the beam-column to the width-to-thickness ratio of plate elements.

● **6.6-69** Matsui, C. and Mitani, I., **Inelastic behavior of high strength steel frames subjected to constant vertical and alternating horizontal loads,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3169–3174.

In order to clarify the restoring force characteristics of high strength steel frames, portal frame specimens were tested under constant vertical and alternating horizontal loads, the vertical load ratio and mechanical properties of the steel being varied. The results of the analysis predict well the experimental behavior, including the upper limit value of the load-carrying capacity, the plastic energy absorption capacity, and the amount of plastic strain accumulated in the section.

● **6.6-70** Takanashi, K., Udagawa, K. and Tanaka, H., **A simulation of earthquake response of steel buildings,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3156–3162.

This paper presents part of ongoing research on seismic design criteria for steel buildings and describes simulation results of the earthquake response of steel structures with inelastic beams. The response analysis was made using a computer actuator on-line system developed by the authors. In order to maintain the stability of structures under a strong earthquake, the permissible magnitude and number of excursions of plastic response deformations of beams have been discussed in comparison with the rotation capacities obtained by the monotonic and cyclic reversed loading tests of beams. The earthquake response of the analytical models of the same structures as those of on-line tests have also been calculated, assuming the hysteresis loops of beams to be the Ramberg-Osgood type and bilinear functions. The adaptability of the analytical model analyses was investigated.

● **6.6-71** Aoyama, H., Umemura, H. and Minamino, H., **Tests and analyses of SRC beam-column subassemblages,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3081–3088.

Tests of nearly full-size steel-reinforced concrete (SRC) subassemblages provided data of the process of destruction of beam-column connections due to excessive shear in transmitting antisymmetric moments in the members. Ultimate strength of connections was satisfactorily predicted, but it was also found that the prediction of ultimate strength of SRC members, which dictates the ultimate external force to the connections, was more difficult than the prediction of ultimate strength of reinforced concrete members. A simplified method to utilize subassemblage test data in the earthquake response analysis of prototype SRC frames is presented.

● **6.6-72** Park, R. and Thompson, K. J., **Some recent research in New Zealand into aspects of the seismic resistance of prestressed concrete frames,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3227–3232.

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The results of an investigation of the seismic resistance of prestressed and partially prestressed concrete frames are described. To obtain seismic design information, the experimental part of the project involved the testing of ten nearly full-scale beam-interior column assemblies under static cyclic loading. Theoretical moment-curvature studies were conducted to establish sectional properties that would lead to ductile behavior. Nonlinear dynamic analyses of single degree-of-freedom systems were conducted using accurate idealizations for the load-displacement loops to compare the behavior of prestressed, partially prestressed, and reinforced concrete systems responding to severe earthquakes.

● **6.6-73** Takayanagi, T. and Schnobrich, W. C., **Computed behavior of coupled shear walls,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3069–3074.

A procedure using a modified frame method is presented for the nonlinear analysis of coupled shear wall structures subjected to strong earthquake motions. The walls and coupling beams are replaced by flexural elements. Axial flexural and shear rigidities of the wall members are considered in the analysis. The coupling beams are considered to be individual beams connected to the walls through a rigid link and rotational springs. The hysteresis rules used are an adaptation of those presented by Takeda et al. A linear acceleration method is used for the solution of the equations of motion.

● **6.6-74** Atalay, B. and Penzien, J., **Inelastic cyclic behavior of reinforced concrete flexural members,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3062–3068.

Presented is a mathematical model for predicting the force-deformation characteristics of reinforced concrete flexural members under inelastic cyclic conditions.

● **6.6-75** Powell, G. H., **Analysis of earthquake effects on pipelines,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3364–3368.

A procedure for computing the seismic response of cross-country pipelines is described, with emphasis on above-ground lines. The procedure accounts for the effects of initial static loads, support nonlinearity, and out-of-phase ground motions at different points along the pipe. The idealization assumptions and theory are described, and the influence of certain parameters on the response is discussed.

● **6.6-76** Tani, S. and Nagasaka, T., **An analytical study on restoring force characteristics of reinforced concrete framed structures,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 2106–2110.

An analytical approach is developed which determines the restoring force characteristics of reinforced concrete framed structures on the basis of the geometrical properties and the material properties obtained from simple tests. The validity is confirmed by comparing analytical results with the test results. The approach is applied to several model framed structures having sectional bracing of differing arrangements at each story.

● **6.6-77** Caughey, T. K. and Vijayaraghavan, A., **Forced oscillations of a piecewise-linear non-linear dynamic system with several degrees of freedom,** *International Journal of Non-Linear Mechanics,* **12,** *6,* 1977, 339–354.

Exact periodic solutions are derived for a dynamic system with several degrees-of-freedom consisting of a series of Reid springs with piecewise-linear nonlinear characteristics. The solutions, however, are restricted to a class of harmonic excitation in the modal form described in the paper. Conditions are derived for the asymptotic stability of the periodic solution and an example has been worked out in detail on the response of a dynamic system with two degrees-of-freedom.

● **6.6-78** Holmes, P. J., **Behaviour of an oscillator with even non-linear damping,** *International Journal of Non-Linear Mechanics,* **12,** *5,* 1977, 323–326.

This paper proves two theorems on the behavior of a single degree-of-freedom oscillator with linear stiffness and even nonlinear damping terms.

● **6.6-79** Yamada, M. *et al.,* **Cyclic deformation behaviour of reinforced concrete shear walls,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3075–3080.

Alternate cyclic loading tests on shear walls were carried out to investigate the fundamental cyclic behavior of reinforced concrete shear walls. The representative hysteresis loop characteristics of reinforced concrete shear walls are clarified for the three types of cyclic loading with constant displacement amplitudes, with constant load amplitudes, and with incremental displacement amplitudes. Specifically, the authors examine the physical significance of the hyteresis loops, the energy absorption as a result of hysteresis, and the low-cycle fatigue of reinforced concrete shear walls by using an idealized mechanical model with equivalent compressive concrete bracing. Finally, the proposed mechanical idealization clarifies the hysteresis characteristics of reinforced concrete shear walls.

● **6.6-80** Iwan, W. D. and Miller, R. K., **The steady-state response of systems with spatially localized non-linearity,** *International Journal of Non-Linear Mechanics,* **12**, 3, 1977, 165–173.

This paper presents an approximate analytical approach for determining the steady-state response of a class of systems with spatially localized nonlinearity. Fundamental properties of the response are identified. An example illustrates the nature and accuracy of the results of the approximate analysis.

● **6.6-81** Irwin, A. W. and Bolton, C. J., **Torsion of tall building cores,** *Proceedings, The Institution of Civil Engineers,* Part 2, **63**, Paper 8015, Sept. 1977, 579–591.

Results from elastic torsion tests on two reinforced concrete and one acrylic model tall building cores and from elastoplastic torsion tests on the reinforced concrete model structures are presented. Values of rotation, strain, and warping displacements computed using a theoretical analysis are compared with the results of the model tests.

● **6.6-82** Clough, R. W. and Huckelbridge, A. A., **Preliminary experimental study of seismic uplift of a steel frame,** *UCB/EERC-77/22,* Earthquake Engineering Research Center, Univ. of California, Berkeley, Aug. 1977, 171. (NTIS Accession No. PB 278 769)

This study represents the preliminary portion of a research program into the effects of allowing column uplift in steel building frames responding to severe seismic loading. Included in this report are experimental and analytical results for a 3-story steel frame, both with and without column uplift allowed. Uplift response results are presented for tests using two sets of impact elements with stiffnesses differing by approximately an order of magnitude.

Allowing column uplift is shown for this frame to significantly reduce both the seismic loading and ductility demand, when compared to the fixed base response for a similar input motion. An analytical technique employing bilinear elastic foundation support elements, with no tensile capacity or stiffness in the upward direction, is shown to accurately predict the uplift response of this frame, even in the presence of large rigid-body rotations. An analytical technique using concentrated bilinear plastic hinges is shown to accurately predict the nonlinear fixed base response for moderately nonlinear response.

● **6.6-83** Roeder, C. W. and Popov, E. P., **Inelastic behavior of eccentrically braced steel frames under cyclic loadings,** *UCB/EERC-77/18,* Earthquake Engineering Research Center, Univ. of California, Berkeley, Aug. 1977, 335. (NTIS Accession No. PB 275 526)

A unique, practical structural system, the eccentric bracing system, which possesses many advantages in the seismic design of steel structures, is described in this work. This system employs diagonal braces with deliberately large eccentricities with respect to the beam-column joint. The eccentricity is introduced to provide a ductile fuse which will prevent brace buckling at extreme loads, such as those that may occur during a severe shake, and to avoid the poor energy dissipation characteristics which result from this buckling. The system is also stiff structurally, since linear elastic analysis indicates that the lateral stiffness remains essentially constant over a wide range of small-to-moderate eccentricities. Therefore, eccentrically braced frames offer the elastic strength and stiffness of a braced frame and the energy dissipation of a steel moment-resisting frame. The system is suitable for the design of earthquake-resistant structures, and it has numerous potential applications.

Short beams which initially yield in shear are tested in cyclic loading. These beams were designed to simulate the behavior of an eccentric element. It was found that cyclic shear yielding of the eccentric element is the most desirable energy dissipation mechanism because of its greater stability during large cyclic deflections. An analytical model for predicting the behavior of such beams is developed from the test results. The model is based on sandwich beam theory, which includes the effect of cross-sectional warping caused by shear yielding. The inelastic model is used to perform inelastic dynamic analysis of a 20-story eccentrically braced prototype structure under the 1.5 times El Centro and unreduced Pacoima Dam acceleration records. The results of these analyses are compared with the computed response for similar ordinary braced and moment-resisting frames. The eccentric bracing system performs well in this comparison because it combines strength, stiffness, and energy dissipation. The moment-resisting frame did not have sufficient strength of stiffness, and the ordinary braced frame lacked good energy dissipation; thus, the alternate structures did not perform as well as the eccentric system.

Two one-third scale model eccentrically braced test frames were designed and tested. The frames were three stories high, and were modeled to represent the lower corner of the 20-story prototype structure. The loading program simulates the response of the eccentrically braced frame under the 1.5 times El Centro and unreduced Pacoima Dam acceleration records in sequence. The tests indicate that the eccentrically braced frame can be expected to survive two such sequential earthquakes without a structural failure. Further, for severe earthquakes of this intensity, the frames exhibit very sound, unpinched hysteresis loops which do not deteriorate in strength or stiffness. The tests are also compared with the inelastic model, and the comparison indicates that the behavior predicted by the model is in very close agreement with the test results.

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Finally, design recommendations are made. The combination of these, the analytical procedures, and the test results can be used to produce structures which are able to withstand severe earthquake excitations. In addition to applications in building design, which were emphasized in this work, the eccentric bracing system offers attractive possibilities for supports of water towers, large-span roofs, and other systems in areas with severe seismic activity.

● 6.6-84 Huckelbridge, A. A., **Earthquake simulation tests of a nine story steel frame with columns allowed to uplift**, *UCB/EERC-77/23*, Earthquake Engineering Research Center, Univ. of California, Berkeley, Aug. 1977, 189. (NTIS Accession No. PB 277 944)

The computed lateral loading imposed on a structure during a major earthquake will often produce an overturning moment in excess of the dead-weight overturning resistance of the system. Assuming that no supplementary anchorage capacity is provided, this condition implies a transient uplift of the structure from its foundation. Linear structural dynamic analysis techniques are not capable of treating this type of highly nonlinear response.

This report presents experimental and analytical response data for a model nine-story building frame under seismic excitation, both with and without supplementary anchorage of the columns provided. The experimental work was carried out on the shaking table of the Univ. of California, Berkeley, Earthquake Simulator Lab. Appreciable amounts of column uplift were observed in the tests for which column uplift was permitted, with significant reductions in the lateral loading, when compared to the fixed base response.

An analytical technique employing bilinear foundation support elements with zero tensile capacity and stiffness in the upward direction is shown to predict the uplifting response with excellent accuracy. Analytical predictions of the nonlinear fixed base response, employing concentrated bilinear plastic hinges, are also shown to be accurate for the levels of nonlinearity encountered.

From the results of this study, it appears that intentionally designing uplifting capability into prototype structures in regions of high seismicity would be both rational and economical. The lateral loading and/or ductility requirements under severe seismic excitation could be significantly lowered, resulting in potential cost savings for the superstructure system. In addition, there is a potential cost savings in the substructure system by eliminating the necessity of providing tensile capacity to resist high overturning moments.

● 6.6-85 Soni, R. Y., Krishna, J. and Chandra, B., **Energy approach to earthquake resistant design**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. II, 1977, 1848-1853.

This paper examines the seismic behavior of structures through the mechanics of energy absorption, which is governed by the force deflection relation. An attempt is made to relate the seismic energy demand with the associated ductility for elastoplastic systems. Two parameters, namely, "energy coefficient" and "sway ratio," are suggested to study the seismic behavior of the systems; and a method of design, using these parameters, is proposed.

● 6.6-86 Yamada, M. and Kawamura, H., **Resonance - fatigue - characteristics for evaluation of the ultimate aseismic capacity of structures**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. II, 1977, 1835-1840.

A method, based upon fracture phenomena, for the evaluation of the ultimate aseismic capacity of structures is described. By comparison with earthquake wave characteristics, deterministic and probabilistic evaluation methods are given. Steady-state resonance and steady forced vibration are compared. A new concept, resonance capacity or resonance coefficient, is proposed to show the energy absorption capacity of structures. Experimental resonance fatigue characteristics of reinforced concrete rigid frames and shear walls are given, and an aseismic design philosophy for reinforced concrete structures is presented.

● 6.6-87 Smilowitz, R. and Newmark, N. M., **Seismic force distributions for computation of shears and overturning in buildings**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. II, 1977, 1819-1825.

Calculations were made for a series of buildings with varying degrees of frame and shear wall interaction considered in their design. Using a simplified response spectrum, the authors have determined shear and flexural moment (overturning moment) distributions over the height of these buildings and have given equations for the corresponding seismic forces. The results differ somewhat from those in a previous study, but are useful in the preliminary design, or possibly even in the final design, of multistory buildings.

● 6.6-88 Shibata, A. and Sozen, M. A., **Substitute-structure method to determine design forces in earthquake-resistant reinforced concrete frames**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. II, 1977, 1905-1910.

The substitute structure method of determining seismic design forces in multistory reinforced concrete frames is described. The method, which recognizes energy dissipation in the nonlinear range of response, utilizes substitute linear models and response spectra. The paper includes a

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description of the substitute structure method, an example of its use in determining design forces in an eight-story frame, and the results of a nonlinear response analysis of the designed frame to earthquake motions.

● **6.6-89** Korchynsky, I. L. and Aliev, G. A., **Behaviour of structures during earthquakes beyond the limits of the elastic stage,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2861-2867.

● **6.6-90** Fintel, M., Schultz, D. M. and Iqbal, M., **Design and construction of large-panel concrete structures, report 2, philosophy of structural response to normal and abnormal loads,** Portland Cement Assn., Skokie, Illinois, Aug. 1976, 133.

Response of large panel structures to both normal and abnormal loadings is discussed qualitatively. Conditions particular to these structures which can affect their behavior are indentified and methods for reducing the risk of progressive collapse from abnormal loads are examined. A philosophy of design based on bridging local damage is introduced which emphasizes continuity and ductility within and across connections. A methodology is developed to be used in establishing minimum detailing criteria to ensure integrity of the structure. In this general structural integrity approach, code requirements for minimum ties will be provided to assure an alternate path for the load in the event of a local failure of a primary load bearing element. Design considerations for both individual elements and connections are discussed conceptually.

● **6.6-91** Nelson, I. and Weidlinger, P., **Development of interference response spectra for lifelines seismic analysis,** *IR-2,* Weidlinger Assoc., New York, July 1, 1977, 44.

● **6.6-92** Pique, J. R., **On the use of simple models in nonlinear dynamic analysis,** *R76-43,* Dept. of Civil Engineering, Massachusetts Inst. of Technology, Cambridge, Sept. 1976, 280.

An evaluation of simple models to predict the inelastic dynamic response of frames is presented. Two types of models were considered, a shear-beam model with the floor springs determined on the basis of an incremental static analysis, and an equivalent one degree-of-freedom system resulting from the same analysis. Three heights of frames were studied, a four-story frame and three ten-story frames. Analyses were performed under El Centro 1940 (N-S) earthquake for different intensities and Taft (N69W).

The simple models can provide a reasonable approximation to the response, with the largest discrepancies of the order of 27%. Solutions obtained with the equivalent single degree-of-freedom system are comparable to those of the shear-beam model or even better for the four- and ten-story frames. For the taller structure, however, the response deteriorates slightly. Changes in motion intensity did not seem to alter the degree of accuracy of the response; neither did the different ground motions. The structural responses were, however, very different for the two earthquakes. When the bending yield criterion was used, an improvement in the agreement of the responses was obtained. Local member ductilities were estimated, based on the predictions from the simple models and the static analysis. The results show reasonable agreement with the point-hinge model response.

● **6.6-93** Ferritto, J. M., **Optimum dynamic design of nonlinear reinforced concrete slabs under blast loading,** *Technical Note N-1494,* Civil Engineering Lab., U.S. Naval Construction Battalion Center, Port Hueneme, California, July 1977, 47.

A computer program was developed to determine the nonlinear dynamic response of reinforced concrete slabs subjected to blast pressure loading. Given the explosive parameters and geometry of the slab, the program computes the blast environment and the structural resistance, mass, and stiffness of the slab and solves for the dynamic response. The program contains optimization subroutines that provide for automatic optimum design of least-cost structural slabs. The program will assist engineers in the design and analysis of facilities that are intended to contain the effects of accidental explosions. The report gives a user's guide and sample problems with data input and program output.

● **6.6-94** Paskaleva, I., **Nonelastic hysteretic behaviour of RC columns and infilling walls,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3133-3137.

Investigation of the response of structures to seismic excitation requires the determination of the inelastic behavior of the principal structural and nonstructural elements, i.e., columns, beams, and infilling walls. Reinforced concrete columns with different cross sections, reinforcements, and stirrups are tested for axial load combined with reversed bending and shearing. The influence of the stirrups space on the bearing capacity and history of distortion is investigated by the testing of short columns. The stiffness degradation, mode of failure, ductility factor, and absorption of energy in terms of the vertical and horizontal load are determined. The bearing capacity of the columns subjected to compression, shearing, and bending in terms of longitudinal and cross reinforcement is analytically represented. The results obtained can be used for the investigation of the response of RC frame buildings with infilled masonry walls in the inelastic stage.

● **6.6-95** Wakabayashi, M. *et al.*, **Hysteretic behavior of steel braces subjected to horizontal load due to earthquake,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3188–3194.

In this paper, discussed is the elastic-plastic hysteretic behavior of steel braces, which are important earthquake-resistant elements in steel structures. The results of experimental and theoretical investigations are presented. Based on a parametric analysis of these results, idealized post-buckling curves and hysteresis loops are formulated and proposed for design use.

6.6-96 Reinhorn, A., Rutenberg, A. and Gluck, J., **Dynamic torsional coupling in asymmetric building structures,** *Building and Environment,* 12, *4,* 1977, 251–261.

An approximate method is proposed for the dynamic analysis of torsionally coupled tall building structures by utilizing the properties of their uncoupled counterparts. An exact solution is first given for the particular case in which the lateral and torsional stiffness matrices are uncoupled by the same transformation. The method is then applied to a wider class of structures where this condition is only approximately satisfied by reducing the dynamic coupling problem to an approximate two degree-of-freedom system. Simple formulas and graphical representations of dynamic magnification of static eccentricity are given. Two numerical examples illustrate the use of the proposed method; the results are checked for accuracy and compared with seismic code provisions.

● **6.6-97** Masri, S. F., **Analytical and experimental studies of nonlinear system modeling and scaling: progress report May 1976 to November 1976,** *NUREG-0192-1,* Civil Engineering Dept., Univ. of Southern California, Los Angeles, Feb. 1977, 279.

This is the first report under a continuing research project, the objective of which is to gain a better understanding of the basic phenomena present in devices such as valves, pumps, etc., that are related to nuclear reactor safety. The results of a comprehensive experimental and analytical study of the behavior of a nonlinear dynamic system involving beam/stop impact are presented in this report. An exact closed-form analytical solution is derived for the steady-state motion of an equivalent lumped parameter system that is subjected to harmonic excitation. Experimental measurements with a mechanical model completely corroborate the theoretical predictions, particularly the existence of jump phenomena associated with multivalued branches of the frequency-response curve. An important conclusion of this study is that scale effects play a significant role in determining the response of actual (as opposed to mathematical) nonlinear dynamic systems. In spite of careful scaling, substantial differences are encountered in the dynamic response of such systems.

● **6.6-98** Wang, T. Y., Bertero, V. V. and Popov, E. P., **Hysteretic behavior of reinforced concrete framed walls,** *EERC 75-23,* Earthquake Engineering Research Center, Univ. of California, Berkeley, Dec. 1975, 367. (NTIS Accession No. PB 267 298)

Two identical three-story framed wall specimens, representing the lower portion of a ten-story framed wall building, were tested under monotonic and cyclic loading to study the behavior of the walls under seismic excitations. One-third-scale models of the specimens were used. The code-designed building consisted of ductile moment-resisting frames with two framed walls in the north-south direction and four framed walls in the east-west direction. Its floor system consisted of a flat reinforced concrete slab. To simulate the boundary condition of the prototype wall as well as to transfer uniformly the applied shear force through the whole width of the wall, a portion of the flat slab was cast with the wall specimen. Shear force, axial force, and bending moment were applied to simulate the effects of gravity loads and earthquake excitations on the prototype. After incipient failure, each specimen was repaired to study the effectiveness of the repairing technique.

Free vibration tests were carried out to determine the critical damping ratio and the frequency of vibration of each specimen before and after loading them to different levels of damage. The test data permitted comparison of: (1) the directly measured lateral displacements at different floor levels with the computed lateral displacement based on the measured flexural and shear deformation, (2) the external energy applied to the specimens with the internal energy dissipated by the specimens, and (3) the measured strength with the theoretical strength. Based on the mechanical behavior of the wall element, nonlinear dynamic analyses were carried out to study the response of the prototype building under different ground excitations. The main objectives of these analyses were to determine the distribution of shear and axial forces and bending moments in the different structural elements and to define the ductility demands at the critical regions of these elements.

Experimental and analytical results showed that walls of a wall-frame structural system could fail in shear when subjected to severe seismic ground motions. Depending on the plastic hinge rotation capacity of the critical regions of the frame elements, columns and beams, and on the dynamic characteristics of the ground excitations, wall failure could also lead to collapse of the entire building. Present code design methods for wall and wall-frame systems are assessed. Recommendations for designing the wall against shear failure and for improving present methods of designing dual bracing systems are also offered.

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6.7 Nondeterministic Dynamic Behavior of Nonlinear Structures

● 6.7-1 Suzuki, K., **Analysis of uncertainty in seismic response of secondary appendage system,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(b), Paper K 6/2, 12. (For a full bibliographic citation, see Abstract No. 1.2-5.)

This report involves uncertainty analysis of the maximum response properties of the secondary appendage system such as piping and other mechanical equipment in nuclear power plants and other industrial facilities. Analysis is performed by using a simplified coupled supporting-appendage model. Nineteen representative records of strong-motion earthquakes are introduced for this analysis. Attention is focused on the response characteristics of the appendage at the worst condition where the natural period of the supporting structure coincides with that of the appendage. Response properties are represented by the form of a proposed floor response amplification factor (FRAF). Uncertainty of the FRAF is investigated by calculating the expected value and coefficient of variance. It is concluded that the uncertainty in the response properties of the appendage can be considered to be the product of the uncertainty depending on the fluctuation of the response spectrum and depending on the characteristics of the FRAF. Therefore, when a certain design response spectrum is given, it is sufficient to investigate only the uncertainty in the appropriate FRAF.

● 6.7-2 Kobori, T., Minai, R. and Suzuki, Y., **Stochastic seismic response and reliability of hysteretic structures,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1083-1088.

The nonstationary, nonlinear response of hysteretic structural systems to stochastic earthquake-like excitations is studied in relation to the assessment of the structural reliability. An analytical procedure for the stochastic response analysis of hysteretic systems with strong nonlinearity is developed without recourse to the equivalent linearization technique. The ultimate aseismic safety of structures subjected to destructive earthquakes is discussed, mainly from a random low-cycle fatigue viewpoint. A numerical example is given for the typical bilinear hysteretic system, and the results are compared with those of the Monte Carlo simulation.

● 6.7-3 Gazetas, G. and Vanmarcke, E. H., **Stochastic prediction of seismic response of inelastic multidegree-of-freedom structures,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1071-1076.

This paper presents an approximate probabilistic dynamic analysis of the interstory displacement response to earthquake-like excitation of n-story, elastoplastic, shear structures. The starting point is the (elastic) response of an associated linear system having the initial properties of the elastoplastic system. The yielding activities of the stories are grouped into n "states," each characterized by the story which yields first. Hysteretic dissipation of energy and change of vibrational frequencies due to yielding are estimated in order to compute statistics of the response during each yielding state. The transition from the elastic to the elastoplastic response is determined by a set of first-crossing probabilities. Finally, combination of the n-states, conditioned by their probabilities of occurrence, leads to a statistical description of the maximum plastic displacement for each story. Results of the method for two and four degree-of-freedom structures are compared with the results of simulation studies.

● 6.7-4 Kameda, H. and Ang, A. H.-S., **Simulation of strong earthquake motions for inelastic structural response,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 483-488.

A method for simulating earthquake motion suitable for inelastic structural response is developed. Simulated accelerograms are generated so that deformation spectra and total hysteretic energy for elastoplastic systems will agree, in the mean, with corresponding recorded accelerograms. For this purpose, the equivalent duration and optimum spectral function of the simulated accelerograms are required; determination of these parameters is discussed.

● 6.7-5 Takemiya, H., **Low-cycle fatigue under random earthquake motions,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 592-598.

The safety of a randomly excited bilinear hysteretic system is discussed from the viewpoint of a low-cycle fatigue process as well as of first excursion over a specified barrier. The response under a quasi-nonstationary filtered shot noise is evaluated using a linearization technique. The reliability function is computed by assuming a Poisson type for level crossings and the Palmgren-Miner hypothesis for fatigue accumulation. Comparison is made between the above different types of failure to investigate the nonlinear effects.

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● **6.7-6** Goto, H. and Iemura, H., **Earthquake response characteristics of deteriorating hysteretic structures,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1106–1111.

A simple method for representing deteriorating hysteretic structures is developed. The equivalent stiffness and the energy-absorbing capacity of structures are assumed to degrade with decreasing residual strength derived from the theory of low-cycle fatigue. The calculated mean-square response of a model subjected to earthquake-type random excitation exhibits the typical dynamic failure process of structures. The proposed method can cover wide ranges of deteriorating hysteretic structures, suggesting that it is useful in practical earthquake response analysis.

● **6.7-7** Goto, H., Kameda, H. and Iemura, H., **Estimation of maximum hysteretic response to non-white random excitation,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1112–1117.

Random response of single degree-of-freedom bilinear hysteretic structures is analyzed to furnish useful information for aseismic structural design with emphasis on ductility requirements. Stationary and nonstationary random excitations with a single peak frequency are employed to clarify the effects of nonwhite frequency components of earthquake ground motion. The response of bilinear structures is predicted by means of equivalent linearization techniques. Using these results, a probability distribution of the maximum response is predicted through pure and envelope methods. Monte Carlo simulation performed on a digital computer verifies applicability of the analytical methods.

● **6.7-8** Tani, S. and Soda, S., **Vertical load effect on structural dynamics,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1028–1033.

The object of this investigation is to examine the P-Δ effect of gravity or the effect of the vertical component of the earthquake excitation in the dynamics of beam-yielding steel structures with a bilinear force-displacement relationship. In the analysis, a single degree-of-freedom shear model is used. The statistical mean square response of that system which is subjected to white noise excitation at the base in the horizontal and the vertical directions is expressed by a Volterra-type integral equation and is solved by applying the Laplace transformation theory. In order to examine the dynamic behavior of elastic-plastic structures, the authors developed an original method of equivalent linearization.

6.8 Soil-Structure Interaction

6.8-1 Hadjian, A. H., **Soil-structure interaction—An engineering evaluation,** *Nuclear Engineering and Design,* 38, 2, Aug. 1976, 267–272.

The two methods of analysis for soil-structure interaction, the impedance and the finite element methods, are reviewed with regard to their present capabilities to address the significant factors of the problem. The objective of the paper is to evaluate whether or not an adequate engineering solution to the problem is provided by either approach. Questions related to the reduction of seismic motions with depth, scattering of incident waves, the three-dimensionality of the real problem, soil damping, strain dependency of soil properties and the uncertainties associated with all of the above are discussed in detail. All conclusions are based on referenced material. It appears that, although both methods as presently practiced have not yet completely solved the problem, the impedance approach has come closer to addressing the more significant issues. Because of this finding, in addition to its simplicity and low cost, the impedance approach is the preferred engineering method for soil-structure interaction.

6.8-2 Costantino, C. J., Miller, C. A. and Lufrano, L. A., **Soil-structure interaction parameters from finite element analysis,** *Nuclear Engineering and Design,* 38, 2, Aug. 1976, 289–302.

A series of two-dimensional finite element computer runs were made to compute the frequency dependent soil-structure interaction coefficients. Variations in the element size, mesh dimensions, boundary conditions, and soil hysteretic damping ratio to determine their influence on the computed interaction coefficients were made. From the calculations, it has been determined that the primary requirement of the mesh is a transmitting boundary formulation. For low damping conditions, roller support boundary conditions must be placed exceedingly far from the structure to ensure convergence of the results to the analytic solution. In addition, with such boundary conditions, the addition of artificial hysteretic soil damping cannot be used to simulate radiation behavior of the continuum. A frequency dependent criteria is also presented to determine minimum size elements that must be used in any calculation.

6.8-3 Hall, Jr., J. R. and Kissenpfennig, J. F., **Special topics on soil-structure interaction,** *Nuclear Engineering and Design,* 38, 2, Aug. 1976, 273–287. (For an additional source, see Abstract No. 1.2-2.)

This paper discusses some of the important details of soil-structure interaction theory to provide a common means of comparison and to introduce some new approaches to simplify the solutions of deeply embedded

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foundations. A review of recent literature on soil-structure interaction reveals several important facts. First, conflicting comparisons between the lumped parameter and finite element solutions apparently exist. In some cases, both approaches give similar results while for others the results vary widely essentially because different models are used for comparison of both methods. Second, fundamental errors are made concerning finite element mesh size parameters and boundary conditions. Third, misunderstandings of soil mass and foundation or structural modal damping lead to gross errors in the lumped parameter approach. Finally, the limitations of the various approaches are not always understood. It is noted that both the finite element and lumped parameter approaches should yield similar results if they are appropriately used to solve the same problems. A summary of the advantages and limitations of both approaches is presented and discussed with a short presentation regarding the state of the art in the determination of soil stiffness and material damping characteristics.

Furthermore, the paper illustrates one type of analysis technique which uses a hybrid approach of both finite element results and lumped parameter solutions. Such an approach is developed to account more accurately for the influence of embedment. Results using both pure finite element solutions and lumped parameter models show that the influence of embedment can be accurately considered even for deeply embedded structures (depth to width ratio equal to 1.0). The key to the approach lies in establishing the coupling between horizontal and rocking modes of foundation vibration. Once the coupling parameter is accounted for, it is then possible to develop lumped parameter models that account for the variation of soil motions below the ground surface. Previous lumped parameter models have not accounted for these variations from the surface to the depth of the foundation.

Details of the lumped parameter approach for embedded foundations and an illustration with numerical examples are provided. Recommendations are then presented for a procedure for analyzing soil-structure interaction of deeply embedded foundations. The primary advantages are that (1) the model is more easily generated than a finite element model, (2) the model is less susceptible to modeling errors, such as mesh size, model size, and boundary influences, (3) parametric studies may be easily conducted since parameters such as damping may be more directly controlled, and (4) the computer costs for analysis are significantly reduced.

6.8-4 Hamilton, C. W. and Hadjian, A. H., **Probabilistic frequency variations of structure-soil systems**, *Nuclear Engineering and Design*, 38, 2, Aug. 1976, 303-322. (For an additional source, see Abstract No. 1.2-2.)

During earthquakes, structure-soil systems act as filters, greatly amplifying the response of equipment whose frequencies are at or near their natural frequencies. Thus, the estimation of these structure-soil system frequencies assumes significant importance both for safety and cost. Actual in-situ frequencies of structures differ from calculated frequencies due both to variations in mathematical modeling techniques and to variations of material properties. This paper studies the second source only. This variability is usually gauged by the "worst case" analyses technique in which extreme high- and low-parameter values are assumed and the associated frequencies are used as upper and lower bounds. This approach is not entirely satisfactory because it does not provide any indication of the probability of these limits being exceeded, of the distribution between these limits, or of the level of conservatism introduced into the design process. The present approach provides this additional information. The emphasis in this paper is both on developing the methodology and on the results obtained. It covers both the fixed-base structure and the effects of soil-structure interaction. Empirical data on concrete properties were obtained from previously published results. Much less is known about variability of soil properties, so the soil-structure interaction coefficients are assumed to be normally distributed. As data on the variations of soil properties become available, they can be readily incorporated via the methodology developed here.

For the soil-structure system, the key methodological problem is to obtain the probability distribution of eigenvalues of matrices with random variable elements. Since no analytic relation exists between the eigenvalue and the elements, a numerical procedure was designed. The approach consists in using selected fractiles and conditional cumulative distributions; it yields more exact results with less computational effort than alternate methods such as Monte Carlo simulations. Also, since the primary purpose here was to determine selected high and low fractiles of the frequencies, this approach was preferred to moment-estimating techniques.

For fixed-base structures, this study shows that the frequency variations are independent of the structural properties and are the same for all modes, both for continuous and lumped-mass systems. However, for structure-soil systems, the probability distributions differ considerably from mode to mode. Further, the assumed variability of the impedance coefficients may increase or decrease the overall frequency variation fractiles from those of the fixed-base structure. As expected, the soil variability impacts only on the interacted frequencies. These frequencies are easily identified from the conditional cumulative distributions and other relationships presented in the paper.

● **6.8-5** Fedyakova, S. N., **Special analysis of the vibrations of rigid buildings from recordings of the Petropavlovsk-Kamchatskii earthquake** (Spektralnyi analiz kolebanii zhestkikh zdanii po zapisyam zemletryasenii v Petropavlovske-Kamchatskom, in Russian), *Seismostoikoe stroitelstvo, 4,* 1977, 37–41.

The vibrations of three five-story precast panel buildings situated on soils of varying density in Petropavlovsk-Kamchatskii are investigated. The MINSK-32 computer was used to construct the Fourier spectra of vibrations. The effects of geological conditions on the soil-structure interaction during the earthquake were investigated.

● **6.8-6** Takada, S. and Nagao, S., **Dynamic frictional forces and efficiency of joint parts for aseismic strength of buried pipelines,** *Proceedings of U.S.-Japan Seminar on Earthquake Engineering Research with Emphasis on Lifeline Systems,* Japan Society for the Promotion of Earthquake Engineering, Tokyo, Nov. 1976, 211–222.

The dynamic behavior of underground pipelines which are allowed to slide during earthquakes is studied. First, model experiments were performed to determine the characteristics of nonlinear frictional forces between the pipe and surrounding ground. The analytical solution is also deduced to obtain the relative displacement between the pipe and the ground, making use of the results of the experiments. Second, numerical response analyses were carried out for models of weld-jointed steel pipelines and cast-iron pipelines with many mechanical joints. From the results of this study, the necessity of using flexible joints in pipelines to absorb the relative displacement between the pipelines and the surrounding ground is emphasized.

● **6.8-7** Medvedeva, E. S., **Effects of the depth of structure on seismic excitation** (Vliyanie zaglubleniya sooruzheniya na velichinu seismicheskogo vozdeistviya, in Russian), *Seismostoikoe stroitelstvo, 2,* 1977, 37–40.

The effects of the depth of a structure on vibration intensities during earthquakes are investigated. The interaction of the soil and the underground part of the structure is effected by the reflection and refraction of incoming seismic waves. Horizontal seismic excitation is transmitted both horizontally and vertically. The amplitude of seismic waves decreases with increasing distance from the surface. An approximate method of analysis is used. The formation of standing waves in layered soils due to multiple reflections is discussed.

● **6.8-8** Savinov, O. A. and Uzdin, A. M., **A method for calculating the effects of soil-structure interaction on seismic response** (Metod ucheta vzaimodeistviya sooruzheniya s osnovaniem v raschetakh na seismicheskie vozdeistviya, in Russian), *Seismostoikoe stroitelstvo, 1,* 1977, 3–9.

Soil-structure interaction during seismic vibrations is analyzed in a linear framework. The authors investigate the simplest problem of a single-mass oscillator on a semi-infinite beam. The method presented can be applied to more complex systems with several masses by separating vibrations of differing mode shapes.

● **6.8-9** Bycroft, G. N., **Soil-structure interaction at higher frequency factors,** *Earthquake Engineering and Structural Dynamics,* 5, *3,* July–Sept. 1977, 235–248.

Approximate solutions to the forced vibrations of a rigid circular plate attached to the surface of an elastic halfspace are presented for large values of the frequency factor. These results are important when solving soil-structure interaction problems when such problems involve high-frequency factors. This situation arises when high-frequency components of earthquakes are associated with a relatively rigid foundation of a large base area and located on a soft terrain. Similar situations occur in cases of blast loadings and impact and in the foundations of large high-speed machinery. These solutions are used to solve the problem of the motion of a rigid mass on an elastic halfspace subjected to steady state and transient horizontal accelerations. From these results, it is deduced that a large rigid mat foundation located on soft terrain significantly attenuates input accelerations and consequently may be useful as the foundation of large, massive, rigid structures such as a nuclear power station.

● **6.8-10** Wong, H. L., Luco, J. E. and Trifunac, M. D., **Contact stresses and ground motion generated by soil-structure interaction,** *Earthquake Engineering and Structural Dynamics,* 5, *1,* Jan.–Mar. 1977, 67–79.

A study has been made of the dynamic contact stresses that the foundation of a nine-story reinforced concrete building exerts on the soil during forced-vibration tests. The effects of the flexibility of the foundation on the contact stress distribution and on the force-displacement relationship for the foundation have been examined in an attempt at testing several simplifying assumptions commonly used in soil-structure interaction studies. Comparisons of calculated and observed ground displacements induced by soil-structure interaction in the immediate neighborhood of the building have also been presented.

● **6.8-11** Abdel-Ghaffar, A. M. and Trifunac, M. D., **Antiplane dynamic soil-bridge interaction for incident plane SH-waves,** *Earthquake Engineering and Structural Dynamics,* 5, *2,* Apr.–June 1977, 107–129.

The analysis of dynamic soil-bridge interaction has been performed in three steps. These are (1) the analysis of input motions; (2) the force-displacement relationships for the foundations; and (3) the dynamic analysis of the structure itself, i.e. the bridge.

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Based on the exact solution of the first two steps, the dynamic interaction of a simple two-dimensional bridge model erected on an elastic halfspace has been investigated for a single-span case. The two-dimensional model under study consists of an elastic shear girder bridge supported by two rigid abutments and rigid foundations which have a circular cross section and are welded to the halfspace. It has been shown that the dynamic interaction depends on the incidence angle of plane SH-waves, the ratio of the rigidity of the girder and the soil, the ratio of the girder mass to the mass of the rigid abutment-foundation system, and the span of the bridge. The dynamic response of the girder and the effect of the radiative damping in the halfspace on the interaction of the girder have been studied.

● **6.8-12** Audibert, J. M. E. and Nyman, K. J., **Soil restraint against horizontal motion of pipes,** *Journal of the Geotechnical Engineering Division, ASCE,* **103,** *GT10,* Proc. Paper 13303, Oct. 1977, 1119–1142.

The safe and economical design of buried pipes subjected to lateral motion requires an accurate knowledge of the subgrade reaction. Too often values of the coefficient of horizontal subgrade reaction reported in the literature are used without understanding the author's intentions and without thought as to whether or not the conditions in the particular application are compatible to those in the literature. A survey of the available literature discloses that the numerical values and formulations proposed were such as to underestimate the soil restraint against flexible culverts and laterally loaded piles. These values are inadequate for the safe design of buried pipes laterally displaced by settlement, thermal expansion, earthquake shaking or wave action. The results of a laboratory testing program show the complex soil-conduit interaction and the associated failure mechanisms. An analytical method is presented to determine the load-displacement curve for any size pipe embedded at any given depth. A moderate size in-situ test was performed, which confirmed that the results of the laboratory model tests could be successfully applied to in-situ conditions.

● **6.8-13** Valera, J. E. *et al.,* **Seismic soil-structure interaction effects at Humboldt Bay power plant,** *Journal of the Geotechnical Engineering Division, ASCE,* **103,** *GT10,* Proc. Paper 13306, Oct. 1977, 1143–1161.

The results of a study of the distribution of ground motions and structural response in the Humboldt Bay nuclear power plant during the Ferndale earthquake of June 7, 1975, are presented. Based on a knowledge of the motions recorded at the ground surface in the free-field, computations are made to determine the characteristics of the motions likely to develop at the base of the buried reactor caisson at a depth of 85 ft (26 m) below the ground surface and within the refueling building at the ground surface level. The computed motions are shown to be in reasonably good agreement with those recorded at these locations in the same earthquake.

● **6.8-14** Mizuno, N. *et al.,* **Seismic response analysis of nuclear reactor buildings under consideration of soil-structure interaction with torsional behaviour,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(a), Paper K 2/16, 14. (For a full bibliographic citation, see Abstract No. 1.2-5.)

This paper is a continuing report of a previous paper in which an explanation is presented of the experimental and analytical studies of the No. 1 reactor building (BWR) of the Hamaoka nuclear power station. The seismic response analysis is described in detail for estimating soil-structure interaction effects with torsional behavior.

The analytical method is shown for estimating the stiffness of the reactor building using the bending-shear and torsion theory for the thin-wall sections. The three-dimensional behavior of the structure can be more easily obtained by the proposed method. The dynamic soil-foundation coefficient for estimating the dissipation of vibrational energy on the ground is derived using H. Tajimi's theory which is based on a solution of the propagation of seismic waves caused by point excitation on the surface of the elastic halfspace medium. The above results give the vibrational impedances of the soil-foundation corresponding to the static soil coefficient, which is defined to be the excitation force in the frequency domain. The dynamic soil-foundation coefficient is approximated as the frequency transfer function of displacement. Complex damping is used to more suitably estimate the elastic structural damping effects of the structure. A regression analysis of many degrees-of-freedom is used to estimate the natural periods and equivalent viscous damping ratios directly from the forced vibration experimental results. The analytical results are shown to simulate and compare with the above-mentioned experimental results.

It is concluded that the radiation damping effects are remarkable in the reactor building and that the proposed seismic response analysis is reasonable for estimating soil-structure interaction when there is torsional structural behavior.

● **6.8-15** Abdel-Ghaffar, A. M., **Studies on the effect of differential motions of two foundations upon the response of the superstructure of a bridge,** *EERL 77-02,* Earthquake Engineering Research Lab., California Inst. of Technology, Pasadena, Jan. 1977, 87.

This report contains two studies that were made of the effects of differential motion of the foundations upon the response of the superstructure of a bridge. The first study deals with the dynamic response of a "long beam" model of a bridge to both steady-state and random excitations applied at the supports. The study has been simplified by considering a long shear beam, simply supported at two ends; this beam is subjected to two end excitations in the form of ground displacements. Harmonic excitations, differing in phase at the ends, were considered in the frequency domain by analyzing the steady-state vibrations and calculating the displacement amplitudes at specific points on the beam. The energy content of the system has been presented, and the correlation between the two end excitations has been considered. For the random excitations, the analysis has been made in the time domain; two different cases of random motions have been considered.

The second study develops a method to analyze the dynamic soil-bridge interaction of a simple two-dimensional bridge model erected on an elastic halfspace, with the input motion in the form of incident plane SH-waves. The bridge model consists of an elastic shear girder supported by two rigid abutments and rigid foundations which have a circular cross section and which are welded to the halfspace. Finally, the dynamic response of the bridge and the effect of the radiative damping in the halfspace on the interaction of the bridge are also studied.

● **6.8-16** Evison, R. J., **Rocking foundations,** *77-8,* Dept. of Civil Engineering, Univ. of Canterbury, Christchurch, New Zealand, Feb. 1977, 96.

This report concerns the effects on seismic structural response of permitting a structure to rock on its foundations. Experimental verification is sought for theories developed elsewhere describing the rocking phenomenon as applied to a rigid block on a rigid base. Experimental tests with a shaking table model confirmed much of the theory. Further tests using more flexible foundations revealed similar results. This suggests that sub-base conditions are not critical in analyzing a rocking situation except inasmuch as they control the natural rocking frequency of a system, and hence the magnitude of the structure's response. A design method for determining peak rocking displacements is presented based on the theory examined and on existing earthquake response spectra.

Computer methods for analyzing structures rocking under dynamic earthquake loading are investigated. Results indicate that an exact analysis is possible only if the timestep between integrations is exceptionally small. With realistic timesteps, the computational and truncation errors accumulate to unacceptable magnitudes for the rigidly based structure. On more flexible foundation materials, satisfactory results are obtained as the errors become less significant.

Experimentally observed high accelerations occurring on impact of a rocking structure with a rigid base render the analysis impractical and indicate for design purposes little benefit results from allowing the structure to rock. Both experiment and computer analyses indicate that worthwhile reductions in peak accelerations are present in a flexible foundation situation with associated acceptable levels of displacement.

● **6.8-17** Takemori, T. *et al.*, **Comparative aseismic response study of different analytical models of nuclear power plant,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(a), Paper K 2/2, 12. (For a full bibliographic citation, see Abstract No. 1.2-5.)

As nuclear power plant structures are generally rigid, the influence of rigidities of the foundation rock can be significant in aseismic design of nuclear power plants. The modeling of the foundation rock and the characteristics of the input acceleration are very important to structural response analysis. In the spring-mass model, the foundation rock is usually represented as springs calculated assuming the semi-infinite elastic solid. This spring-mass model is inadequate to show the magnification of acceleration at the surface soil level. This study consists of two major sections: one is a comparison of the magnification factor of input acceleration between the finite model and the spring-mass model; the other is evaluation of modified spring-mass models for the aseismic design of nuclear power plants.

The structural model used in this study is a PWR reactor containment building composed of the outer shield wall, the steel containment, and the internal structure. The rigidity of the foundation rock is represented by the shear wave velocity Poisson's ratio, and density. The magnification of bedrock acceleration at the structural foundation, main floors, and the free surface of foundation rock is calculated using axisymmetric finite elment analytical models with various rock rigidities. The outer shield wall and the steel containment are represented by shell elements, and the internal structure and foundation rock are represented by quadrilateral elements. Each nodal point has four degrees-of-freedom, in a shell element, and three in a quadrilateral element. The total degrees of this analytical model are large, so the eigenvalue is calculated by the subspace iterative method. The responses are calculated by the time history of acceleration and the response spectrum based on the modal superposition method. Comparative aseismic response results of the two models are presented and modified spring-mass models are proposed for aseismic design.

● **6.8-18** Arya, A. S. *et al.*, **Dynamic analysis of a reactor building on alluvial soil,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor*

● *See **Preface**, page v, for availability of publications marked with dot.*

Technology, Vol. K(a), Paper K 4/8, 12. (For a full bibliographic citation, see Abstract No. 1.2–5.)

The reactor building analyzed in this paper consists of a reinforced concrete internal framed structure enclosed in double containment shells of prestressed and reinforced concrete, all resting on a common massive raft. The external cylindrical shell is capped by a spherical dome while the internal shell carries a cellular grid slab. The building is partially buried underground. The soil consists of alluvial soil to 1000 m depth. The site lies in a moderate seismic zone. The paper presents the dynamic analysis of the building including soil-structure interaction.

The mathematical model consists of four parallel, suitably interconnected structures, namely, inner containment, outer containment, internal frame, and the calandria vault. Each of the parallel structures consists of lumped-mass beam elements. The soil below the raft and on the sides of the outer containment shell is represented by elastic springs in both horizontal and vertical directions. The various assumptions required to be made in developing the mathematical model are briefly discussed in the paper. The transfer matrix technique has been used to determine the frequencies and mode shapes. The deformations due to bending, shear, and the effect of rotary inertia have been included. Various alternatives of laterally interconnecting the internals and the shells have been examined and the best alternative from earthquake considerations has been obtained. In the study, the effects of internal structure flexibility and calandria vault flexibility on the whole building have been studied. The resulting base raft motion and the structural timewise response of all floors have been determined for the design-basis (safe-shutdown) earthquake by modal superposition.

It is found that the flexibilities of the internal structure and the calandria vault may be ignored in the determination of the first few periods and modes of vibration in the horizontal direction. The raft motion is also not much altered by considering the flexibility of the internal structure. The effects of base soil and side soil stiffnesses have been studied; it is found that the base soil considerably influences the dynamic characteristics of the structure and hence these properties should be properly evaluated at the site. It turns out that for shear wave velocity of soil more than 1200 m/sec, the structure may be assumed to be fixed at the base. Side soil has relatively less influence for embedments up to one-fourth the height of structure. It is found that the first mode is predominantly a rocking type, that the second mode is predominantly a translation type and that higher modes are structural modes. For vertical excitation, the derived raft motion and free-field motion have been found to be very similar. The first mode is mainly due to soil deformation, the second mode is the vibration of the cellular grid and the third mode is the structural mode. The first three modes are sufficient for computing the seismic forces and the first mode only is sufficient for calculating the vertical deflections of the structure.

● **6.8-19** Pajuhesh, J. and Hadjian, A. H., **Dynamic interaction of components, structure, and foundation of nuclear power facilities,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology*, Vol. K(a), Paper K 3/9, 8. (For a full bibliographic citation, see Abstract No. 1.2–5.)

A solution is formulated for the dynamic analysis of structures and components with different stiffness and damping characteristics, including the consideration of soil-structure interaction effects. Complex structures are often analyzed approximately, in particular with regard to damping. For example, the reactor and other equipment in nuclear power plant structures are often analyzed by assuming them uncoupled from the supporting structures. To achieve better accuracy, the coupled system is analyzed as a composite component-structure-soil system. Although derivation of mass and stiffness matrices for the component-structure-soil system is a simple problem, the determination of the damping characteristics of such a system is more complex. This emphasis on the proper evaluation of system damping is warranted on the grounds that, when resonance conditions occur, the response amplitude is governed to a significant degree by the system damping. The damping information is usually available for each substructure separately with its based fixed or devoid of rigid-body modes of motion. The rigid-body motions are often free of damping resistance but sometimes, such as in the case of soil-structure interaction, or in the case of aerodynamic resistance, are uniquely defined. The composite damping matrix for the complete structure is derived from the above-mentioned information. Thus, the damping matrix is first obtained for the free-free model of each substructure (the model containing the structural degrees-of-freedom together with rigid-body modes of motion), and then the submatrices for the free-free models are assembled to form the composite damping matrix in accordance with an assembly technique relating the substructure coordinates to the global coordinates of the composite structure.

To demonstrate the assembly technique, two examples are considered: (a) a steel structure sitting on a concrete stem and linked by a steel bridge to another concrete structure, and (b) an actual model of a nuclear power plant containment structure. In the latter example, the model consists of the containment structure, the internal structure, the pedestal, the shield wall, and the complete model of a boiling water reactor, including rod elements and hydrodynamic effects. All the subsystems are interconnected with each other and with the foundation. The stiffness matrix is also set up by the proposed method as well as an independent standard procedure to verify the validity of the proposed assembly technique.

● *See **Preface**, page v, for availability of publications marked with dot.*

● **6.8-20** Seed, H. B. and Lysmer, J., **Soil-structure interaction analysis by finite element methods—state-of-the-art,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(a), Paper K 2/1, 11. (For a full bibliographic citation, see Abstract No. 1.2-5.)

Analyses of soil-structure interaction effects during earthquakes for nuclear power plant structures are usually made by one of two methods—either by means of an idealized complete interaction analysis involving consideration of a compatible variation of motions in the structure and the adjacent soil, or by means of an inertial interaction analysis in which the motions in the adjacent soil are assumed to be the same at all points above the foundation depth. For surface structures, the distribution of free-field motions with depth in the underlying soils has no influence on the structural response and, thus, provided the analyses are made in accordance with good practice, good results may be obtained by either method of approach. For embedded structures, however, consideration of the variation of motions with depth is essential if adequate evaluations of soil and structural response are to be obtained without undue conservatism. The finite element analysis procedure is particularly well suited for evaluating the response of embedded structures since it can readily provide consideration of the variation of soil characteristics with depth, the different nonlinear deformation and energy absorbing capacities of the various soil strata, the variation of motions with depth in accordance with the general principles of engineering mechanics, the three-dimensional nature of the problem, and the effects of adjacent structures on each other.

At the present time, analyses incorporating the above considerations may be made for (1) an axisymmetric structure in an extensive soil deposit; (2) an axisymmetric structure where the extensive soil deposit is represented by a relatively small mesh equipped with transmitting boundaries; (3) a plane strain analysis of a structure in a soil deposit equipped with transmitting boundaries; (4) an approximate three-dimensional analysis of multiple structures where radiation damping effects are represented by transmitting and viscous boundaries; (5) a probabilistic three-dimensional analysis of multiple-structures using a relatively small mesh with transmitting and viscous boundaries. All of the above analyses are customarily made for vertically propagating waves. However consideration can now be given to nonvertically incident waves and to horizontally propagating waves in the analysis procedures. While the cost of such analyses has been high in the past, the development of increasingly efficient computer programs now seems to have overcome this limitation.

It is believed that these procedures provide a powerful tool for use in the design of nuclear plants. However, like all such procedures, they must be used with an intimate knowledge of the technical details which can be built into the various computer programs and which are necessary for adequate modeling of soil-structure systems. Used in this way and in conjunction with good engineering judgment, they can provide evaluations of response with a level of accuracy entirely adequate for engineering design, as evidenced by the recent completed study of the strong motions developed at the Humboldt Bay power plant in the Ferndale earthquake of June 1975.

● **6.8-21** Romo-Organista, M. P., Lysmer, J. and Seed, H. B., **Finite element random vibration method for soil-structure interaction analysis,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(a), Paper K 2/3, 12. (For a full bibliographic citation, see Abstract No. 1.2-5.)

Most current methods of soil-structure interaction are deterministic in nature and consider only a limited sample of possible time histories which approximately fit some given design response spectrum. However, earthquake motions are intrinsically random in nature, and it would be desirable to develop analytical methods which retain this randomness in both the definition of the design motion and the computed response. The authors present a method of this type. In this procedure, the seismic environment is defined directly in terms of the given design response spectrum. Response spectra cannot be used directly for random analysis; thus using extreme value theory, a new procedure has been developed for converting the design response spectrum into a design power spectrum. This procedure is reversible and can also be used to compute response spectra from power spectra. This process leads to a random ensemble of design response spectra, the distribution of which can be expressed in terms of confidence limits. Knowing the design power spectrum, the resulting output power spectra and their statistical distribution can be computed by a response analysis of the soil-structure system in the frequency domain. Due to the complexity of soil-structure systems, this is most conveniently done by the finite element method. Having obtained the power spectra for all motions in the system, these spectra can be used to determine other statistical information about the response such as maximum accelerations, stresses, bending moments, etc., all with appropriate confidence limits. This type of information is actually more useful for design than corresponding deterministic values.

The authors have developed a computer program, PLUSH, which can perform the above procedures. Results obtained by the new method are in excellent agreement with the results of corresponding deterministic analysis. Furthermore, the probabilistic results can be obtained at a fraction of the cost of deterministic results.

● **6.8-22** Melosh, R. J. and Buyukozturk, O., **Numerical analysis of soil-structure systems of unbounded geometry,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(a), Paper K 2/4, 9. (For a full bibliographic citation, see Abstract No. 1.2-5.)

When the behavior of points in an engineering system far removed from the boundaries is of concern, the system can be regarded as unbounded. This paper addresses finding equilibrium states of these systems using numerical analysis. It particularizes the discussion to the analysis of soil-structure systems, though the methodology is applicable to steady heat transfer fluid flow, and electromagnetic field problems as well. The usual approach to analysis of these systems is to circumscribe the region of behavioral concern by an artificial boundary and to analyze the truncated domain, imposing zero displacements along the boundary. To obtain a measure of the quality of this approximate model, a larger truncated region is analyzed. If the difference of responses between the two analyses is negligible, predictions are assumed to be accurate.

This paper offers a nonlinear iterative process which can determine the exact boundary conditions for the artificial boundary without recourse to analytical results. It describes the process and exhibits its effectiveness with illustrative problems. The process involves an adaptive evaluation of boundary conditions using a relaxation solution process. It establishes conditions along the artificial boundary such that decay of deformations within the truncated region is consistent with their vanishing at infinity. Iteration is performed by relaxing displacements at each boundary point in turn. A second analysis with a larger domain produces absolute measures of analysis error.

Illustrations include linear systems in one-, two-, and three-dimensional space: Winkler's beam-foundation problem and Boussinesq's. The numerical analysis uses finite element models and produces data for comparisons with the exact solutions. In each case, the exact responses are predicted, within the limitations of the finite element articulation, computer manipulation, and process errors. When accuracy of results in the usual approach is unacceptable, the same truncated region produces results of negligible error using the iteration. Analysis running times are less than four times those of the usual approach. Adding condensation logic could reduce this time factor to less than 1.50. The procedure is directly applicable to finite difference or boundary integral modeling. It can encompass certain types of nonlinear systems.

● **6.8-23** Khanna, J. K., Setlur, A. V. and Pathak, D. V., **Nonlinear seismic soil-structure interaction analysis of nuclear power plant structures,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(a), Paper K 2/5, 11. (For a full bibliographic citation, see Abstract No. 1.2-5.)

A heterogeneous and nonlinear soil medium and a detailed three-dimensional structure are synthesized to determine the seismic response to soil-structure systems. The approach is particularly attractive in a design office environment since it (a) leads to detailed structural response without the additional step of exciting the structure by interactive motion at the soil-structure interface; (b) uses existing public domain programs such as SAP IV, LUSH and FLUSH with marginal modifications; and (c) meets current regulatory requirements for soil-structure interaction analysis. Past methods differ from each other depending on the approach adopted for soil and structural representations and procedures for solving the governing differential equations. Advantages and limitations of these methods are reviewed.

In the current approach, the three-dimensional structure is represented by the dynamic characteristics of its fixed-base condition. This representation is ideal when structures are designed to be within the elastic range. An important criterion is the design of the nuclear power plant structures. Modal damping coefficients are varied to reflect the damping properties of different structural component materials. The detailed structural model is systematically reduced to reflect important dynamic behavior with simultaneous storing of intermediate information for retrieval of detailed structural response. The approach uses current concepts in the finite element idealization of the soil medium as developed by Lysmer et al. and as incorporated in the LUSH and FLUSH programs. Unlike other approaches that extract soil modes or predetermine soil impedance functions, the present approach retains the physical configuration of the soil medium throughout the analysis. Structural representation is put in the form of "eigenelements" having complex material properties. These eigenelements are coupled with physical finite element representation of the soil. The approach thus facilitates automated iterative usage of the frequency domain integration to provide a practical solution of a truly nonlinear problem. The resulting structural responses, which are of primary concern to designers, correctly account for the inertial, rigid body translational, and rocking effects. It has been demonstrated that neglecting the rocking effect in computing structural response produces erroneous results. Simple dynamic models of adjacent buildings used with the modal representation of the main building account for the effect of interbuilding interaction. Governing equations of motion and flow charts for procedural implementation are presented. The approach documented in an earlier report is the first step in consolidating structural response programs with current soil-structure interaction programs for obtaining seismic response of structural systems. Validity of the approach has been established with simple numerical experiments. A numerical example of the soil-structure interaction analysis of a reactor building is presented to show the efficiency and effectiveness of the new approach.

● *See **Preface**, page v, for availability of publications marked with dot.*

● 6.8-24 Kausel, E. *et al.*, **Dynamic analysis of embedded structures,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(a), Paper K 2/6, 10. (For a full bibliographic citation, see Abstract No. 1.2-5.)

The paper presents simplified rules to account for embedment and soil layering in the soil-structure interaction problem to be used in dynamic analyses. The relationship between the spring method and a direct solution (in which both soil and structure are modeled with finite elements and linear members) are first presented. It is shown that for consistency of the results with the two solution methods the spring method should be performed in the following three steps: (1) Determination of the motion of the massless foundation (having the same shape as the actual one) when subjected to the same input motion as the direct solution. For an embedded foundation, it will yield, in general, both translations and rotations. (2) Determination of the frequency-dependent subgrade stiffness for the relevant degrees-of-freedom. This step yields the so-called soil springs. (3) Computations of the response of the real structure supported on frequency-dependent soil springs and subjected at the base of these springs to the motion computed in step (1).

The first two steps require, in general, finite element methods, which would not make the procedure attractive. It is shown in the paper, however, that excellent approximations can be obtained, on the basis of one-dimensional wave propagation theory for the solution of step (1) and correction factors modifying for embedment the corresponding springs of a surface footing on a layered stratum for the solution of step (2). Use of these rules not only provides remarkable agreement with the results obtained from a full finite element analysis, but results in substantial savings of computer execution and storage requirements. This frees the engineer to perform extensive studies, varying the input properties over a wide range to account for uncertainties, in particular with respect to the soil properties.

● 6.8-25 Masao, T. *et al.*, **Earthquake response of nuclear reactor building deeply embedded in soil,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(a), Paper K 2/7, 12. (For a full bibliographic citation, see Abstract No. 1.2-5.)

Experimental and theoretical investigations have been performed on a model of a nuclear reactor building embedded in soil. The height of the concrete model is 3.75 m, the bottom cross section is 5 x 5 m, and the weight is 173 tons. The top of the model was excited by an eccentric mass vibration generator that can generate a maximum force of 3 tons. Earth pressures were measured at the bottom and at the side wall of the model, and displacements were also measured at the top of the model (6 components) and at the ground surface (18 components). These tests were conducted at a field site where embedded depth was changed at 5 steps which were 0, 20, 47, 73, 100% (against the height of the model). Using these experimental results and soil properties, the authors studied dynamic characteristics, including resonant frequency, radiation damping, vibrational mode, frequency response, and earth pressure distribution around the model at varying embedments. A lumped-mass model, a cylindrical elastic wave model, and finite element models (thin-layer elements) were used.

In conclusion, it was found that deeply embedded reactor buildings, especially over the center of gravity, have a great deal of radiation damping and a high eigenfrequency, and, that under earthquake motion, the building moves together with the soil. Even in the case of 100% embedment, the ratio of rocking motion in the response of the model is larger than the swaying motion.

● 6.8-26 Chopra, A. K. and Gutierrez, J. A., **Earthquake analysis of structures including structure-soil interaction by a substructure method,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(a), Paper K 2/8, 10. (For a full bibliographic citation, see Abstract No. 1.2-5.)

A general substructure method for analysis of response of nuclear power plant structures to earthquake ground motion, including the effects of structure-soil interaction, is summarized. The method is applicable to complex structures idealized as finite element systems and the soil region treated as either a continuum, for example as a viscoelastic halfspace, or idealized as a finite element system. The halfspace idealization permits reliable analysis for sites where essentially similar soils extend to large depths and there is no rigid boundary such as soil-rock interface. For sites where layers of soft soil are underlain by rock at shallow depth, finite element idealization of the soil region is appropriate; in this case, the direct and substructure methods would lead to equivalent results but the latter provides the better alternative. Treating the free field motion directly as the earthquake input in the substructure method eliminates the deconvolution calculations and the related assumption—regarding type and direction of earthquake waves—required in the direct method. The substructure method is computationally efficient because the two substructures—the structure and the soil region—are analyzed separately; and, more important, it permits taking advantage of the important feature that response to earthquake ground motion is essentially contained in the lower few natural modes of vibration of the structure on a fixed base.

For sites where essentially similar soils extend to large depths and there is no obvious rigid boundary such as a soil-rock interface, numerical results for earthquake response of a nuclear reactor structure are presented to demonstrate that the commonly used finite element method may lead to unacceptable errors; but the substructure method leads to reliable results.

● 6.8-27 Costes, D., **Monodimensional schematization of soil for seismic response analysis** (in French), *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(a), Paper K 2/11, 12. (For a full bibliographic citation, see Abstract No. 1.2-5.)

Soil-structure interaction is currently analyzed either by the use of soil springs and dampers derived by the theory of elastic continuum, or by the use of finite element schematization of a certain part of the soil. Limitations of the soil springs method are (1) exact evaluation of impedances cannot be obtained for embedded foundations, for material damping, and for several soil strata; (2) linear springs and dampers are only an approximation for exact impedances; and (3) nonlinear effects cannot be taken into account. Limitations of finite element methods are related to the difficulty of simulating nonreflective boundaries for the perturbations coming from the foundation, and to the great number of finite elements required in representative three-dimensional models.

A monodimensional model has been applied in a computer program which takes into account embedment and material damping, obtains three-dimensional effects with very few finite elements, and investigates the problem of nonreflective boundaries. The soil is schematized by embedded tangentially rigid hemispheric cups, subjected to horizontal relative translations and to relative rotations; the central rigid hemisphere may be considered as equivalent to an actual cylindrical embedded foundation. The efforts between the tangentially rigid cups are elastically computed, but nonlinear effects could be taken into account. Nonreflective boundaries are obtained by imposing to the last external cup the exact motion deduced from the motion of the preceding one, when radiation laws are applied, which involves use of delay loops. The program incorporates some possibilities of schematization for structures fixed to the hemispheric foundation. Any type of excitation may be applied, by the use of inertial forces on the differential masses. The program proceeds by a simple step-by-step dynamic method and is very economic. Conditions of independence of the soil response toward the geometric progression ratio of the cup radii and toward the total radial extension have been investigated. Use of 7 to 10 cups appears practically sufficient when no material damping exists, and less when there is damping. Validation of the model is related to the true deformations of the halfspace excited by a hemispheric foundation. In the absence of an analytic solution, even in the static case, only approximations may be presented. It is estimated that tangentially rigid cups are justified, with certain reductions of the actual elasticity coefficients and specific mass.

The program has been applied to a nuclear building and the results are compared to the ones obtained for a flat superficial foundation without damping. The good results obtained with the nonreflective boundaries are promising for utilization in true three-dimensional programs, because of the very reduced geometric extension which may be allowed.

● 6.8-28 Johnson, J. J., Wesley, D. A. and Almajan, I. T., **The effects of soil-structure interaction modeling techniques on in-structure response spectra,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(a), Paper K 2/12, 12. (For a full bibliographic citation, see Abstract No. 1.2-5.)

The seismic design of nuclear power plant components is normally based on in-structure response spectra generated from an overall plant analysis which accounts for soil-structure interaction. For preliminary design, it is desirable to consider a broad range of site conditions which requires an economical analysis of the soil-structure interaction phenomenon. The purpose of this paper is to compare the results obtained from a simplified approach utilizing stiffness-proportional composite damping with those obtained utilizing more refined representations of the soil-structure interaction phenomenon. Thereby, the limits of applicability of the simplified approach are determined.

The structure considered for this investigation consisted of the reactor containment building and prestressed concrete reactor vessel for an HTGR plant. A conventional lumped-mass dynamic model in three dimensions was used in the study. The horizontal and vertical responses, which are uncoupled due to the symmetry of the structure, were determined for horizontal and vertical excitation. Five different site conditions ranging from competent rock to a soft soil site were considered.

The simplified approach to the overall plant analysis utilized stiffness-proportional composite damping with a limited amount of soil damping consistent with U.S. NRC regulatory guidelines. Selected cases were also analyzed assuming a soil damping value approximating the theoretical value. The results from the simplified approach were compared to those determined by rigorously coupling the structure to a frequency-independent halfspace representation of the soil. Finally, equivalent modal damping ratios were found by matching the frequency response at a point within the coupled soil-structure system determined by solution of the coupled and uncoupled equations of motion. The basis for comparison of the aforementioned techniques was the response spectra at selected locations within the soil-structure system. Each of the five site conditions was analyzed and in-structure response spectra were generated. The response spectra were combined to form a design envelope which encompasses the entire range of site parameters. Both the design envelopes and the site-by-site results were compared. The results of this investigation led to separate conclusions concerning the applicability of the simplified approach to the determination of the soil-structure system depending on whether response was in the horizontal or vertical direction. The in-structure response spectra in the horizontal direction compared on a site-by-site basis and as a design envelope showed only minor deviations due to the analysis technique applied. However, the in-structure response spectra in the vertical direction as determined by the simplified approach were significantly higher than those found by utilizing a more refined representation of the soil-structure interaction phenomenon. Further, the response spectra from the simplified approach exhibited highly pronounced peaks, whereas the response spectra from more refined analyses were of a broad-band nature. In conclusion, a simplified approach to the computation of the in-structure response spectra for horizontal excitations was shown to be valid. However, its application to the computation of the vertical response is unnecessarily conservative.

● **6.8-29** Bernreuter, D. L., **Assessment of seismic wave effects on soil-structure interaction,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(a), Paper K 2/14, 9. (For a full bibliographic citation, see Abstract No. 1.2-5.)

One of the most common hypotheses made for soil-structure interaction analyses is that the earthquake input motion is identical at all points beneath the structure. Several papers have recently shown that this assumption may be overly conservative and that the effect of wave passage is extremely important. These studies typically employ a relatively simple model, namely, the base mat is represented by a rectangular rigid foundation resting on top of the soil and connected to the soil by a continuously distributed set of soil springs. The seismic input is applied at the base of the soil springs and is assumed to be traveling at a constant wave velocity across the site. It is possible to improve on the soil/structure model by use of finite element methods; however, little is known about how to model the input seismic energy and typically a simple traveling wave is used. In this paper, the authors examine the available data to determine: (i) the appropriate wave velocity to use, and (ii) if the currently available analytic models are adequate.

The choice of the appropriate apparent wave velocity to use in the analysis is not simple because in the near field of an earthquake a number of complex arrivals give rise to the strong motion. These arrivals would be associated with the high apparent wave velocities of the lower layers, whereas the surface wave arrivals would be associated with the much lower wave velocities of the near surface layers. Because strong-motion instruments are triggered at some threshold level, it is not possible to make reasonable estimates of the appropriate wave velocities to use from the available earthquake data. Considerable data exists from underground nuclear explosions. This data was analyzed and shows that the appropriate velocity to use relative to wave passage as observed by various surface accelerometers is much higher than the near surface values. This high apparent wave train velocity is observed out to at least 10 times the depth of energy release.

The adequacy of the currently proposed methods of analysis was qualitatively determined by comparing the overall response of structures to real traveling waves to that predicted qualitatively by the various models. Although few cases exist of strong motion recorded both in buildings and nearby in the free field, there are a number of cases where groups of buildings with different base mat areas have recorded data. By comparing buildings in the same general area with different base mat areas, the lack of free field motion is minimized. The adequacy of current theoretical models is obtained by contrasting the differences in foundation level spectra between structures of different base mat areas to the variation predicted by the currently available methods.

The results are mixed. If the apparent seismic wave velocity is taken to be that of the upper layers (approximately 2000 ft/sec), then the theory predicts considerable difference in response for buildings with different base mat areas. If, as suggested by the data from underground nuclear explosions, the apparent wave velocity is much higher than the upper layers, then the theory predicts only small changes in the spectra between buildings of different base mat areas. Actual comparisons of the foundation level spectra between buildings are also mixed. It appears that there are many complex effects which clearly cannot be lumped together in a simple averaging scheme.

● **6.8-30** Parker, J. V., Ahmed, K. M. and Ranshi, A. S., **The multiple structure-soil interaction problem,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(a), Paper K 2/17, 11. (For a full bibliographic citation, see Abstract No. 1.2-5.)

A nuclear power station generally consists of a complex of different buildings, each having separate foundation rafts and containing a wide range of equipment. In designing these structures against earthquake ground motions, dynamic interaction or coupling effects through the ground are frequently ignored, each raft being treated as isolated from any such effects. This coupling effect is investigated in this paper for a number of different but simple arrangements of nuclear power station structures. The arrangements considered include: (i) Two twin SGHWR nuclear islands on separate foundation rafts. Each raft is 70 m square and the response is calculated for a range of distance between rafts varying from 35 m to 105 m; (ii) A single SGHWR nuclear island plus a fuel handling block on a separate foundation raft.

The structures are idealized using beam elements and rigid elements, the foundation rafts being assumed to be rigid. The ground itself is idealized as elastic spring-viscous damper elements. The stiffness matrix for the elements representing the ground is obtained by inverting the flexibility matrix consistent with elastic halfspace solutions. This procedure is shown to give good agreement with the stiffness derived directly using a three-dimensional finite element idealization of the elastic halfspace. The damping matrix for the elements representing the ground is obtained by a congruent transformation of the stiffness matrix in which the transformation matrix is obtained simply by a consideration of the relation between radiation damping and stiffness for single foundation rafts. This relation is assumed to hold true where there is soil coupling between foundation rafts. However the importance of coupled damping in this interaction effect is illustrated by also considering cases in which the damping is uncoupled.

The time history dynamic responses are calculated using the June 1966 Parkfield 5, N5W, component as the free field ground motion and for both soft (shear wave velocity 500 m/sec) and hard (shear wave velocity 2000 m/sec) grounds. The interaction effects are illustrated by computing the floor response spectra at various locations in the structures and comparing these with the uncoupled floor response spectra. Results obtained so far indicate that for hard soils (shear wave velocity 2000 m/sec), the dynamic response of plant items is not appreciably affected by the coupling although a slight mutual reinforcement of the peaks in the floor response spectra (up to 15%) does occur. For soft soils however (shear wave velocity 500 m/sec), there is a significant change of the peaks in the floor response spectra (up to 50%) when the distance between foundation rafts is small. The results and conclusions of the study are considered to be applicable to any reactor system. The study offers guidelines for judging the need to include structure-soil-structure coupling in seismic response analysis.

● 6.8-31 Cameron, G. R., **Computer study of foundation influence on tower dynamics,** *ME-77-1,* Bonneville Power Admin., U.S. Dept. of the Interior, Jan. 1977, 7.

A computer study of the influence of the soil mass surrounding the foundation of a tower, on the dynamic properties of that tower, was carried out. A SAP IV finite element computer model was used. The model was placed on springs. It was found that the stiffness of the soil mass has a substantial effect on the dynamic characteristics of a tower. As the spring stiffness decreases, the fundamental frequency of vibration of the tower decreases and the elastic curve of tower deflection becomes a rigid body rotation about the footings. The fundamental frequency of vibration of a tower is directly related to the type of soil surrounding the tower foundation.

● 6.8-32 Roesset, J. M. and Ettouney, M. M., **Transmitting boundaries: a comparison,** *International Journal for Numerical and Analytical Methods in Geomechanics,* **1, 2,** Apr.-June 1977, 151-176.

The use of discrete models for the dynamic analysis of a continuum requires the existence of a finite domain with well-defined boundaries. When these boundaries do not exist naturally, but have to be artificially imposed, it may be necessary to apply appropriate conditions on forces or displacements at the boundary nodes to reproduce the physical behavior of the actual problem. In the solution of soil-structure interaction problems, these conditions are simulated through the use of transmitting boundaries. In this paper, several of these boundaries are evaluated comparing the results they produce in the amplification of seismic motions, the determination of foundation stiffness, and the structural response. The distance of the boundaries to the zone of interest, the level of excitation (influencing the amount of internal soil damping), the geometry of the problem (finite soil layer versus a halfspace), and the relative frequency of the structure with respect to the soil and the specified motion are all parameters which must be taken into account for this evaluation.

● 6.8-33 Yokota, H., Ichinose, K. and Yamahara, H., **Earthquake observations and analyses of an LPG tank and its foundation soft soil,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2429-2434.

Earthquake observations were used to investigate the dynamic behavior of a soft soil and an LPG tank. Much useful information concerning the dynamic behavior of the base layer, the surface layer, and the tank during earthquakes was obtained. Earthquake motions of the surface layer were computed employing the multireflection theory in which the soil was considered to have linear viscoelastic properties. Furthermore, the tank, foundation, piles, and soil were represented by a lumped-mass system, and the responses of the system were computed by applying observation records to the base layer. The computed results were compared with the observation records and were found to agree well.

● 6.8-34 Joh, O. and Ohno, K., **Vibration analysis of buildings with consideration for the in-plane deformation of floor slabs,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1223-1228.

The vibration characteristics of buildings with long and narrow plans are influenced by the in-plane deformation of floor slabs. Soil-structure interaction is analyzed by using two-dimensionally distributed multi-mass models in order to obtain the effective factors for in-plane deformation of slabs. Eigenvalues and response values of soil-structure systems are computed and discussed for cases in which a partial basement is laid at different positions in the plan of the building, the soil formation under the site has a dislocation or consists of duplicated bias layers, or the shape of the floor plan is complex. The relations between the maximum response and the direction of ground motion are also investigated.

● 6.8-35 Reddy, D. V., Moselhi, O. E. and Sheha, S. A. E., **Dynamic structure-medium interaction of underground nuclear reactor containments,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2594–2599.

The inherent structural, biological, and ecological benefits of underground siting are of considerable interest to the nuclear industry. Parametric studies of the structural characteristics of the four principal underground concepts: (a) cut-and-cover, (b) unlined cavity, (c) lined cavity, and (d) lined cavity with annular filling of soft material indicate (1) the horseshoe shape to be the best profile, (2) active rock bolting is superior to all other types of cavity reinforcement, (3) considerable decrease in liner membrane forces (by 80%) and stresses in the medium (by 10-15%) due to isolation, (4) significant reduction of stresses in the structure and the medium by proper combination of the density and elasticity of the backfill in the cut-and-cover concept. The work has a number of spin-off applications outside the nuclear industry.

● 6.8-36 Arya, A. S. *et al.,* **Seismic analysis of reactor building of an atomic power plant on alluvial soil,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2673–2679.

This paper describes the determination of base-raft motion of a reactor building consisting of three substructures (i) outer containment, (ii) inner containment, and (iii) internal structures. The building is modeled as a multimass branched system. The soil on the sides and at the base is replaced by linear springs. The representation of the internal structural system as a single mass at the end of the rigid link is considered adequate for purposes of determining motion at the raft level. A parametric study is carried out in which shear wave velocity, modulus of subgrade reaction, weight of the internal structure, and height of the internal structure above the raft are varied over a practical range to determine their influence on the earthquake response.

● 6.8-37 Umemura, H. and Tanaka, H., **Elastic-plastic dynamic analysis of reactor buildings,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2633–2638.

A computer program for nonlinear dynamic analysis of reactor buildings is developed. The nonlinear moment-rotation and shear stress-shear deformation characteristics of walls used in the analysis are based upon the experiments of model structures. The geometrical nonlinearity of an equivalent soil spring for rocking due to foundation separation is also considered. The damping matrix in the program is formulated to accommodate the different characteristics of soil and building damping. A mathematical formulation and the results of an analytical investigation are presented.

● 6.8-38 Kunar, R. R. and Beresford, P. J., **Effects of site conditions on floor response spectra,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2600–2608.

This paper presents a numerical study of the variation of horizontal floor response of a nuclear power plant for different site conditions. Four site configurations are considered in this study: a flexible bedrock foundation, a thin soil layer on flexible bedrock, a soil layer of medium thickness on flexible bedrock, and an infinite soil layer. For each of these configurations, the shear wave velocities and damping ratios are varied within a range determined from a statistical evaluation of a large number of data logs for nuclear power plant sites. A finite element linear elastic analysis is used to evaluate the floor response spectra for each model subjected to an artificially generated time history input that matches the USNRC spectrum at foundation level. Efficient use is made of viscous dampers for modeling nonreflecting boundaries in the finite element models. The results presented in this paper may represent a useful guide in early design decisions or feasibility studies for nuclear power plants.

● 6.8-39 Arya, A. S. and Paul, D. K., **Earthquake response of tall chimneys,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1247–1259.

The seismic response of four tall chimneys, 120 m to 180 m in height, has been evaluated, taking into account soil-structure interaction. The earthquake force is in the form of average acceleration response spectra. The soil stiffness is varied, corresponding to shear wave velocities of 150 m/sec to 1200 m/sec. Rock base is also considered. The distribution of shear forces and bending moments along the height of the chimneys has been studied and shows considerable departure from that recommended by Housner and adopted in Indian Standards IS:1893. Alternative distributions are proposed.

● *See **Preface,** page v, for availability of publications marked with dot.*

● 6.8-40 Yamada, Y., Takemiya, H. and Kawano, K., **Seismic response analysis of long-span suspension bridge tower and pier system,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1280–1287.

This paper reports the dynamic response characteristics of the tower and pier system of long-span suspension bridges. Particular emphasis is placed upon the foundation effect on the behavior of the superstructure. A frequency-independent model is developed for the foundation reactions which are originally of a frequency-dependent nature. From the practical design point of view, the equivalent viscous damping factor for the foundation is also discussed.

● 6.8-41 Sotirov, P., **Determination of damping of real structures,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1325–1329.

Hysteretic damping of a soil-structure system is considered. For evaluation of the damping of each story of a building, the components are presented as a system of elements with complex stiffness.

● 6.8-42 Kobayashi, H. and Sugiyama, N., **Viscous damping of structures related to foundation conditions,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2803–2809.

Ground conditions affect the viscous damping characteristics of structures. In this paper, observed data of the viscous damping of tall buildings in Tokyo are examined. These data include the effects of ground conditions, i.e., the relationship of the fundamental and higher modes of vibration. The viscous damping mechanism is also discussed. Based on the results of this study, the authors propose a damping mechanism for a lumped mass system with a rocking foundation.

● 6.8-43 Arya, A. S. and Thakkar, S. K., **Interaction effect in gravity dam with earth backing,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2519–2525.

The paper describes the method of analysis and earthquake response of a typical masonry dam with earth backing on its downstream face. The analysis shows that under a probable maximum earthquake about 20% of the dam below the top of the earth backing could lose contact during vibrations. The model tests on a shaking table indicate separation at a depth comparable with the calculated depth. It is concluded that, though the earth backing has the effect of increasing damping, it does not prevent the cracking of the dam to any appreciable extent.

● 6.8-44 Chapiro, G. A. and Ashkinadze, G. N., **On the non-linear deformations base of earthquake-proof buildings under oscillations,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2892–2898.

It is assumed that compressive deformations of the ground are elastic-plastic and that the ground cannot act under tension. Based on these assumptions, a nonlinear analytical model of a large-panel building base under rocking oscillations was developed. Good corroboration of the model was obtained from vibration tests of a 1/4-scale model of a 10-story building erected on a ground base. Principal dynamic properties and seismic response of buildings on the nonlinear base are dealt with in this paper.

● 6.8-45 Takada, S., **Earthquake resistant design of underground pipelines,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3376–3381.

The dynamic behavior of underground pipelines allowed to slide during earthquakes is studied. This sliding is resisted by friction between the pipe and the surrounding ground. Model experiments were performed to determine the characteristics of nonlinear frictional forces. The analytical solution is also deduced to obtain the relative displacement between the pipe and the ground, using the results of the experiments. The necessity of flexible joints in pipelines which absorb relative displacements is emphasized.

● 6.8-46 Chandra, B. and Reddy, S. R., **Seismic behaviour of framed foundations for turbo-generators,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1621–1630.

A turbogenerator foundation usually consists of a heavy base slab and large supporting columns to carry the top slab on which the machine and other equipment rests. This paper examines soil-structure interaction for such a system. It also explores the aseismic design of such systems and presents results of a parametric study describing seismic behavior.

● 6.8-47 Kawamura, S., Umemura, H. and Osawa, Y., **Earthquake motion measurement of a pile-supported building on reclaimed ground,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1563–1569.

To determine the interaction effects between a pile-supported structure and its surrounding soft subsoil, earthquake motion measurement in and around a seven-story, reinforced concrete apartment house has been conducted.

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Although the earthquake motions measured were not intense, amplitude and period characteristics could be clarified. Theoretical analyses were also done in the elastic range using lumped mass models, and results were compared with observations. The appropriate volume of additive soil mass and modal damping ratios of soil and structure could be estimated.

● 6.8-48 Rashidov, T. R. *et al.*, **Investigations of earthquake resistance of constructions interacting with soil,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1658–1653.

The results of theoretical and experimental studies of dynamic stress deformations of underground structures are given. The validity of accepted dynamic theory regarding underground structures of different kinds is verified. Experimental research of the physical nature of soil-structure interaction under the effect of static and dynamic loads is presented.

● 6.8-49 Sugimura, Y., **Earthquake observation and dynamic analysis of a building supported on long piles,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1570–1575.

Earthquake records observed in and around a pile-supported 14-story building are discussed. The soil-pile-structure system and soil-pile interaction are analyzed and compared. Results indicate that the soil and piles behave almost identically except in the vicinity of the pile cap, leading to the conclusion that the behavior of the soil controls that of the piles. The relatively simple analytical model examined simulates well the actual time-domain and frequency-domain conditions.

● 6.8-50 Westermo, B. D. and Wong, H. L., **On the fundamental differences of three basic soil-structure interaction models,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1455–1460.

The theoretical modeling of a flexible soil-foundation system undergoing dynamic interaction with a superstructure is often represented by a rigid base placed on a soil medium which resembles either an infinite halfspace, a layered stratum, or a soil medium with finite size. Due to wave interferences caused by the geometrical boundaries, these models possess some marked differences in their dynamic behavior. In this paper, the differences between these models are discussed on the basis of exact solutions for two-dimensional soil-structure interaction problems. The results indicate that the halfspace model induces "radiation damping" on the structural response because of geometrical spreading of waves. In the other extreme, the finite "box-like" soil medium may trap all or a significant portion of the wave energy within, thus creating unrealistic resonant conditions. For the case of an infinite horizontal layer, some resonant modes occur even though radiation damping is present.

● 6.8-51 Osawa, Y., Kitagawa, Y. and Irie, Y., **Evaluation of various parameters on response analysis of earthquake motions including soil-building system,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1461–1466.

The influence of five dimensionless parameters on interaction effects were quantitatively examined using a soil-structure interaction model. In order to evaluate these parameters, an earthquake observation system for an existing simple model of a building and its surrounding subsoil has been carried out. Observational data have been recorded and analyzed using results of some auxiliary experimental and analytical studies. It is pointed out that the shear wave velocity in soil layers plays an important role in determining the suitable model of a soil-structure system.

● 6.8-52 Herrera, I. and Bielak, J., **Soil-structure interaction as a diffraction problem,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1467–1472.

A mathematical formulation is presented for the problem of dynamic soil-structure interaction considering linear behavior of the soil material and arbitrary nonlinear structural properties. Using the unperturbed soil motion as a point of departure, the problem is formulated in a fashion similar to that for diffraction. Conditions for traction and displacement jumps that define a minimal set of data required to determine the resulting motion of the structure are established.

● 6.8-53 Petrovski, J. and Jurukovski, D., **Influence of soil-structure interaction on dynamic response of structures,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1631–1638.

Existing methods for analysis of dynamic response of structures, assuming a fixed structure in the soil media, consider soil as a rigid nondeformable media. This assumption is widely adopted since it simplifies mathematical solution of the problem, but it has no physical justification. Using experimental results from dynamic tests of embedded foundations and forced vibration studies of five- and nine-story full-scale buildings for the same soil conditions, the soil-structure interaction parameters of the buildings were determined. Comparing dynamic responses of the fixed and flexible base models, an increase in base shear of an order of 20 to 29 and 20 to 50 percent for close and distant earthquakes respectively was obtained.

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● **6.8-54** Hamada, M., Akimoto, T. and Izumi, H., **Dynamic stress of a submerged tunnel during earthquakes,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1509–1515.

Earthquake observations were carried out to clarify the dynamic behavior of a submerged tunnel and the interaction between the tunnel and the ground during earthquakes. The dynamic strain of the tunnel was computed using a new mathematical model. The computed bending strain was in good agreement with the observed strain.

● **6.8-55** Tajimi, H., Minowa, C. and Shimomura, Y., **Dynamic response of a large-scale shaking table foundation and its surrounding ground,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1516–1521.

The dynamic response of a large-scale shaking table foundation and its surrounding ground, when the loaded table is driven horizontally and vertically in sinusoidal motions, is described. Measured earthquake response is included and measured data are compared with the theoretical prediction.

● **6.8-56** Wight, L. H., **Soil-structure interaction in nuclear power plants: a comparison of methods,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1549–1554.

An extensive parametric survey to analyze the differences between two methods of calculating soil-structure interaction was performed. One method involves discretizing the soil-structure system and solving for the complete response with the LUSH computer code. The other method solves for the lumped mass structural response with Whitman soil springs. Twelve soil-structure interaction problems are solved by each of these methods. Representative results are presented and discussed.

● **6.8-57** Hadjian, A. H. and Luco, J. E., **On the importance of layering on the impedance functions,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1675–1681.

The effects of layering on the vertical, horizontal, and rocking impedance functions for a rigid circular foundation are examined. Two basic soil models corresponding to a single layer and a double layer on rock are studied in detail. By varying the geometrical and physical properties of the layers, a number of case studies are generated. The results obtained are presented graphically and their significance discussed.

● **6.8-58** Novak, M., **Soil-pile interaction,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1555–1562.

Dynamic stiffness and damping of a soil-pile system results from interaction between the pile and the soil. This interaction is theoretically investigated using the assumptions of linear elasticity or viscoelasticity and small displacements. All the vibration modes, including torsion and floating piles, are analyzed. Impedance functions of the pile cap are established and a comprehensive parametric study is conducted. The theory is compared with experiments.

● **6.8-59** Gutierrez, J. A. and Chopra, A. K., **Evaluation of methods for earthquake analysis of structure-soil interaction,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1449–1454.

Three methods for analysis of earthquake response of structures, including structure-soil interaction, are evaluated for their effectiveness and applicability in handling various types of structures and soil conditions encountered in practice. Two of these—a simple substructure method and a direct finite element method—have been employed in analysis of practical problems for some time, whereas the third, a general substructure method, has been developed only recently.

● **6.8-60** Abdel-Ghaffar, A. M. and Trifunac, M. D., **Antiplane dynamic soil-bridge interaction for incident plane SH-waves,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1609–1614.

A study is made of the dynamic interaction between the soil and a simple bridge erected on an elastic halfspace. The two-dimensional bridge model consists of a series of elastic shear beams supported by rigid end abutments, intermediate rigid piers, and rigid foundations. The nature of the dynamic interaction depends on (1) the angle of incident plane SH-waves, (2) the ratio of the rigidity of the girders to the rigidity of the soil, (3) the ratio of the girders' mass to the masses of the abutment-foundation and pier-foundation systems, and (4) the relative ratios of the different span lengths.

● **6.8-61** Chen, M. and Penzien, J., **Nonlinear soil-structure interaction of skew highway bridges,** *UCB/EERC-77/24,* Earthquake Engineering Research Center, Univ. of California, Berkeley, Aug. 1977, 127. (NTIS Accession No. PB 276 176)

This report is one in a series to result from the study "An Investigation of the Effectiveness of Existing Bridge Design Methodology in Providing Adequate Structural Resistance to Seismic Disturbances," sponsored by the U.S.

Dept. of Transportation, Federal Highway Admin. Descriptions are given of the analytical investigations of the seismic response of skew highway bridges where soil-structure interaction effects are important.

Four different mathematical model elements are incorporated into the three-dimensional computer program which possesses the capability of performing linear or nonlinear time-history dynamic response analysis. Solid finite element modeling is used for the backfill soils and the abutment walls. The bridge deck, pier columns, and pier caps are modeled using prismatic beam elements. A frictional element is used to model the discontinuous behavior at the interfaces of the backfill soils and abutments. Boundary elements provide foundation flexibility at the base of columns supported on either piles or spread footings. In the nonlinear mathematical model, the effects of separation, impact, and slippage at the interfaces between the abutment walls and the backfill soils are taken into consideration. Computational efficiency is achieved through the use of mathematical techniques including matrix reduction procedures, iteration procedures, and variable time steps. A number of analytical solutions are carried out considering a skewed three-span bridge with backfill soils. Different mathematical models are used to study the parameters which may influence the seismic response of the bridge. Finally, conclusions are deduced from the analytical results.

● **6.8-62** Liou, D. D.-N. and Penzien, J., **Seismic analysis of an offshore structure supported on pile foundations,** *UCB/EERC-77/25,* Earthquake Engineering Research Center, Univ. of California, Berkeley, Nov. 1977, 111. (NTIS Accession No. PB 283 180)

This report presents an analytical study of the seismic response characteristics of an offshore structure supported on pile foundations. To allow the basic modeling of the structure-foundation system, a simple mathematical model of a pile foundation based on the three-dimensional theory of elasticity is developed. The earthquake surface ground motion is prescribed in the time domain, the solution of the system is carried out in the frequency domain, and the desired response quantities are transformed back to the time domain. Foundation-structure interaction effects are examined by comparing response quantities obtained for models with and without foundation flexibility. The interaction effects are found to be quite significant.

● **6.8-63** Valera, J. E. *et al.*, **Soil-structure interaction effects at the Humboldt Bay power plant in the Ferndale earthquake of June 7, 1975,** *UCB/EERC-77/02,* Earthquake Engineering Research Center, Univ. of California, Berkeley, Jan. 1977, 49. (NTIS Accession No. PB 265 795)

This report presents the results of a study of the distribution of ground motions and structural response in the Humboldt Bay nuclear power station during the Ferndale earthquake of June 7, 1975. Based on a knowledge of the motions developed at the ground surface in the free field, computations are made using an idealized complete interaction procedure based on finite element analysis to determine the characteristics of the motions likely to develop at the base of the refueling building at a depth of 85 ft below the ground surface, within the refueling building at a depth of 85 ft below the ground surface, and within the refueling building at the ground surface level. The computed motions are shown to be in reasonably good agreement with those recorded at these locations in the same earthquake. In addition, the recorded motions are compared with those computed by an analysis procedure which generally meets existing requirements of the Nuclear Regulatory Commission, and it is shown that the regulatory requirements lead to an adequate but not excessively conservative margin of safety based on the motions recorded in this event.

● **6.8-64** Bielak, J. and Palencia, V. J., **Dynamic behavior of structures with pile-supported foundations,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1576–1582.

The steady-state response of a simple pile-supported system is analyzed in order to assess the effects of the various parameters involved in soil-pile-structure interaction. Numerical results indicate that for linear systems compliance of point-bearing piles is negligible for tall buildings but may be significant for medium-tall and broad, lowrise structures. Interaction reduces the fundamental frequency of the system, but the primary effect is to reduce the peak response. This effect becomes more pronounced as soil deformability increases.

● **6.8-65** Ciongradi, I. and Ungureanu, N., **A dynamic model for the soil-foundation-structure interaction in the earthquake analysis of framed structures,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1602–1608.

An analysis referring to the aseismic code calculation of structures, based on conventional seismic spectra and the dynamic characteristics of the systems, is presented. The paper corrects this calculation to determine the effect of the interaction between the structure, substructure, and soil. A means of determining the dynamic characteristics of the structural system is presented which takes into account the static deformation of the structure, substructure, and soil. The interaction effect concerning dynamic characteristics of systems with or without damping is discussed with application to framed structures.

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● **6.8-66** Aydinoglu, M. N. and Cakiroglu, A., **Dynamic interaction between soil and a group of buildings,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1596–1601.

An approximate mathematical model is presented to analyze soil-structure systems comprised of one or more superstructures. The soil medium is idealized as a two-dimensional linear-elastic single layer and is discretized by means of nodal points on the surface. This model is capable of analyzing the ground compliance of multi-foundation systems. For the cases of a single and two identical in-phase rigid foundations, the ground stiffness matrices are obtained. Through these matrices, the first mode behavior of the soil-structure systems, with a single and two identical buildings, is analyzed. Numerical results for the two buildings are compared with the soil–single building system.

6.8-67 Dong, R. G., **Seismic loads in modularized and unmodularized large pools located on hard or intermediate hard sites,** *Nuclear Engineering and Design,* **44,** *3,* Dec. 1977, 397–406.

To augment the present capacity of pools for storing spent nuclear fuel elements, pools larger than those in current use are being planned. These pools may or may not be modularized into cells. Because of the large size of the pools, seismic loads are of significant interest. In particular, the effects of modularization and site hardness are of concern. The study presented in this paper reveals that modularization is generally unfavorable, because it creates the option of leaving one or more cells empty which in turn results in higher structural loads. The wall which separates a filled cell from an empty cell, or the wall which bears against earth on one side and faces an empty cell on the other, becomes very highly stressed. For the particular pool geometries examined, a hard site is generally preferred over an intermediate hard site in terms of structural loads.

● **6.8-68** Weidlinger, P., **Behaviour of underground lifelines in seismic environment,** Weidlinger Assoc., New York, July 1977, 22.

Formulation of a procedure for the analysis and design of underground lifelines in a seismic environment is considered. Current engineering seismology procedures are not sufficient for this purpose. The detailed definition of the displacement field as a result of seismic motion needs to be extended to include spatial and temporal variations in a broader frequency range. The displacement field may interact weakly or strongly with a buried pipeline, depending on the pipe's dynamic characteristics as modified by the surrounding soil. The effect of this interaction is presented in an "interference spectrum" which gives the peak response of a damped oscillator, subject to simultaneous excitation at two spatially separated points. Spectral amplitudes are used to determine the response of the system, in terms of a damage matrix which quantifies the failure parameters of a system consisting of various types of pipes, joints, and other parts.

For purposes of risk analysis, optimization, and cost-benefit determination for existing or planned systems covering large areas, a statistical method is developed which provides the expected value of free-field gradients, as affected by the subsurface and geology. These gradients are used to estimate the probable performance of the system during an earthquake defined by a power spectral density function.

● **6.8-69** Unemori, A. L. and Whitman, R. V., **Nonlinear inelastic dynamic analysis with soil-flexibility in rocking,** *R76-13, Seismic Design Decision Analysis Report No. 25,* Dept. of Civil Engineering, Massachusetts Inst. of Technology, Cambridge, Feb. 1976, 219.

Conventional structural analyses of the dynamic response of buildings subjected to earthquake ground motions assume that the foundation behaves as a rigid mass. While this assumption may lead to accurate results for conventional structures founded on rock and other competent soils, its validity for structures founded on clays may be questionable. This study compares the dynamic response of a building on a rigid foundation designed for earthquakes to the dynamic response of the same building on a nonlinear inelastic flexible foundation. Specifically, a 17-story reinforced concrete shear wall apartment building was placed on each one of two limiting cases of clays. Foundation systems were designed for each soil profile according to existing codes, and moment-rotation relationships for each foundation system were generated. An existing dynamic response computer program was modified to incorporate the predicted moment-rotation characteristics of the foundations. Response of the building on both flexible foundation systems was compared to the response of that building on a rigid foundation, and significant differences identified.

● **6.8-70** Zaslawsky, M., **The comparison of methods of solving the earthquake response of structures with foundation flexibility,** *UCRL-51986,* Lawrence Livermore Lab., Univ. of California, Livermore, Jan. 13, 1976, 61.

● **6.8-71** Fujiwara, T., **Seismic motions of very soft ground,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 395–401.

One of the characteristics of ground shaking which has a significant effect on structures built on soft ground is large values of maximum and relative displacements. The author's objective is to develop a means of preventing

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trains running at high speeds from derailing or overturning during an earthquake. When a train passes a bridge built over soft ground, and the deformation of a track caused by an earthquake exceeds a certain limit, the train will probably be derailed, even though the bridge itself may not be damaged. Frequently, such deformation is caused by soil-structure responses to seismic stress. Observation of seismic motions with long-period wave components is conducted in soft ground by the array method. The results clarify the nature of ground shaking which causes relative displacement. The authors find that differences of phase and amplitude between waves at adjacent points is affected by the kind of waves, the path of waves, and the inclination of the boundary between the surface soft layers and the hard underlying base.

● 6.8-72 Hamada, M. and Sato, S., **Behavior of underground tank during earthquakes,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1503–1508.

The seismic response of an underground tank constructed on reclaimed land was investigated. Based on the observation, a method of seismic response analysis for this type of tank is proposed. The method uses a new mathematical model in which the tank is assumed to be a cylindrical structure on an elastic foundation. The strain obtained by this method is in good agreement with the strain observed during the earthquake.

● 6.8-73 Khachian, E. E., **A study of seismic influences on structures considering their length and height,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 2058–2063.

A typical seismic analysis assumes that all parts of a structural foundation are subject to the same displacements and accelerations simultaneously and that seismic disturbances propagate instantaneously along the structure's vertical dimension. For small structures this assumption may be acceptable; however, for long and tall structures, the assumption distorts the true dynamic behavior. The report deals with some problems of introducing the length of structures and noninstantaneous vertical propagation of seismic disturbance in design analysis.

● 6.8-74 Medvedeva, S. V., **The influence of the overground and underground parts of construction heights correlation on the intensity of seismic effect,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1654–1657.

An approximate solution is obtained for the problem of the influence of the underground parts of structures (basements, etc.) on the oscillation intensity of the entire structure. It is pointed out that the interaction of aboveground and underground parts of a structure depends upon the reflection and refraction of approaching seismic waves. The fact that horizontal seismic action is transferred through horizontal and vertical surfaces is taken into account. It is found that, with increased distance from the earth's surface, seismic wave amplitudes decrease.

● 6.8-75 Dungar, R. and Clarkson, J. A., **The effects of high intensity earthquakes on the stability of the Chira Piura spillway, Peru - a nonlinear foundation/structure analysis,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1391–1392.

● 6.8-76 Barkov, Y. V. and Glina, Y. V., **The research into the earthquake-proof capacity of in-situ concrete frameless residential buildings by vibro-tests of the large-size reinforced concrete model,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2881–2888.

The objectives, methods, and results of static and dynamic tests of a 10-story model of a frameless building, built from in-situ concrete on a natural soil foundation, are discussed. A brief description of the model and methods of its erection is given. The results of the model design, including the actual physical and mechanical characteristics of structural materials and yielding of the soil foundation, are shown. The joint behavior of piers and lintels, as well as the interaction between the model and its soil foundation under variously intense horizontal inertia loads applied by means of vibrators, is analyzed.

● 6.8-77 Minami, J. K. and Sakurai, J., **Earthquake response spectra for soil-foundation-building systems,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1473–1483.

The seismic response of buildings, using a cyclic truss-type model to simulate soil-foundation-structure systems, has been investigated. The interacting effects of the supporting soil medium and the foundation construction, including performance of piles, piers, and basements for buildings of one to fifteen stories above grade are taken into account. One horizontal component and the vertical component of the 1940 El Centro and 1952 Taft earthquake accelerations have been selected as input excitations. The seismic response is largest for low, short-period buildings on hard soil, least for filled ground, and intermediate for rock formation and soft-type soil. Response amplitudes vary for different soil types, foundation construction, and building height, and are reduced with increasing soil damping. The vertical acceleration input produces considerable axial forces in the columns of low buildings supported on hard-type soil.

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● **6.8-78** Arya, A. S. *et al.*, **Response of foundation wells of bridges during earthquake,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1702–1703.

● **6.8-79** Sakurai, J. and Minami, J. K., **Effects of soil profiles on the seismic response of buildings,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 991–996.

Structural damage from earthquakes can result from nonuniform soil profiles. The seismic response of structures located on concealed valley, exposed mound, continuous valley, and continuous mound profiles are investigated in this paper. It is assumed that the mound profile is of hard diluvium and the valley profile is of soft alluvium confined by hard soil at both ends. It is found that low rigid structures suffer greater damage than tall flexible structures and that seismic response is high for lowrise structures located at the edge of the mound profile and near the center of the valley profile. Higher responses occur with increasing width of valley profiles. The seismic response of structures on nonuniform soil layers differs markedly from that of structures on horizontally uniform soil layers.

● **6.8-80** Windham, J. E., **Effect of backfill compaction on design criteria for hardened facilities: results of soil-structure interaction calculations for dry types I and II backfill materials,** *Technical Report S-76-4,* Soils and Pavements Lab., U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, June 1976, 92.

The results of a series of two-dimensional plane-strain, dynamic finite element, structure-medium interaction code calculations are presented. These calculations were made to parametrically investigate the effect of variations in constitutive properties of the backfill region around a hypothetical, shallow-buried protective structure. Two parameter studies were conducted on this plane-strain idealization of a simple buried structure while under a long-duration local surface airblast loading. The first study investigated the differences in the dynamic response of the structure under this loading brought about by changing the surrounding backfill from a dense (or well-compacted) glacial till to the same material in a loose (or poorly compacted) condition. The second parameter study is identical with the first except that dense and loose clay shale materials were simulated.

The calculations performed indicated that, for the particular idealized problem investigated, the use of loose rather than dense backfill results in (1) increased deflections across backfill sections, (2) increased loads on, deflections of, and thrusts, shears, and bending moments within the structure, and (3) increased amplitudes of the shock spectra for points on the inside surface of the structure. These results are for the case of a 50-psi overpressure loading caused by a megaton range detonation over a stiff structure. Because of the long positive phase duration, negligible stress attenuation occurs in the loose backfill in spite of its high hysteresis. This would not be the case for very short-duration airblast loadings. It is concluded that the trends observed regarding the effect of backfill compaction on dynamic structural response should not be extrapolated to a vastly different airblast loading condition.

6.9 Fluid-Structure Interaction

6.9-1 Johnson, H. W., Vaish, A. K. and Porter, F. L., **Three-dimensional dynamic response modeling of floating nuclear plants using finite element methods,** *Nuclear Engineering and Design,* 38, *3,* Sept. 1976, 503–510. (For an additional source, see Abstract No. 1.2-2.)

A modeling technique which can be used to obtain the dynamic response of a floating nuclear plant (FNP) moored in an artificial basin is presented. Hydrodynamic effects of the seawater in the basin have a significant impact on the response of the FNP and must be included. A three-dimensional model of the platform and mooring system (using beam elements) is used, with the hydrodynamic effects represented by added mass and damping. For an essentially square plant in close proximity to the site structures, the three-dimensional nature of the basin must be considered in evaluating the added mass and damping. However, direct solutions for hydrodynamic effects with complex basin geometry are not as yet available. A method for estimating these effects from planar finite element analysis is developed.

First, added mass and damping values are obtained from plane-strain finite element models of vertical cross sections through the platform. Fluid finite elements are used to model the seawater. For added mass calculations, the planar models include the platform cross section, the basin profile and the seawater in the basin. For hydrodynamic damping calculations, the planar model includes the platform cross section, the seabed and seawater, infinite in horizontal extent. Added mass and damping values are obtained for each significant mode of platform response. Estimates of three-dimensional added mass and damping are then obtained through combinations of the planar values. The release of the planar constraints of seawater motion and the reflection of gravity waves back to the platform are considered. Effective damping values applicable, on an average, for the entire response time are calculated for each plant mode of response. Since added mass and damping are frequency dependent, the selection of values to be used for a specific loading condition is usually an iterative process.

The accuracy of the planar finite element model in obtaining two-dimensional added mass and damping is shown through comparison with existing and documented

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results. In addition, a comparison is shown for open ocean added mass and damping with a three-dimensional solution using velocity potential functions. It is concluded that the overall technique results in a reasonable and conservative calculation of the dynamic response of the floating nuclear plant.

● 6.9-2 Mamradze, G. P. and Gvelesiani, T. L., **Calculation of hydrodynamic pressure on a dam subjected to seismic excitation** (Opredelenie gidrodinamicheskogo davleniya na plotinu pri seismicheskom vozdeistvii, in Russian), *Seismostoikoe stroitelstvo, 6,* 1977, 14–19.

Hydrodynamic pressure on a dam is usually calculated on the basis of stationary horizontal ground motion. The effects of the vertical component of the seismic excitation are not sufficiently understood. The two-dimensional problem of calculating hydrodynamic pressure arising from the vertical component of seismic excitation is analyzed for the case of a rectangular water reservoir. Numerical calculations were performed on a computer using the formulas derived.

● 6.9-3 Veletsos, A. S. and Yang, J. Y., **Dynamics of fixed-base liquid-storage tanks,** *Proceedings of U.S.-Japan Seminar on Earthquake Engineering Research with Emphasis on Lifeline Systems,* Japan Society for the Promotion of Earthquake Engineering, Tokyo, Nov. 1976, 317–341.

The dynamics of cylindrical liquid-storage tanks has been the subject of continuing studies at Rice Univ. during the past four years. The objectives of these studies have been (1) to provide improved insight into the factors that control the response of such systems to earthquakes; and (2) to develop information and concepts which may be used to evolve rational methods of analysis and design. The tanks were considered to be of circular cross section, fixed at the base, and to have a free liquid surface. Primary attention has been given to the effects of a horizontal component of ground shaking.

The work to date has comprised: studies of the hydrodynamic effects induced in rigid tanks, including comparisons of the exact solution with Housner's approximate solution; the development of a simple approximate procedure for assessing the effects of tank flexibility; the formulation and application of a more refined analysis in which the tank is analyzed by use of a shell theory; and studies of the natural frequencies and modes of vibration of both empty and liquid-filled tanks.

This paper records the highlights of the principal results that have been obtained, with particular reference to the magnitude and distribution of the hydrodynamic wall pressures and the magnitude of the associated base shears. With few exceptions, current design procedures for liquid-storage tanks subjected to earthquakes appear to be based on hydrodynamic wall pressures computed on the assumption that the tank is rigid. It is shown that this approach may significantly underestimate the magnitude of the resulting forces.

● 6.9-4 Sogabe, K., Shigeta, T. and Shibata, H., **Aseismic design of liquid storages,** *Proceedings of U.S.-Japan Seminar on Earthquake Engineering Research with Emphasis on Lifeline Systems,* Japan Society for the Promotion of Earthquake Engineering, Tokyo, Nov. 1976, 343–363.

The authors have conducted experiments and analyses of liquid-storage tanks for the past several years. Their objective is to use the results to develop a design procedure. To identify the fundamental vibrational characteristics of liquid storage tanks, the authors conducted experiments of the earthquake response of cylindrical and spherical storage tanks and an experiment of the effect of vertical motion on the side-slip of a cylindrical storage tank. These results are reported. The results have been used to develop a formula for calculation of the sloshing response of the liquid inside storage tanks.

● 6.9-5 Paul, R. J., **Dynamic behaviour of storage tanks,** *76/10,* Dept. of Civil Engineering, Univ. of Canterbury, Christchurch, New Zealand, Feb. 1976, 81.

This report investigates the dynamic behavior of liquid storage tanks. Theoretical methods of analysis are reviewed and finite element analysis is investigated. Two cylindrical perspex model tanks were constructed, one representing a prestressed concrete water reservoir, and the other an oil storage tank. These models were subjected to static lateral loading and to the dynamic loading of both sinusoidal displacement, and to simulated earthquake displacement using a suitably scaled record of the El Centro 1940 N-S component. Tests were carried out to find the influence of applied accelerations on wave motion and to investigate the stresses induced in the tank wall under dynamic loads. Uplift at the base of the tank, the effects of wave motion on roof structures, and the influence of viscosity on dynamic resonse are studied. Where possible the results are compared with available theory, and the relevance of observed behavior in design considerations is discussed.

● 6.9-6 Au-Yang, M. K. and Skinner, D. A., **Effect of hydroelastic coupling on the response of a nuclear reactor to ground acceleration,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(b), Paper K 5/5, 18. (For a full bibliographic citation, see Abstract No. 1.2-5.)

This paper presents an analytical and experimental study of the effects of hydroelastic coupling on the seismic response of a reactor vessel and its internal components. The hydrodynamic mass matrix for cylindrical shell structures with arbitrary D/l ratios, boundary conditions, and

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mode shapes is derived. In common with published results, this hydrodynamic mass matrix is real and symmetric with positive diagonal elements (virtual masses) and negative off-diagonal elements (coupling coefficients). Thus, while the virtual masses are additive to the physical masses of the structures, hydraulic coupling tends to cancel the elastic coupling between components of a nuclear reactor.

Numerical results, which have been confirmed experimentally, show that: (1) For cylindrical shell structures with D/l ratio typical of what is encountered in a reactor, Fritz's formula, in spite of its wide acceptance, overestimates the hydrodynamic mass for beam-mode response by more than a factor of two in most cases; (2) The hydrodynamic masses are not sensitive to the boundary conditions or local irregularities of the structure. Therefore, the expressions can be applied to cylindrical shell structures with nonclassical boundary conditions and local irregularities, provided that its in-air frequencies and mode shapes can be found by finite element structural analysis. A procedure is suggested to incorporate the computed hydrodynamic mass matrix into existing finite element structural analysis computer codes for the seismic analysis of reactor internal components.

Two specific examples are included to illustrate the effect of hydroelastic coupling on the response of a PWR to ground acceleration. The first example is the coupled beam-mode response of the reactor vessel-reactor internal. The second example is the coupled shell-mode response of the core support cylinder and core basket. The natural frequencies of the systems, the amplitude ratios, and inertia loadings on each component are computed. In the first case, it is shown that the effect of hydroelastic coupling is minimal, whereas in the second case, it is significant. In both cases, the virtual masses do not contribute directly to inertia loading of the components. Rather, their effects are completely cancelled by the off-diagonal coupling terms.

● **6.9-7** Fujita, K. and Shiraki, K., **Approximate seismic response analysis of self-supported thin cylindrical liquid storage tanks,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(b), Paper K 5/4, 12. (For a full bibliographic citation, see Abstract No. 1.2-5.)

Most of the conventional liquid storage tanks are low in height in comparison with their diameters. Recently, tall thin tankers such as refuelling water storage tanks at nuclear power plants have come to be used for special purposes. The seismic responses of such a tank are those of a coupled vibration system consisting of a tank structure and liquid and a sloshing vibration system consisting of liquid surface oscillation, for which the conventional method of analysis is inapplicable. For an accurate analysis of the seismic responses of tall thin tanks of this kind, the authors assumed that the motions of the liquid in the tank would be in accordance with the velocity potential theory, and derived a method of approximate analysis which was applicable to both the liquid-tank coupled vibration system and the sloshing vibration system.

To investigate the appropriateness of this analytical method, the authors constructed a reduced-scale plastic model of a cylindrical tank and obtained the vibration characteristics and seismic response characteristics of the model by using a shaking table. The experimental values showed good agreement with the theoretical values. It is recommended that this new analytical method be employed for the design of tall thin tanks.

● **6.9-8** Lev, O. E. and Jain, B. P., **Seismic response of flexible liquid containers,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(b), Paper K 5/3, 7. (For a full bibliographic citation, see Abstract No. 1.2-5.)

Seismic analysis of liquid storage containers is based, in present design practice, on the method developed by Housner and subsequent modifications and extensions such as by Veletsos and more recently by Epstein. According to this method, part of the liquid contained moves rigidly with the excited container while another part at the top is flexible and participates in sloshing. The container walls are assumed to be rigid, so that the hydrodynamic forces created may be calculated as a product of the flexible mass by the maximum ground or input acceleration. This simple approximation of the exact solution of the wave equation is intuitively appealing and in most cases satisfactory.

The purpose of this work is to demonstrate that the errors introduced by the assumption that the container is rigid could be avoided in practical design, since they may not be on the conservative side. Indeed, the flexibility of the container should not be ignored in cases where the structural period (say 0.10 sec) corresponds to a "peak" in the response spectra. On the other hand, these errors may yield excessively conservative results. In the method described herein, the effects of liquid damping, in the order of .5%, are neglected. Effects of sloshing (nonhorizontal free surface) and liquid compressibility are accounted for.

A practical analysis procedure is presented based on the solution of the two-dimensional wave equation for the liquid, with time-dependent boundary conditions imposed by the dynamic response of the flexible container walls. The liquid and the walls are allowed to iteratively interact until the final common frequencies and hydrodynamic pressures are obtained. The method while still approximate is superior to other such methods where accounting for flexibility is important. It is also generally applicable to containers of any practical shape and structure, as standard dynamic analysis procedures can be applied iteratively. The procedure begins by finding the dynamic response in the

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absence of the liquid. The frequency and displacement of each mode are used in the time-dependent boundary condition in solving the wave equation for the liquid. A pressure function is then obtained and converted to an equivalent mass, which is used to correct the displacement shape and frequency. The process is repeated until no significant correction of the frequency is found. The implementation is done on an open rectangular tank, containing water. Four different cases of analysis are presented, accounting for sloshing and compressibility.

● **6.9-9** Kana, D. D., **Seismic response of flexible cylindrical liquid storage tanks,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(b), Paper K 5/2, 12. (For a full bibliographic citation, see Abstract No. 1.2-5.)

Liquid slosh and tank wall vibrations are studied experimentally in a flexible model storage tank subject to simulated earthquake environments. The program objective is to determine the validity of simple analytical models for representing this earthquake dynamics problem. A similitude study is first performed to develop the important nondimensional parameters. It is found that simulation of both membrane stresses and bending stresses is difficult for geometric scales smaller than 1/3, unless plastic-type materials are used for the model shell. However, an approach is given for studying each independently for 1/16 and 1/7 scale models. These scales are applied to an aluminum model tank of 62.87 cm diameter, 76.2 cm height, and 0.51 mm wall thickness, having flat rigid bottom and top cover. The tank is instrumented to measure liquid slosh amplitudes and pressures, and tank wall vibrations and strains.

Simulation of a scaled earthquake environment is provided by a biaxial shaking table having the the capability of simultaneous independent horizontal and vertical excitations. Ground motion time histories are developed from random signals and are modified to produce specified different earthquake response spectrums along each axis. Both excitation displacements and accelerations are recorded on analog instrumentation tape, along with liquid and tank responses for subsequent processing. Data are acquired for motion of several magnitudes with horizontal only, vertical only, and combined horizontal and vertical excitation.

It is found that liquid slosh amplitudes are principally related to horizontal seismic displacements, while pressures and shell wall flexural vibrations are principally related to both horizontal and vertical seismic accelerations. Wall vibrations and strains are further found to be as sensitive to vertical accelerations as they are to horizontal accelerations. The wall response is comprised principally of the first several flexural or breathing modes which have multiple waves around the shell circumference. Influence of vertical excitation on this type response is surmised to occur through either geometric eccentricities or through parametric excitation mechanisms. Shell wall responses damp out rapidly after the seismic event has subsided. However, low-frequency liquid slosh, which occurs principally in the first antisymmetric mode, persists for a long time after the seismic event. By comparison with predictions from a simple analytical model, liquid pressures are separated into impulsive, or acceleration loads, and convective, or slosh loads. The simple analytical model is shown to be inadequate for certain parameter ranges and earthquake conditions. Explanations for the discrepancies are offered.

● **6.9-10** Clough, R. W. and Clough, D. P., **Seismic response of flexible cylindrical tanks,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(b), Paper K 5/1, 12. (For a full bibliographic citation, see Abstract No. 1.2-5.)

An experimental study of the seismic behavior of thin shell circular cylindrical liquid storage tanks is described. The investigation was planned to evaluate the adequacy of present methods of tank design, and was conducted using the Earthquake Simulator Facility of the Univ. of California, Berkeley. The model tank considered in this paper was 6 ft high by 12 ft in diameter, and was welded from thin sheet aluminum to simulate a steel tank 36 ft in diameter. During testing, the tank had an open top, held 60 in. of water, and was subjected to a time scaled El Centro (1940) earthquake, amplified to a peak acceleration of 0.5 g. Both base free and base fixed conditions were studied.

Results of the experiments demonstrate that fluid pressures included both impulsive and convective components, and that the wave sloshing followed basic theory quite closely. But it also was apparent that the tank flexibility influenced the hydrodynamic pressures, as indicated by pressure amplification in the clamped tank, and by a total change of pressure history in the in the unclamped case. Significant out-of-round distortions of the tank were developed, of a three lobe form for the free base case and with four lobes in the fixed base case. Uplift of the tank base was closely related to the out-of-round deformation of the unanchored tank, whereas initial eccentricities apparently caused the section distortions in the anchored system. Stresses in the tank wall do not follow the expected pattern of response to overturning moment; instead they seem to be mainly associated with the section distortions. At present, there is no analytical procedure for predicting these distortions.

● **6.9-11** Sanchez-Sesma, F. J. and Rosenblueth, E., **Hydrodynamic pressure in semicylindrical reservoir,** *Journal of the Engineering Mechanics Division, ASCE,* **103,** *EM5,* Proc. Paper 13298, Oct. 1977, 913-919.

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Solutions are presented for modal analysis of hydrodynamic pressures generated by the three translational seismic components—longitudinal, vertical, and transverse—on a dam limiting a semicircular cylindrical reservoir. The main purpose is to show the influence of the cross-sectional shape of the reservoir on the hydrodynamic responses. Results are compared with those for a rectangular cross section. Responses are found to be systematically smaller with a semicircular cross section for a given cross-sectional area and depth. It is concluded that for design purposes the fundamental mode adequately approximates the response for various excitations of practical interest.

● 6.9-12 Murtha, J. P. and Kirkley, O. M., **Response spectra for ocean structures**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. II, 1977, 985-990.

The lateral forces caused by earthquakes on offshore structures include fluid-structure interaction effects in addition to structural inertia. The interaction forces include the added mass force, friction, and radiation damping. The study describes the effect of these additional forces on the response spectrum. Also presented are hydrodynamic parameter spectra for determining the kinematic quantities for use in selecting coefficients associated with these forces.

● 6.9-13 Tani, S., Tanaka, Y. and Hori, N., **Dynamic analysis of cylindrical shells containing liquid**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. II, 1977, 1229-1234.

The free oscillations of thin cantilever cylindrical shells partially filled with liquid are studied. The shells considered are comparatively long, and the assumed displacement functions are of beam-type mode in the longitudinal direction. Donnell's equilibrium equations are used. The equations for the natural frequencies are derived from the variational equation by the Rayleigh-Ritz method and the effect of the internal liquid on the shell mode shapes is considered using an added hydrodynamic mass. The results of some computations are presented.

● 6.9-14 Barbat, H., Breaban, V. and Ionescu, C. D., **Dam-reservoir interaction for a dam with flat upstream face during earthquakes**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. II, 1977, 1301-1306.

The dam-reservoir seismic interaction problem is solved for dams with flat upstream faces. The compressibility of water and the effects of horizontal and vertical components of the ground motion are taken into account. The influence of the foundation on the seismic response of the dam is included, so that the problem is treated as a triple interaction. The solution is based on the use of a combined finite element-finite difference technique and on the fast Fourier transform algorithm. Numerical results and conclusions are also given.

● 6.9-15 Reddy, D. P. and Ts'ao, H. S., **Nonlinear analysis of a gravity dam for seismic loads**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. II, 1977, 1294-1300.

A nonlinear dynamic analysis of a gravity dam subjected to earthquake excitations was performed, taking into account the interaction of the reservoir water and the dam. The dam section was modeled by a two-dimensional finite element system. The results of the nonlinear analysis indicated that the dam was stable when subjected to safe shutdown earthquake loads with the water level of the reservoir at a 25-year flood condition.

● 6.9-16 Breaban, V., Barbat, H. and Ionescu, C. D., **Arch dam-reservoir interaction during earthquakes**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. II, 1977, 1267-1272.

An analysis of an arch dam-reservoir system subjected to seismic motion is presented. Using a mathematical model, the theoretical aspects of the interaction problem between the structure and the fluid mass during earthquakes are included. The reservoir-dam system is considered to have linear elastic behavior. The problem is solved starting from the assumption that the arch dam is a flexible structure, the foundations and the abutments are rigid, and the water in the reservoir is compressible. The small displacements hypothesis is considered to be valid. A combined finite element-finite difference method is used in the analysis, taking into account only the horizontal seismic motion along the valley. Finally, numerical results and conclusions are given.

● 6.9-17 Gvelesiani, T. L., Kilasonia, J. N. and Mamradze, G. P., **Wave generation in a reservoir and hydrodynamic pressure in tunnel and conduit of hydroelectric plant during an earthquake**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. II, 1977, 2046-2051.

Water wave generation in a reservoir due to residual deformations (seismotectonic displacements, landslides, and avalanches) arising in the reservoir area during an earthquake is theoretically investigated. As a result of the numerical analysis, simplified formulas are obtained for the determination of the maximum rise of water at the dam site. The results are compared with experimental data and observations.

On the basis of analytical solutions obtained for some simplified schemes of hydroplant pressure systems, the probabilistic characteristics of hydrodynamic pressure resulting from an earthquake are determined. Numerical

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calculations were carried out for the system, consisting of a tunnel, a surge reservoir, and a pressure conduit. Simplified formulas, convenient for design, are also offered.

● 6.9-18 Negoita, A. and Barbat, H., **Seismic analysis of conical frustum elevated water tanks,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 2101-2102.

A dynamic model using the finite element and the finite difference methods, which makes possible the inclusion in the analysis of the hydrodynamic structure-liquid interaction forces, is proposed for the seismic analysis of conical frustum elevated water tanks. The structure-foundation interaction is also taken into account.

● 6.9-19 Paul, D. K. and Swamee, P. K., **Hydrodynamic pressure on dams,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 2118-2119.

Hydrodynamic pressure on rigid dams has been investigated by Westergaard and Chopra. For a sinusoidal ground acceleration, these approaches do not satisfy the initial condition of zero pressure. In this paper, ground accelerations are expressed as a Fourier sine series, and equations that can be used to find hydrodynamic pressure are given.

● 6.9-20 Dungar, R. and Eldred, P. J. L., **The effect of earthquakes on the foundation stability of gravity oil platforms,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1496-1502.

In many cases, gravity platforms are analyzed only for static loads, but the existence of a massive, relatively rigid structure resting on a weak foundation material poses a problem which is potentially of a dynamic nature. This paper presents results for the response of such a structure, analyzed by the finite element method and including boundary dashpots to represent the energy radiated from the structure. The problem of reflecting surfaces caused by foundation layering is investigated; and the response of the structure to dynamic earthquake loading is compared with that predicted for dynamic wave loading.

● 6.9-21 Sogabe, K. and Shibata, H., **Aseismic design of cylindrical and spherical storages for their sloshing phenomenon,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3356-3363.

The aseismic design of liquid storage tanks is discussed. Earthquake response observation and analysis of a cylindrical liquid storage tank model and response analysis of a spherical storage tank are considered. Also presented is an analysis of the effects of vertical motion on the side-slip of a cylindrical storage tank.

● 6.9-22 Kamil, H., Kost, G. and Gantayat, A., **Soil-pile-structure-fluid interaction under seismic loads,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1590-1595.

The major problems associated with the dynamics of soil-pile-structure-fluid interaction are discussed. State-of-the-art procedures for seismic analysis of structural systems involving soil-pile-structure interaction and fluid-structure interaction are reviewed. The application of these techniques to large-scale problems in light of recent experience by the authors is discussed, with emphasis on the use of soil and fluid finite elements. The use of probabilistic methods in explicitly treating the uncertainties in soil properties and structural response characteristics are also briefly reviewed.

● 6.9-23 Diaconu, D. *et al.*, **Seismic response of elevated water-towers, with tronconic tanks and central tube, taking into account the water swinging effect,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2816-2822.

The seismic response and oscillation characteristics of water and the seismic and resonant pressures of liquid at various levels upon the walls of tanks are analyzed. The coupled seismic response of the tank's members when the tank is loaded with a critical volume of water, up to failure, and aspects of the tank's collapse are also discussed.

6.10 Vibration Measurements on Full Scale Structures

● 6.10-1 Chen, C. K., Czarnecki, R. M. and Scholl, R. E., **Vibration tests of a 4-story reinforced concrete test structure,** *JAB-99-119,* URS/John A. Blume & Assoc., San Francisco, Jan. 1976, 108.

This report presents the results of a series of vibration tests conducted in 1974 to determine the dynamic response characteristics of a 4-story concrete test structure located at the Nevada Test Site. The testing of this structure can be generally categorized as follows: nondestructive tests consisting of frequency sweep tests and frequency dwell tests, exciting four translational modes of vibration in the north-south (longitudinal) direction and three modes in the east-west (transverse) direction at low and intermediate force levels; a destructive test consisting of a frequency dwell test, exciting the lowest north-south translational mode at intermediate and high force levels, during which the structure exhibited inelastic response and suffered major structural damage; and a series of post-destructive tests using force levels similar to the nondestructive tests,

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conducted to investigate the response of the structure after it had experienced structural damage.

● **6.10-2** Norman, C. D., Crowson, R. D. and Balsara, J. P., **Dynamic response characteristics of a model arch dam,** *Wind and Seismic Effects,* VI-85–VI-117. (For a full bibliographic citation, see Abstract No. 1.2-4.)

The dynamic response characteristics of a model arch dam are given in detail. These characteristics were determined by subjecting the model to a vibratory loading. Resulting deformations and velocities are given.

● **6.10-3** Liu, S.-C., Fagel, L. W. and Dougherty, M. R., **Earthquake-induced in-building motion criteria,** *Journal of the Structural Division, ASCE,* **103,** *ST1,* Proc. Paper 12684, Jan. 1977, 133–152.

Two 10-member ensembles of digitally generated waveforms that simulate real earthquake accelerograms are used to excite mathematical models of two-story to 20-story buildings of widths ranging from 100 ft to 300 ft and founded on soils characterized by shear-wave velocities of up to 4,000 fps. Realistic equivalent structural damping is determined and used in this analysis based on the actual building response data recorded during the San Fernando earthquake of 1971. Calculated upper bound motion responses on floors throughout the buildings are presented in terms of peak values of acceleration, velocity, and displacement. Envelopes of damped response spectra derived from the in-building accelerograms are also presented, and the use of these spectra is demonstrated with an illustrative example.

● **6.10-4** Gundy, W. E. *et al.,* **A comparison of vibration tests and analysis on nuclear power plant structures and piping,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(b), Paper K 8/6, 16. (For a full bibliographic citation, see Abstract No. 1.2-5.)

This paper presents the analysis and testing methods, selected theoretical and experimental results, and comparisons of these results for a series of ambient, forced vibration, snapback, and low-level explosive tests performed on the Heissdampfreaktor (HDR), a decommissioned nuclear power plant facility in the Federal Republic of Germany. These tests and theoretical analyses were used to determine the dynamic characteristics and response of the reactor containment structure and several piping systems under simulated seismic excitation. The primary concern was the identification of the critical eigenfrequencies, modal deformations, and damping values of each structural system tested.

The HDR is a nuclear power plant with a containment structure consisting of a cylindrical steel liner and a concrete outer shell. The structure is only slightly embedded and the liner and outer shell are structurally separate over much of their height. A two-dimensional finite element model, a three-dimensional lumped mass model, and an axisymmetric shell model were prepared prior to testing. Soil-structure interaction effects were included. A unique feature of these models was that the response of the structure to eccentric mass shakers at various locations was predicted prior to forced vibration tests. A comparison of theoretical and experimental results indicated that the three-dimensional lumped mass model gave best results. The comparison gave insights into steps needed to improve the reliability of the modeling process.

Finite element techniques were employed to determine the eigenparameters and response to sinusoidal forced vibration testing of a complex nuclear piping network comprising a representative section of the superheated, steam-saturated steam system at HDR. Modeling and testing techniques employed are described, and analytic and experimental results are compared. Less than a 23% difference between calculated and measured resonant frequencies was found. Disagreement in mode shapes was less than 20%. Damping values were found to be between 1% and 3% of critical in the first five modes at peak responses on the order of 10% g.

Other conclusions and recommendations are drawn from the results of the experimental tests and theoretical predictions relating to the analysis and design of nuclear power plants for earthquake interaction, analytical modeling techniques, calculation and identification of critical damping values, identification of primary system modes, and improved methods of full-scale dynamic testing.

● **6.10-5** Ibanez, P. *et al.,* **In-situ testing for seismic evaluation of Humboldt Bay nuclear power plant for Pacific Gas and Electric Company,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(b), Paper K 8/3, 16. (For a full bibliographic citation, see Abstract No. 1.2-5.)

Changing Nuclear Regulatory Commission requirements have resulted in a seismic reevaluation of the Humboldt Bay nuclear power plant based on current, more modern seismic criteria and methods of analysis. The original design was based on an equivalent static load of 0.25 times the dead load and equipment load. The current seismic reevaluation is based on a 0.25 g operating-basis earthquake utilizing dynamic analysis methods. In-situ vibration testing was used to supplement theoretical analysis of selected storage tanks, pumps, motors, valves, electrical cabinets, instrumentation, piping, and major structures. These techniques proved useful in providing damping values, confirmation of theoretical models, development of

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models of exceedingly complex structures, and, to a limited extent, for in-situ proof testing. Seismic upgrading of several structures was subsequently performed.

Five case studies are discussed in this paper. In each case, a physical description, reasons for requiring tests, the testing performed, and use of test data to aid analysis is given. The basic conclusion of this paper is that in-situ testing is a feasible and useful method to improve and guide theoretical studies and seismic design of nuclear power plant equipment and structures.

● **6.10-6** Hirasawa, M., Okajima, S. and Satoh, K., **Dynamic behavior of a nuclear reactor building,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(b), Paper K 8/2, 9. (For a full bibliographic citation, see Abstract No. 1.2-5.)

Forced-vibration tests were performed on a nuclear reactor building to investigate its dynamic characteristics. Tests were made on two orthogonal directions, one was parallel where a stack 50 m high was located at 50 m distance from the reactor building and the other was perpendicular. Two close resonant frequencies were obtained from a resonant curve of the parallel direction, 5.1 Hz and 5.9 Hz, while one was obtained on the perpendicular direction, 5.1 Hz. Critical damping values were from 15 to 18%. The displacement was mainly caused by rocking vibration and deformation of the building was small.

Attempts were made to correlate the experimental results with finite element models that included the building-soil-stack system. It was found that the frequency 5.1 Hz was a natural frequency of the building-soil system and that the 5.9 Hz was one of the ground, and at these frequencies two peaks were produced in the parallel direction, because the vibration decreased between the two frequencies under the coupling effect of the second vibration of the stack, while the coupling effect was negligible on the perpendicular direction.

In conclusion, the experimental and analytical research indicates that the effect of structure-soil-structure interaction should be considered for the prediction of the seismic response and that the finite element model can be a useful method.

● **6.10-7** Crowson, R. D. and Norman, C. D., **Comparison of vibration test results for a model and prototype arch dam,** *Technical Report N-77-1,* Weapons Effects Lab., U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, Mar. 1977, 51.

Vibration tests were conducted on both the prototype and a 1:24 scale model of the North Fork Dam, a double curvature arch dam, to determine natural frequencies, mode shapes, damping ratios, and hydrodynamic pressures. Two vibrators mounted on the crest of the dam were used as input excitation sources for both series of tests. Electromagnetic shakers capable of a 40 lb output were used in the model tests, while counterrotating, eccentric mass exciters capable of a 5000 lb output were employed for the prototype. Velocities were measured along the crest and downstream face of the model, whereas accelerations were measured in the same locations on the prototype. Measurements in both curves were taken at the dam-reservoir interface while the structures were excited at resonant frequencies. Variations in the measured natural frequencies for the model and prototype ranged from approximately 3% for the third and fourth modes to approximately 25% in the second mode. Comparisons of corresponding mode shapes were quite good. A linear three-dimensional finite element code, SAP, was also used to compute mode shapes and natural frequencies for the dam. This analytical calculation was very accurate, as the variation in prototype and predicted natural frequencies ranged from less than 1% for modes 3 and 4 to 11.6% for mode 2. The finite element analysis also indicated the first natural frequency (compression mode) to be very close to the first bending modal frequency. In both model and prototype tests, these two modes could not be separated, and the compression mode was not excited. Damping in both model and prototype ranged from approximately 2 to 5% of critical. These values are consistent with structural damping values for these types of structures.

● **6.10-8** Muto, K. *et al.*, **Two-dimensional vibration test and simulation analysis for a vertical slice model of HTGR core,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(b), Paper K 7/8, 12. (For a full bibliographic citation, see Abstract No. 1.2-5.)

This paper describes a two-dimensional vibration test and the simulation analysis for a vertical slice model of an HTGR core. The purpose is to verify by experiments and by analytical calculations such dynamic properties as resonant characteristics and vibration modes of the core, lumping of blocks, collision pattern of blocks, and distribution of reflector reactions.

● **6.10-9** Hudson, D. E., **Dynamic tests of full-scale structures,** *Journal of the Engineering Mechanics Division, ASCE,* **103,** *EM6,* Proc. Paper 13446, Dec. 1977, 1141-1157.

Basic objectives of dynamic testing are outlined, and major test types are summarized. Force generation equipment suitable for tests of full-scale structures is described, and typical results are given, with examples drawn mainly from the field of earthquake and wind excitations. Results of low-level ambient vibration tests are compared with higher level vibration generator tests, and with excitations

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caused by strong earthquake ground motion. Some instrumentation systems suitable for such testing are described. Information is given on the ground motions caused by large underground explosions, and the use of such explosions to simulate damaging earthquake excitations is considered.

● **6.10-10** Nasu, N. *et al.*, **Seismometric investigation of the motion of a submerged tunnel in earthquake and at ordinary time,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2554–2559.

A submerged tunnel was completed in 1974 by the Nippon Kokan Kabushiki-kaisha, joining the Mizue and Oogishima sites of the company which lie on both sides of the Keihin Canal. A seismological observation is being carried out to clarify the vibrational characteristics of the tunnel. This is necessary for the control of the tunnel. The seismographs are installed not only in the tunnel, but also on the ground nearby. Strain meters have been placed on the tunnel. Thus, the simultaneous observation of the movements of the ground and the tunnel is possible. Some results of observations of microtremors, earthquakes, and blast motions are reported.

● **6.10-11** Stephen, R. M. and Bouwkamp, J. G., **Dynamic behavior of an eleven story masonry building,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2711–2716.

The results of a forced vibration study of an eleven-story reinforced masonry structure are compared to a dynamic analysis. The experimental work determined the natural frequencies, mode shapes, and damping values. Considerable flexibility of the foundation was noted in the experimental studies. The analytical model was developed with both a fixed and a flexible base. Experimental and analytical resonance data are compared.

● **6.10-12** Foutch, D. A. and Jennings, P. C., **Dynamic tests of full-scale structures,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2705–2710.

Dynamic tests of full-scale structures provide a unique opportunity to verify the validity of analytical models used in analysis for design and research. In the study reported here, the present status of experimental verification of analysis is discussed; and newly applied experimental procedures, which involve closely spaced three-dimensional measurements of motion on selected floors of a building and the measurement of strain in structural members, are presented. These detailed measurements help resolve uncertainties which may arise in current modeling practices. Examples illustrating these points are drawn from recent forced vibration tests of a nine-story reinforced concrete building and a twelve-story steel frame structure. Suggestions regarding future experimental efforts are presented.

● **6.10-13** Muto, K. and Kuroda, T., **Two dimensional vibration test for a vertical slice model of HTGR core,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2614–2619.

The high temperature gas-cooled reactor (HTGR) is considered a nuclear reactor with higher performance compared to conventional nuclear reactors, and one will be constructed in Japan in the near future. Selecting a vertical slice (25 block columns x 12 or 13 layers, and 2 reflector columns) across the core, a forced vibration test of a 1/5 scale model was performed by using a shaking table to obtain dynamic behaviors of the core. In addition to the test, a computer code was developed and its application was successful in simulating the test results.

● **6.10-14** Chen, C. K., Czarnecki, R. M. and Scholl, R. E., **Vibration tests of a 4-story concrete structure,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2753–2758.

A series of forced vibration tests on a full-scale four-story reinforced concrete test structure were performed to investigate its dynamic response before, after, and during the time it underwent structural damage. First, nondestructive tests were conducted, exciting four translational modes at force levels within the elastic limit, during which the structure suffered no structural damage. Next, a destructive test excited only the lowest translational mode at high-amplitude destructive levels, during which the structure exhibited inelastic response and suffered major structural damage. Postdestructive tests used force levels similar to the nondestructive tests.

● **6.10-15** Paskalov, T. A. *et al.*, **Forced vibration full-scale tests on earth-fill and rock-fill dams,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2829–2834.

The influence of several factors which affect the evaluation of the dynamic characteristics of earth and rockfill dams is discussed. The principles and procedure of full-scale forced-vibration studies of such structures are presented. Experimental results of dynamic tests carried out on two earthfill and two rockfill dams are also included.

● **6.10-16** Rainer, J. H. and Van Selst, A., **Dynamic properties of a suspension bridge,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2835–2842.

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The measurement program is described for determining the natural frequencies, mode shapes, and damping values of the Lions' Gate suspension bridge at Vancouver, British Columbia, Canada. The measured natural frequencies are compared with the calculated frequencies using a continuum model and a lumped parameter model.

● **6.10-17** Ohta, T. *et al.*, **Results of vibration tests on tall buildings and their earthquake response,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2717–2722.

Forced vibration tests were carried out on 18 tall buildings to investigate their natural periods and damping coefficients. Critical damping ratios were estimated. Also, observations of the dynamic behavior of 11 tall buildings during earthquakes were performed to obtain their natural periods and distribution of maximum acceleration. From the acceleration records of these buildings, the authors estimated the ratio of critical damping.

● **6.10-18** Foutch, D. A. and Housner, G. W., **Observed changes in the natural periods of vibration of a nine story steel frame building,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2698–2704.

A case study is presented of the variable nature of the dynamic properties of a nine-story steel frame building based on vibration tests conducted during the last 12 years at the Jet Propulsion Lab. in Pasadena, California. Observed changes of nearly 50 percent in the natural periods of vibration are attributed to observed changes in the mass and stiffness distribution of the structure caused by the addition of building materials during construction, by large relative displacements and dynamic forces experienced during the 1971 San Fernando earthquake, and by the repair program conducted after the earthquake. The implications of these changes in the dynamic characteristics of the structure for those who perform dynamic analysis are discussed.

● **6.10-19** Sugimoto, Y. and Sato, Y., **Telecommunications equipment seismic effect study,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3310–3316.

On-site vibration tests and earthquake observations were carried out on a switch frame-building system. In addition, vibration tests on bay-type equipment and data processing equipment units were conducted, using a shaking table. The dynamic properties and seismic resistance capacity of telecommunications equipment and installation practices were clarified and some improvements made.

● **6.10-20** Shah, P. C. and Udwadia, F. E., **Optimal sensor location for identification of building structures using response to earthquake ground motions,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2949–2955.

This paper deals with finding the optimal position along the height of a building structure where a sensor may be located, such that records of strong ground shaking obtained there would yield a maximum amount of information about the dynamic characteristics of the structure. After modeling a structure by a shear beam, a general technique for solving this problem has been indicated. Some preliminary results for the estimation of the structural stiffness as a function of height have been presented together with their verification using simulated data.

● **6.10-21** Shapiro, G. A. and Zakharov, V. F., **The dependence of inertial force upon displacements in the nonlinear area under oscillations,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2889–2891.

Vibration resonance tests of full-scale buildings of various structural systems were carried out. These tests make it possible not only to achieve designed inertial seismic loads, but also to considerably exceed them. Tests of large models and full-scale buildings permitted observation of the behavior of the structures at all stages of loading and evaluation of the extent of seismic resistance.

● **6.10-22** Hagio, K. *et al.*, **Earthquake motion measurement of plant towers on soft subsoil,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1522–1528.

Earthquake motion in and around the towers of a petrochemical plant built on reclaimed soil has been measured since December 1974 to determine the dynamic behavior of a long-period structure supported on piles, the surrounding soft subsoil, and soil-structure interaction. Also studied were the dynamic characteristics of a combined footing. The measurement system and analytical results are discussed.

● **6.10-23** Denisov, B. E. *et al.*, **Automatized system of the processing and analysis of the earthquake records at the engineering-seismometric service stations,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 884–887.

Presented is an earthquake research program which is being carried out at engineering-seismometric stations in the U.S.S.R. Studies of two basic problems are discussed: the interaction of load-bearing components of a building with its foundation, and the distribution of seismic forces between the members of spatial load-bearing structures.

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● **6.10-24** Kanai, K. and Shimosaka, S., **On the seismic damping of actual buildings (the case of the San Fernando earthquake),** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1536–1541.

● **6.10-25** Petrovski, J. *et al.,* **Dynamic behavior of a multistory triangular-shaped building,** *EERC 76-3,* Earthquake Engineering Research Center, Univ. of California, Berkeley, Oct. 1976, 135. (NTIS Accession No. PB 273 279)

As a part of a continuing program to evaluate the dynamic response of actual structures and to accumulate a body of information on the dynamic properties of structures, especially when these structures have novel design features, a dynamic test program was conducted on the forty-story Century City Theme Tower building. The dynamic tests of the building included both a forced vibration study and an ambient vibration study. These results are compared and, in general, show very good correlation. A mathematical computer model of the Theme Tower was formulated and the results of the analysis are presented and compared to the experimental results. The results compare favorably.

● **6.10-26** Ashkinadze, G. N. and Simon, Y. A., **The study of non-linear performance of structures in frameless earthquake-proof residential buildings by means of powerful vibrators,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2874–2880.

The ability of large-panel buildings to withstand strong earthquakes was tested by means of vibration generators. Nonlinear dynamic response, slackening, and safety factors affecting individual elements and joints are studied.

● **6.10-27** Delpiano P., A., **Analysis of the measurements of the periods of vibration of Peruvian buildings** (Analisis de las medidas de periodos de vibracion de edificios Peruanos, in Spanish), Dept. de Ingenieria, Pontificia Univ. Catolica del Peru [Lima], Feb. 1977, 20.

6.11 Experimental Facilities and Investigations

● **6.11-1** Figarov, A. G. and Iskhagi, Zh. N., **Investigation of the strength and deformation properties of complex limestone brickwork structures under central compression** (Issledovanie prochnostnykh i deformativnykh svoistv kladki kompleksnoi konstruktsii iz izvestnyakogo kamnya pri tsentralnom szhatii, in Russian), *Seismostoikoe stroitelstvo, 3,* 1977, 43–47.

In the Azerbaidzhan S.S.R., limestone is the most commonly employed construction material. Because of the seismicity of the region, complex brick structures are utilized. Presented are methods and results of a series of experiments testing the strength and deformation properties of such brick structures. Experimental and analytical results are compared.

● **6.11-2** Katen-Yartsev, A. S. *et al.,* **Investigation of gravitational seismic damping systems for buildings using physical models** (Issledovanie na fizicheskikh modelyakh gravitatsionnoi sistemy seismoizolatssii zdanii, in Russian), *Seismostoikoe stroitelstvo, 3,* 1977, 34–37.

Methods and results of investigations of the behavior of gravitational seismic damping systems using physical models and full-scale structures are presented. The systems considered are based on the interaction between a vibrating soil base and a structure supported by ellipsoids of revolution. Theoretical considerations show that by suitably choosing the parameters of the ellipsoids natural frequencies may be lowered substantially. In order to compensate for displacements caused by weak low-frequency seismic or wind excitations additional supports may be installed which would disengage when subjected to intensive seismic excitations.

6.11-3 Takei, A. *et al.,* **Structural shock tests of prototype FBR "Monju" scale models,** *Nuclear Engineering and Design,* 38, *1,* July 1976, 109–129.

This paper presents some results of experiments which simulate the structural dynamic response of a LMFBR primary coolant boundary to a hypothetical core disruptive accident (HCDA) based on scale models and high explosives. It was noted that high explosives are no longer a good simulant of the HCDA. However, the main purpose of the program, which included this experiment, is not to experimentally predict the dynamic response of the reactor structure at the HCDA, but to validate computer codes, which describe the pressure wave propagation and damage process in the reactor structures, using data obtained from these model experiments. The experiments were undertaken using many 1/15 scale simple models of the reactor vessels and internal structures, as well as 1/15 and 1/7.5 scale complex models of the interim design of prototype LMFBR "MONJU." Simple model experiments involved a series of shock tests using pentolite to investigate the configuration effects of the vessel restraining section, the dipped-plate effect and the core barrel effect, respectively.

● **6.11-4** Barda, F., Hanson, J. M. and Corley, W. G., **Shear strength of low-rise walls with boundary elements,** *Reinforced Concrete Structures in Seismic Zones, SP-53,* American Concrete Inst., Detroit, 1977, 149–202.

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Results of tests on eight specimens representing low-rise shear walls with boundary elements are reported and analyzed. The principal variables included amount of flexural reinforcement, amount of horizontal wall reinforcement, amount of vertical wall reinforcement, and height-to--horizontal length ratio. Flexural reinforcement was varied from 1.8% to 6.4% of the boundary element area, horizontal wall reinforcement and vertical wall reinforcement were varied from 0 to 0.5% of the wall area, and height-to--horizontal length ratio was varied from 1/4 to 1. The test program was designed to determine the effect of load reversals. Also, one specimen was repaired and retested. Results indicate that current design procedures underestimate the strength of low-rise shear walls, even when the walls are subjected to reversed load. Finally, a suggested design procedure is presented.

● **6.11-5** Laible, J. P., White, R. N. and Gergely, P., **Experimental investigation of seismic shear transfer across cracks in concrete nuclear containment vessels,** *Reinforced Concrete Structures in Seismic Zones, SP-53,* American Concrete Inst., Detroit, 1977, 203–226.

Shear transfer (across open cracks in reinforced concrete) by the mechanism of interface shear transfer on the rough concrete surfaces is investigated experimentally. Fully reversing cyclic shear stresses of about ± 180 psi (1240 kN/m^2) are applied across crack widths of 0.010, 0.020, and 0.030 in. (0.25, 0.51, and 0.76 mm). The cracks are restrained from opening by steel restraint bars placed outside the concrete specimen to avoid dowel action. Slip at the crack, increase in crack opening, and forces in the restraining bars normal to the crack are measured as functions of initial crack width, shear stress magnitude, cycles of shear stress, restraining bar axial stiffness, concrete strength, aggregate properties, age at loading, and presence of construction joint at the crack. The basic behavioral modes are explained. It is concluded that substantial shear forces may be transmitted across cracks by this mechanism, with predictable bounds on slip and degradation of stiffness with cycling.

● **6.11-6** Ernst, G. C. and Smith, G. M., **Displacement versus force-seeking effects for reinforced concrete frames,** *Reinforced Concrete Structures in Seismic Zones, SP-53,* American Concrete Inst., Detroit, 1977, 227–246.

Oscillatory motions were applied to continuous reinforced concrete frames under laboratory control for both displacement and force-seeking conditions. It was found that a shift in the axis of oscillation can occur when full plastic response develops. Force-seeking oscillations greater than those required to produce full plastic response were found to shift the axis of oscillation further in the direction of the initial shift, thereby resulting in a crawling of the displacement of the frame. In such action, structural damage can be exceptionally severe. For the testing, three basically different procedures were selected. All frames carried a simulated gravity load, but the lateral loading effect was produced by (a) simulated earthquake displacements, (b) a "load-seeking" condition, or (c) a "displacement-seeking" situation. One frame was repaired after collapse and retested. A damaged frame could be successfully repaired if returned to its original configuration after removing all damaged concrete. Load control methods that produced a "force-seeking" condition, rather than a "displacement-seeking" situation were found to clearly develop "off--center shifts" and "crawling" of the frames. Structural distress was found to occur with steel strains in the yield plateau, and such distress progressed with severity as soon as the ultimate moment was attained at a critical section. Degradation of load resistance accompanied repeated cyclic loading at or beyond steel yield strains. Results pointed to the need to include cycling under force-seeking conditions at and beyond steel yield strains at each critical section.

● **6.11-7** Henager, C. H., **Steel fibrous, ductile concrete joint for seismic-resistant structures,** *Reinforced Concrete Structures in Seismic Zones, SP-53,* American Concrete Inst., Detroit, 1977, 371–386.

Tests were made of a ductile concrete joint designed to minimize steel congestion common to seismic-resistant building joints. An experimental comparison was conducted on two full-sized beam-column joints. One was a conventional joint using hoops in accordance with the seismic-resistant design specifications of ACI 318-71; the other was a modified joint using steel fibrous concrete in the joint region in lieu of the hoops. The joints were subjected to loading representing the effect of two major earthquakes. The modified steel fibrous concrete joint developed a higher ultimate moment capacity and was somewhat stiffer than the conventional joint. The modified joint appeared to be more damage tolerant and resisted cracking better than the conventional joint. Fabrication of the modified joint was simpler and concrete placement was made easier because of reduction of steel congestion. It is estimated that use of the modified joint in building construction would result in savings of about $100 per joint.

● **6.11-8** Hassan, F. M. and Hawkins, N. M., **Anchorage of reinforcing bars for seismic forces,** *Reinforced Concrete Structures in Seismic Zones, SP-53,* American Concrete Inst., Detroit, 1977, 387–416.

High-intensity cyclic loading tests are reported for 13 stub cantilever specimens. Bond conditions for the tensile reinforcement in those specimens were presumed to simulate conditions for beam bars in beam-column connections. A displacement-controlled loading device was used. Measurements were made for strains in the beam bar and relative displacements between the concrete and the "attack" and "tail" ends of that bar. Variables were the

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loading history for the bar, its surface geometry, and its end anchorage.

Except for fully reversed loading, failure occurred when the displacement at the attack end reached that for failure under monotonically increasing load. For fully reversed loading, bond deterioration decreased the stiffness, reduced the attack displacement for failure, and caused bars terminating in standard 180° hooks to have resistances dependent only on their "lead-in" lengths. Once slip penetrated the complete anchorage length, capacities on reversal of loading were negligible. Statistical formulas are developed relating the penetration of slip along the bar and energy absorption to the test variables. A simple geometric model is developed verifying the form of those formulas and identifying quantities required where variables differ from those of the test program.

● **6.11-9** Uzumeri, S. M., **Strength and ductility of cast-in-place beam-column joints,** *Reinforced Concrete Structures in Seismic Zones, SP-53,* American Concrete Inst., Detroit, 1977, 293–350.

The results of an experimental study of the behavior of cast-in-place reinforced concrete beam-column joints, subjected to slow load reversals simulating seismic loading, are presented. Variables were the amount and size of joint reinforcement, and stress versus strain characteristics of the joint steel. Specimens were 15 in. x 15 in. (38.1 cm x 38.1 cm) columns and 12 in. x 20 in. (30.5 cm x 50.8 cm) or 15 in. x 20 in. (38.1 cm x 50.8 cm) beams. Four corner joints and four isolated joint specimens were tested. Throughout the test the column was under constant axial compressive force of 520 (2.36 x 10^5 kgf) Hz.

It was found that loading history does not affect the strength but seriously affects the stiffness of beam-column subassemblies; that the ACI equation predicts the cracking shear well but that the 45° truss analogy does not predict the behavior of reinforced joints. Stress versus strain characteristics of the joint steel have a significant effect on confinement provided and the energy dissipating capacity of beams depends significantly on the anchorage provided at the joints. To ensure that the plastic hinge is formed in the beam, the column flexural strength should be based on the core alone.

● **6.11-10** Townsend, W. H. and Hanson, R. D., **Reinforced concrete connection hysteresis loops,** *Reinforced Concrete Structures in Seismic Zones, SP-53,* American Concrete Inst., Detroit, 1977, 351–370.

Twenty-two reinforced concrete beam-column T-shaped joints were constructed using the same strength of material and geometrical configuration for each specimen. Test loading conditions were chosen to represent earthquake loading producing steel strains well beyond yield. The three test parameters examined in detail included magnitude of column axial load including axial tension, the magnitude of hinge rotation, and the number of cycles of inelastic loading. Test results show that these exterior joints were able to sustain a large number of inelastic cycles and still function, and that the most rapid change in hysteretic behavior occurred during the first cycle. The effect of column axial loading is that tension causes more rapid changes in the hysteretic behavior than compression.

● **6.11-11** Inaba, S., **Large-scale testing programs related to wind and seismic effects currently under way in Japan,** *Wind and Seismic Effects,* VI-37–VI-43. (For a full bibliographic citation, see Abstract No. 1.2-4.)

Described are the large-scale testing programs currently under way in Japan, the organization, the name of individuals in charge, and a brief description of the objectives, status, and time schedule.

● **6.11-12** Inaba, S. and Kinoshita, S., **Dynamic test of a circuit breaker for transformer substation,** *Wind and Seismic Effects,* VI-50–VI-60. (For a full bibliographic citation, see Abstract No. 1.2-4.)

During earthquakes dependability of electric power supply systems is required. However, earthquake damage to electric power systems has resulted from the structural failure of porcelain insulators, which are commonly used for electric transmission equipment.

Dynamic tests of a circuit breaker of capacity 72/84 kv used for a transformer substation were conducted using the large-scale shaking table of the National Research Center for Disaster Prevention in 1975. The purpose of the test was to determine the dynamic characteristics of the prototype structure shielded with a porcelain insulator. Sinusoidal waves of resonant frequency and earthquake-simulated waves were applied to the test structure. It was found that the failure of the porcelain insulator governs the seismic resistibility of the circuit breaker and that the maximum allowable acceleration at the top of the structure is 8.0 g.

● **6.11-13** Buland, P. *et al.,* **A calculation model for a HTR core seismic response comparison with experimental results on the VESUVE shaking table,** *International Seminar on Extreme Load Conditions and Limit Analysis Procedures for Structural Reactor Safeguards and Containment Structures,* U 5/5, 9. (For a full bibliographic citation, see Abstract No. 1.2-2.)

This paper presents the experimental results obtained at Saclay on an HTGR core model; comparisons are made with analytical results. Two series of horizontal tests have been performed on the shaking table VESUVE: sinusoidal test and time history response. Acceleration of graphite

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blocks, forces on the boundaries, relative displacement of the core and PCRV model, and impact velocity of the blocks on the boundaries were recorded. Sinusoidal tests consisted of frequency sweeps at constant displacement level and sine dwells at constant g-level.

These tests have shown the strongly nonlinear dynamic behavior of the core. The resonant frequency of the core is dependent on the level of the excitation. These phenomena have been explained by a computer code, which is a lumped mass nonlinear model. Good correlation between experimental and analytical results was obtained for impact velocities and forces on the boundaries. This comparison has shown that the damping of the core is a critical parameter for the estimation of forces and velocities. El Centro time history displacement at the level of PCRV was reproduced on the shaking table. The analytical model was applied to this excitation and good comparison was obtained for forces and velocities.

● **6.11-14** Popov, E. P. and Stephen, R. M., **Tensile capacity of partial penetration groove welds,** *Journal of the Structural Division, ASCE,* **103,** *ST9,* Proc. Paper 13178, Sept. 1977, 1721–1729.

The flanges of six W14x320 sections with milled ends were joined together at midheight with different size partial penetration groove welds to form short columns; one additional specimen had complete penetration groove welds through both flanges. Four of the specimens were failed directly in tension; three were subjected to a few compression-tension cycles until failure. The experiments were designed to simulate conditions which can develop in some columns of steel frames during severe earthquakes. The spliced joints developed their specified strengths with the smallest weld attaining highest stress. Failure for all specimens with partial penetration groove welds occurred through the splice exhibiting very limited ductility. The data generated by these experiments should be useful for earthquake design as well as provide basic information on the behavior of partial penetration groove welds of unprecedented size.

● **6.11-15** Freeman, S. A., **Racking tests of high-rise building partitions,** *Journal of the Structural Division, ASCE,* **103,** *ST8,* Proc. Paper 13136, Aug. 1977, 1673–1685.

The results of a program of racking tests of wall panels can be incorporated into the analysis of high-rise structures influenced by nonstructural partitions. Stiffness and energy-absorbing characteristics were obtained from racking tests of wall panels simulating lateral interstory displacements in high-rise buildings. These characteristics were combined with structural frame periods and damping to estimate overall building periods and damping. Illustrative examples are presented.

● **6.11-16** Duffield, R. C. *et al.*, **Damping characteristics of composite girder,** *Journal of the Structural Division, ASCE,* **103,** *ST1,* Proc. Paper 12685, Jan. 1977, 105–118.

An experimental laboratory program was conducted to determine the damping characteristics of a composite girder at different strain levels and the effect of structural deterioration due to cyclic loading on these characteristics. The cyclic loading was imposed by means of a moving-mass closed-loop electrohydraulic actuator system. Five different methods were employed in the investigation to evaluate the equivalent linear viscous damping of the composite girder. The results show that strain level and structural deterioration have a considerable effect on the damping ratio. Initially, the damping ratio increased as the strain level increased. However, after the composite girder had incurred some structural deterioration, the damping ratio decreased as the strain level increased. All five of the different methods used to determine the damping ratio yield approximately the same value for the damping ratio and predict the same trends with regard to the strain level.

● **6.11-17** Thompson, K. J. and Park, R., **Cyclic load tests on prestressed and reinforced concrete beam-column joints,** *76-8,* Dept. of Civil Engineering, Univ. of Canterbury, Christchurch, New Zealand, June 1976, 57.

Test results obtained from ten concrete beam-interior column frame assemblies subject to static cyclic loading simulating the effect of intense earthquake loading are described. The frame members contained a range of proportions of prestressed and nonprestressed steel. Information concerning inelastic flexural deformation capacity of members and joint core shear design are presented.

● **6.11-18** Lee, D. L. N., Wight, J. K. and Hanson, R. D., **RC beam-column joints under large load reversals,** *Journal of the Structural Division, ASCE,* **103,** *ST12,* Proc. Paper 13405, Dec. 1977, 2337–2350.

An experimental investigation to determine the shear strength of reinforced concrete beam-column joints is presented. The test specimens consisted of six exterior beam-column subassemblages. The primary variables were the amount of transverse reinforcement in the joint, the magnitude of axial load on the column, and the severity of the load reversals applied to the beam. The experimental results indicate that the shear resisted by the concrete in the joint was considerably greater than the values suggested by current recommendations. The increase in shear strength is assumed to be influenced by the ratio of the sum of column flexural capacities to that of the beams framing into the joint. It is concluded that this ratio should be considered when designing a beam-column joint.

● **6.11-19** Clough, R. W. and Bertero, V. V., **Laboratory model testing for earthquake loading,** *Journal of the*

Engineering Mechanics Division, ASCE, **103**, *EM6,* Proc. Paper 13444, Dec. 1977, 1105–1124.

Two types of facilities used at the Univ. of California, Berkeley, for laboratory study of earthquake response characteristics of typical structures are described: (1) a 20 ft sq shaking table; and (2) a variety of controlled loading devices. Shaking table tests of a 0.7 scale concrete building frame, a 1/30 scale highway overcrossing, and a 1/3 scale liquid storage tank are mentioned. Controlled loading facility testing of concrete and steel members, joints, and subassemblages is described, and typical results are presented to demonstrate typical behavior mechanisms. Since loading rate has only a minor influence on the behavior of concrete specimens, earthquake response behavior can be evaluated by pseudo-static testing procedures. Other comparative advantages and disadvantages of the two types of testing facilities are presented.

● **6.11-20** Priestley, M. J. N., **Cyclic testing of heavily reinforced concrete masonry shear walls,** *76-12,* Dept. of Civil Engineering, Univ. of Canterbury, Christchurch, New Zealand, Oct. 1976, 75.

This report summarizes test results of six heavily reinforced concrete masonry shear walls. The test program was designed to investigate the necessity for the low ultimate shear stress specified by masonry codes. Care was taken to accurately model good but realistic design practice in detailing, and variables investigated in the series included steel percentage, influence of vertical load, and confinement of potential crushing areas by mortar bed confining plates. Results are presented which clearly indicate that the maximum current code allowance for ultimate shear stress is unreasonably low. No wall suffered diagonal shear failure despite maximum shear stresses exceeding four times the maximum code level. All walls displayed stable hysteresis loops at a displacement ductility factor of 2, and the less heavily reinforced walls (designed to approximately twice code levels) were satisfactory at DF = 4. Degradation was never catastrophic and occurred due to slip of the entire wall along the foundation beam. Methods for reducing the degradation are discussed. Confining plates did not significantly reduce the degradation of the hysteresis loops, but substantially reduced damage to the walls at high ductility factors. Values of required ductility for walls designed to the loadings code are investigated, and on the basis of these and the experimental results, recommendations are made for relaxation to the ultimate shear provisions of the masonry code.

● **6.11-21** Tonin, R. and Bies, D. A., **Time-averaged holography for the study of three-dimensional vibrations,** *Journal of Sound and Vibration,* **52**, *3,* June 8, 1977, 315–323.

The general theory of time-averaged holographic interferometry has been analytically extended to take account of possible motion of the surface under investigation in three dimensions but in simple harmonic motion at a single frequency. The argument of the characteristic function is replaced with an expression which includes the effects of motion in orthogonal directions as well as the directions of illumination and observation. The amended characteristic function formula is used to calculate the radial and tangential components of a vibrating cylinder from several time-averaged holograms. The components calculated agree well with theoretical predictions for the cylinder Love mode. Thus the problem of the strange shift in amplitude plots previously reported when holograms from different angles were taken of vibrating curved surfaces is resolved.

● **6.11-22** Tyler, R. G., **Dynamic tests on PTFE sliding layers under earthquake conditions,** *Bulletin of the New Zealand National Society for Earthquake Engineering,* **10**, *3,* Sept. 1977, 129–138.

The first part of a program to assess the characteristics of PTFE (polytetrafluoroethylene, a fluorocarbon resin, known under the trade name Teflon) sliding bearings during earthquake motions is described. Tests were performed on pure PTFE elements sliding on stainless steel. Representative normal loads were applied together with a sliding action, giving a maximum acceleration of 0.2 g and a maximum velocity of 38 cm/sec, i.e., motions equivalent to a moderate-to-severe earthquake. At 0°C, friction was found to have maximum coefficients which fell from 17 to 13% as the normal pressure was increased from 15 to 25 MN/m^2. At 20°C, corresponding values were reduced to 15 and 10%. Maximum friction generally occurred in the first cycle of loading after which values fell. After 5 cycles, the range was typically 9 to 5%. Tests on lubricated PTFE layers showed coefficients of friction less than 2% under the above test conditions; the maintenance of this low value over the years would depend on the effective retention of the grease. It is possible that, by careful design of bearings for higher pressures, and by utilizing filled PTFE materials, lower overall friction values for the dry condition may be achieved.

● **6.11-23** Robinson, W. H. and Tucker, A. G., **A lead-rubber shear damper,** *Bulletin of the New Zealand National Society for Earthquake Engineering,* **10**, *3,* Sept. 1977, 151–153.

A lead-rubber damper consisting of a steel-reinforced elastomeric bearing with a lead insert fitted in its center has been tested at earthquake-like frequencies (0.9 Hz) with a vertical load applied. For an engineering shear strain of ±0.59, the damper completed 340 cycles and operated satisfactorily at temperatures of -35 ± 5°C and 45 ± 5°C. The authors believe that the lead-rubber damper is suitable

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for installing in base-isolation systems for the protection of buildings and bridges during earthquakes.

● **6.11-24** Tyler, R. G., **Dynamic tests on laminated rubber bearings,** *Bulletin of the New Zealand National Society for Earthquake Engineering,* **10,** *3,* Sept. 1977, 143–150.

Simulated dynamic tests in shear on laminated rubber bearings are described. The purpose of the tests was to check the performance of the bearings when used as mounts for base isolating buildings against earthquake attack. Over 1000 cycles were applied to the bearings, mostly at 50% shear strain at a frequency in the range 0.2 to 0.9 Hz. Values of damping were checked by conducting separate torsional oscillation tests. It is concluded that the bearings are suitable for use in the dynamic mode for the purpose of achieving base isolation in earthquake-resistant buildings. Damping is likely to be 2 to 3% of critical. Greater values may possibly occur at faster speeds and under cold conditions. The use of separate lead or steel damping elements to prevent the buildup of damaging horizontal oscillations is considered necessary.

● **6.11-25** Plecnik, J. M. and Shipp, J. G., **Bolted joints on trans-Alaska pipeline structures,** *Journal of the Structural Division, ASCE,* **103,** *ST1,* Proc. Paper 12687, Jan. 1977, 87–104.

Results are presented from an experimental program that has been conducted for the purpose of investigating the mechanical behavior of high-strength bolted steel connections assembled with A-320 bolts under probable Alaskan temperature extremes of -55°F (-48°C) to 75°F (24°C). Extensive analyses are presented for the slip coefficients and slip behavior under cyclic loading for connections assembled with plain steel plates, corrosion-resistant painted steel plates, or galvanized steel plates. Also, examinations of plain versus galvanized bolts and the turn-of-the-nut versus calibrated torque wrench methods of insulation are presented. Based on experimental results, recommendations are presented for the optimal design of high-strength bolted steel connections for the Alaskan-type climatic environments.

● **6.11-26** Chowdhury, A. H. and White, R. N., **Materials and modeling techniques for reinforced concrete frames,** *Journal of the American Concrete Institute,* **74,** *11,* Title No. 74-50, Nov. 1977, 546–551.

The reliability of 1/10-scale reinforced concrete models for predicting frame behavior under severe reversing lateral loads is assessed with seven models of four full-scale beam-column joints. The joints were subjected to nine cycles of severe reversing loads with ductility factors of 5 for the last three load cycles.

The properties of appropriate model materials (concrete and reinforcing steel) are presented, and the effects of concrete mix proportioning and heat treatment of the reinforcing wire are described. Comparisons between model predictions and prototype results are given for moment-rotation, displacement, reinforcing stresses, and development of cracking in the beam and column. It is shown that 1/10-scale models can accurately reproduce complex prototype behavior when the model materials have the proper postelastic properties. The tensile strength of model concrete and the bond behavior of model reinforcing are key quantities.

● **6.11-27** Schwaighofer, J. and Collins, M. P., **Experimental study of the behavior of reinforced concrete coupling slabs,** *Journal of the American Concrete Institute,* **74,** *3,* Title No. 74-12, Mar. 1977, 123–127.

This paper is concerned with the response of slabs used to effect the coupling of shear walls. An experimental investigation of a one-third-scale reinforced concrete shear wall-slab model is described. Based on the results of the model, procedures for calculating the initial stiffness, the cracked stiffness, the shear strength, and the flexural strength of the coupling slabs are explained.

● **6.11-28** Paulay, T. and Spurr, D. D., **Frame-shear wall assemblies subjected to simulated seismic loading,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1195–1200.

The elastoplastic response of two simple reinforced concrete coupled frame-shear wall assemblies, subjected to reversed cyclic static loading, was studied; and the highlights of the experimental results are briefly reported. In preliminary tests, the behavior of the beam-wall junctions, where large ductilities were imposed, was studied in particular to establish the influence of the shear force on stiffness degradation and energy dissipation. Two different arrangements of the flexural reinforcement were used in these preliminary specimens. In spite of the severe displacement pattern imposed on the frame-shear wall assembly models, satisfactory response was obtained when the adverse effect of the reversed cyclic shear on the concrete of the plastic hinge zones of the beams was eliminated.

● **6.11-29** Otani, S., **Earthquake tests of shear wall-frame structures to failure,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2759–2765.

Dynamic tests of reinforced concrete structural models were conducted using the Univ. of Illinois earthquake simulator platform. The models were subjected to a series of intense base motions. This paper describes the observed behavior of shear wall-frame structures and frame structures tested to failure.

● **6.11-30** Diaz, J. A. and del Valle, E., **Dynamics laboratory of the National University of Mexico,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2843–2850.

The Inst. of Engineering, National Univ. of Mexico, has recently installed a new dynamics laboratory, the main feature of which is a 2.4 m by 2.5 m shaking table with one horizontal degree-of-freedom. The table is driven by means of a 75 t electrohydraulic actuator, electronically controlled in order to simulate different types of motions.

● **6.11-31** Okamoto, S. *et al.*, **Study of effects of a berm on the stability of rockfill dams during earthquakes,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2232–2237.

Vibration failure experiments and numerical analyses of large-sized two-dimensional models of rockfill dams are carried out to study the stability of dams of this type during earthquakes. The paper describes the results of vibration failure experiments on models of dams, each provided with a berm on one of its slopes. Also described are the results of numerical analyses of the influences of the provision of a berm on stress distribution inside a dam.

● **6.11-32** Kelly, J. M. and Tsztoo, D. F., **Energy absorbing devices in structures under earthquake loading,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1369–1375.

Several different types of energy-absorbing devices have been proposed for incorporation into structures designed to resist earthquakes. The basic concept is that energy dissipation, needed to absorb the kinetic energy imparted to a structure by earthquake loading, can be concentrated in devices specifically designed for that purpose. In a previous paper, the authors have described a particular type of device which operates on the principle of large plastic deformation of mild steel in torsion. It has been shown that the device can provide substantial energy absorption over many cycles of plastic deformation and that the lifetime of the device and its absorption capacity are not decreased by increasing strain rates or by random loading. This paper describes a series of tests in which a half-scale, three-story single-bay steel frame fitted with these torsion-type, energy-absorbing devices was subjected to simulated earthquake loading on a 20-ft-square shaking table. The results of the tests are compared to those for two other series on the same frame when the devices were not used. The test series establishes the feasibility of the energy-absorbing device concept.

● **6.11-33** Hawkins, N. M., Mitchell, D. and Symonds, D. W., **Hysteretic behavior of concrete slab to column connections,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2791–2796.

Reversed cyclic loading tests are summarized for 26 specimens modeling connections in flat plate structures. Variables included the intensity of the reversing moments transferred to the column, the extent and amount of flexural and stirrup reinforcement in the slab, and column proportions. Connections without shear reinforcement failed suddenly by punching. Connections with properly designed stirrups developed rotations eight times those for first yield of the slab reinforcement without significant loss in energy absorption with cycling. Recommendations are made for proportioning flexural and stirrup reinforcement and assessment of strength, stiffness, and energy absorption of such connections.

● **6.11-34** Fiorato, A. E., Oesterle, R. G. and Corley, W. G., **Ductility of structural walls for design of earthquake resistant buildings,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2797–2802.

Tests of nine reinforced concrete specimens representing walls for lateral bracing in earthquake-resistant buildings are described. The model walls were 4.57 m high and 1.91 m wide. Wall thicknesses were 102 mm. Specimens were subjected to in-plane horizontal reversing loads. Controlled variables included the shape of the wall cross section, the amount of main flexural reinforcement, and the amount of hoop reinforcement around the main flexural reinforcement. In addition, one wall was subjected to monotonic loading. A test of a repaired wall is also described. Results, including strength and ductility, are given for the walls.

● **6.11-35** Ward, H. S., **Experimental techniques and results for dynamic tests on structures and soils,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2810–2815.

This paper discusses the experimental techniques that are currently available to determine the dynamic characteristics of structures from full-scale tests, and the influence upon these characteristics of the impedance of supporting soils. Methods and results are presented from tests that have used static, impulsive, steady-state and random forces. The structures tested include multistory buildings, bridges, and an elevated turbine foundation in a nuclear power station.

● **6.11-36** Yamada, M., Tsuji, B. and Nakanishi, S., **Elasto-plastic behavior of braced frames under cyclic horizontal**

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loading, *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2778–2784.

To investigate the behavior of steel braced frames, cyclic horizontal loading tests are carried out using rectangular steel unit rigid frames with various kinds of bracings. Constant and incremental horizontal sway amplitude tests are conducted under the constant vertical load of the column. Deformation analysis is carried out using an equivalent model to analyze cross section of the members and the Bauschinger model to analyze the behavior of the material. The elastoplastic behavior of the braced frames including the postbuckling behavior of the bracing is clarified by the analysis. The fatigue life is investigated using a range count method. The fatigue life of the braced frame is relatively shorter than that of the unbraced frame.

● **6.11-37** Tassios, T., Plainis, P. and Vassiliou, G., **Column-beam joints failed under seismic loading, repaired and re-tested under seismic loading,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2532–2540.

Half-scale column-beam joints of several reinforced concrete frames and one beam have been tested under static and, subsequently, under seismic loading, up to failure. The mechanical behavior of these joints has been reported previously. The same failed joints have been repaired using conventional techniques. This testing of the frames followed practically the same procedure as the previous tests. Stresses of constituent materials were measured by means of a lightbeam oscillograph. Several practical and theoretical conclusions are drawn concerning the dynamic behavior of these simply repaired elements.

● **6.11-38** Klingner, R. E. and Bertero, V. V., **Infilled frames in aseismic construction,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1926–1932.

Results obtained in the experimental phase of an investigation of the effects of engineered masonry infill panels on the seismic hysteretic behavior of R/C frames are presented and evaluated. This experimental phase consists of quasistatic cyclic load tests on a series of 1/3-scale model subassemblages of the lower three stories of an 11-story, 3-bay frame with infills in the two outer bays. Emphasis is placed on simulation of the proper force and displacement boundary conditions, and on the reinforcing details required to attain ductile frame action. The engineered infilled frames offered several advantages over comparable bare frames, particularly with respect to their performance under strong ground motions.

● **6.11-39** Gyoten, Y., Mizuhata, K. and Fukusumi, T., **An investigation of mechanical reliability of shear wall repaired with epoxy mortar,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2526–2531.

Structural walls of reinforced concrete buildings subjected to severe earthquake or differential settlement sustain inclined and x-patterned shear crack damage due to large shear deformation. When the reinforcement bars are not broken and the boundary frame is still stiff, these shear walls will be able to be repaired. Therefore, the effectiveness of the restoration of the repaired wall should be clarified. This paper investigates experimentally the mechanical properties of repaired shear walls.

● **6.11-40** Plecnik, J. M. *et al.,* **Repair of earthquake damaged concrete masonry systems subjected to static and dynamic loads and elevated temperatures,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2492–2498.

This paper presents a summary of results of tests on epoxy-repaired concrete masonry components and shear walls. The test parameters include (1) low- and high-viscosity epoxy adhesives, (2) width of cracks, (3) static and simulated seismic loads, and (4) fire exposure. The test results indicate that masonry shear walls, damaged by earthquakes or other causes, can be effectively repaired with epoxy, the epoxy-repaired components being essentially restored to strength levels existing prior to sustained damage under static and dynamic load conditions. However, under fire exposure, the strength of epoxy-repaired components is substantially reduced.

● **6.11-41** Kahn, L. F. and Hanson, R. D., **Reinforced concrete shear walls for aseismic strengthening,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2499–2504.

Four one-story, one-half scale reinforced concrete frames were strengthened using different types of infilled walls. Reversed cycle tests showed that walls made using single or multiple precast panels demonstrated greater ductility than a wall cast monolithically with the surrounding frame.

● **6.11-42** Higashi, Y., Ohkubo, M. and Fujimata, K., **Behavior of reinforced concrete columns and frames strengthened by adding precast concrete walls,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2505–2510.

Inelastic behavior of reinforced concrete columns strengthened by adding sidewalls and precast concrete panels were investigated using specimens of about one-fourth scale. The methods of welding wall reinforcements

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to existing reinforcements and using special steel anchor-pieces were tried to join walls to existing members. The increasing ratios of ultimate strength after strengthening to calculated strength before strengthening were about two in the columns with added sidewalls, and were about four in the frames with precast concrete panels. Stiffness and strength were analyzed after strengthening in the frames with precast concrete panels on the basis of idealized load transfer mechanisms.

● **6.11-43** Hegemier, G. A. *et al.*, **Earthquake response and damage prediction of reinforced concrete masonry multistory buildings, Part II: selected results,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3019–3024.

Sample results from the research program outlined in Part I (Abstract No. 6.4-15) are presented in this paper. The discussion is confined to basic features of experimental data and analysis. Design recommendations are not made herein; the latter must await completion of appropriate test series, comprehensive data reduction, and interpretation. The results were obtained from specimens constructed from the following constituents: 8 x 8 x 16-in. Type N normal weight concrete block (ASTM C90), Type S mortar (ASTM C270), and 2000 psi fine grout (ASTM C476).

● **6.11-44** Sheppard, P., Tercelj, S. and Turnsek, V., **The influence of horizontally-placed reinforcement on the shear strength and ductility of masonry walls,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3000–3005.

The hypothesis that the shear strength and ductility of masonry walls can be increased by the placing of reinforcement in the horizontal mortar-joints was confirmed by dynamic cyclic loading tests on wall elements, with H-δ diagrams and hysteresis loops being obtained. It was found that shear strength increased with an increasing percentage of reinforcement; but if the entire shear-load carrying capacity of the reinforcement is to be made use of, then, in addition to the provision of suitable anchorage, the unreinforced wall must have sufficient ductility.

● **6.11-45** Gosain, N. K. and Jirsa, J. O., **Bond deterioration in reinforced concrete members under cyclic loads,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3049–3055.

The behavior of intermediate- and high-strength grade anchored bars supporting a cantilever beam is discussed. Tests showed that under cyclic loading there is a progressive reduction in the stress transfer capacity between the reinforcement and the concrete. In general, specimens reinforced with high-strength reinforcement had a more severe development problem than members with intermediate-grade reinforcement. Specimens with intermediate-grade reinforcement were generally less stiff in the inelastic region.

● **6.11-46** Shimazu, T. and Fukuhara, Y., **Experimental study on reinforced concrete truss frames as earthquake resistance elements,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3025–3030.

Twenty one-story, one-span reinforced concrete braced frames of small scale were tested. The frames were subjected to alternating repeated horizontal loads at the top beam level with constant axial load at the tops of the columns. These tests were used to obtain basic data on the earthquake-resistant capacities of reinforced concrete frames. The variables considered in this investigation are the longitudinal reinforcement ratios of the members composing a frame, the level of axial load, the slenderness of the members and the reinforcement arrangements in connections. The test results show that braced frames have considerable ductility in addition to high degrees of stiffness and strength.

● **6.11-47** Razani, R., **Investigation of lateral resistance of masonry and adobe structures by means of a tilting table,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 2130–2131.

Construction of shaking table facilities for earthquake testing in most seismically active developing countries is unfeasible because of high initial costs and the intensive need for skilled technicians for the maintenance and operation of such facilities. In these countries, there exists a need to develop a simpler laboratory tool for the study of the lateral strength of low-cost buildings, especially those of low-rise masonry and adobe type. To satisfy this need, an experimental tilting table is being developed by the author in Shiraz, Iran. In this method of testing, the model buildings will be secured on the table deck. By tilting the deck of this table, a lateral static force will be induced in the test model. Using static laws from the angle of the slope, this force can be easily computed and related to the cracks or damage observed in the test model.

● **6.11-48** Suzuki, T. and Ono, T., **An experimental study on inelastic behavior of steel members subjected to repeated loading,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3163–3168.

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The object of this paper is to explain the deformation capacity of steel members under monotonic and repeated loading. Deformation capacities of members under repeated loading are quite different from those under monotonic loading. The cause is based on the increasing lateral and torsional deflection after local buckling. The critical deformation capacity for repeated loading is lower than half the value for monotonic loading. Some data on the capacity of steel members as energy dissipators in the plastic range are given.

● **6.11-49** Umemura, H. *et al.*, **Aseismic characteristics of RC box and cylinder walls,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3144–3149.

This study was conducted to investigate the behavior of box- and cylinder-type shear walls under cyclic loading simulating earthquake forces on walls of atomic reactors and other structures. Seven specimens, five box-type and two cylinder-type, were tested. Initial stiffness, cracking strength, stiffness after cracking, load-deflection characteristics under cyclic loading, ultimate strength, mode of failure, strain distribution of the reinforcement, etc., were investigated and compared with theoretical results.

● **6.11-50** Anicic, D. and Zamolo, M., **Structural assemblage of shear wall high-rise buildings exposed to cyclic loading,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3138–3143.

The results of laboratory testing of a structural assemblage of shear wall high-rise buildings are presented. Four large RC models (scale 1:2) were tested. They represent characteristic connections between the shear wall and lintel beams in a residential building of 20 x 20 m plan dimensions and a 19 story height. Characteristic hysteresis loops, dependence of the damping coefficient on the introduced rotation, and strain distribution in the main reinforcement are given. The elements underwent bending, and shear stresses were moderate. Cyclic loading was alternated slowly with successive increases in the rotation amplitude. The analytical model representing the behavior of the assemblage was formulated so that it would be suitable for computer postelastic analysis.

● **6.11-51** Higashi, Y., Ohkubo, M. and Ohtsuka, M., **Influences of loading excursions on restoring force characteristics and failure modes of reinforced concrete columns,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3127–3132.

Presented is a summary of an experimental study on restoring force characteristics and failure modes of reinforced concrete columns subjected to constant axial and cyclic lateral loads. Four kinds of loading excursions controlled by lateral displacement were adopted for eight series of test specimens to obtain the characteristics of restoring force and failure. The following problems are discussed emphasizing behavior after yielding: (1) the relation between the failure mode and loading excursion, (2) the relation between the envelope curve and loading excursion, and (3) the relation between the equivalent viscous damping factor and loading excursion.

● **6.11-52** Blakeley, R. W. G. *et al.*, **Performance of large reinforced concrete beam-column joint units under cyclic loading,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3095–3100.

The design and static cyclic (reversed) load testing of three full-scale reinforced concrete beam-column assemblies is described. Member details were based on a frame building designed to current New Zealand practice. Extensive instrumentation enabled a detailed assessment of the performance in the joint region and indicated satisfactory behavior at deformations expected under severe seismic loading. Stable behavior was obtained up to maximum beam displacement ductility factors from 6 to 10 for the three units. Results are presented and assessed, and recommendations are made for revisions to common procedures for joint reinforcement and beam hinge detailing.

● **6.11-53** Sozen, M. A., Aristizabal, D. and Lybas, J. M., **Multi-story walls subjected to simulated earthquakes,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3221–3226.

An experimental investigation of the response to earthquake motions of reinforced concrete structural systems is in progress at the Univ. of Illinois. This report describes some results from studies of slender walls.

● **6.11-54** Omote, Y. *et al.*, **Effect of test technique on masonry shear strength,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3208–3213.

The paper compares the critical tensile strengths of concrete block masonry panels obtained by three different test techniques. Two methods consist of simple diagonal tests on 32 x 32 in. square panels. The test setup used in the third method was designed to simulate the boundary conditions that the pier would experience in a perforated shear wall of a complete building. Each test specimen of the third method was a full-scale panel about 15 feet square consisting of two piers and a top and bottom spandrel. Theoretical formulations are used in an attempt to correlate the experimental results. Good correlation of

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shear strength was achieved when an appropriate theoretical formula was used.

● **6.11-55** Mayes, R. L., Omote, Y. and Clough, R. W., **Cyclic shear tests on masonry piers,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3195-3201.

The results of cyclic in-plane shear tests on seventeen fixed-ended masonry piers are presented. The test setup is designed to simulate insofar as possible the boundary conditions that the piers would experience in a perforated shear wall of a complete building. Each test specimen was a full-scale panel about 15 feet square consisting of two piers and a top and bottom spandrel. The panels were constructed from 6 in. wide x 8 in. high x 16 in. long hollow concrete block units. The variables included in the investigation were the quantity and distribution of reinforcement, the rate of load application, the vertical bearing stress, and the effect of partial grouting. This paper discusses the effect of these parameters on the hysteresis envelopes and ductility of the piers.

● **6.11-56** Tercelj, S., Sheppard, P. and Turnsek, V., **The influence of frequency on the shear strength and ductility of masonry walls in dynamic loading tests,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2992-2999.

Horizontal cyclic-loading tests at various frequencies up to 5 c/sec were carried out on a series of wall elements made of hollow building blocks with expanded-clay aggregate. H-δ diagrams and hysteresis loops were obtained. It was found that the wall's dynamic shear strength increased with frequency. Neither the initial stiffness of the wall, its displacement at maximum load, nor its ductility were significantly affected by frequency change.

● **6.11-57** Mizuhata, K., Gyoten, Y. and Kitamura, H., **Study on low cycle fatigue of structural frames due to randomly varying load,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3031-3036.

In this study, constant and random displacement amplitude low-cycle fatigue tests were conducted for steel frames used in tall residential buildings. After formulating the restoring force characteristics, realistic earthquake response analyses were conducted for 8- to 28-story buildings, and then several random fatigue theories were applied to the results. It has been concluded that the frames fail because of propagation of cracks initiated at the flange welds of the girder ends, and that the maximum ductility factor cannot be a sure measure of damage.

● **6.11-58** Sharda, S. C., Arya, A. S. and Prakash, S., **Tests on well foundation models under horizontal dynamic loads,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1615-1620.

Free vibration tests and cyclic and repetitive lateral load tests were conducted on 15-cm and 20-cm square-section laboratory models, the main variables being magnitude of vertical load, depth of embedment, and magnitude of skin friction on sides. A field model of square base with 1.5-m sides was also similarly tested. The frequency and damping values as influenced by the various parameters are presented.

● **6.11-59** Kvitsaridze, O. I. *et al.*, **Experimental study of prestressed reinforced concrete elements of antiseismic buildings,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3202-3207.

Various institutions have conducted experiments to determine the characteristics of prestressed elements of reinforced concrete structures with different reinforcements and different degrees of prestressing. Attention also has been given to changes in the natural vibration frequency and the logarithmic decrement in the consideration of the results. The energy absorbing capacity of the models has depended upon the cohesiveness of the stressed reinforcement with the concrete. On the basis of experimental results, frames for public buildings have been developed; and further design recommendations for prestressed reinforced concrete structures for seismic regions are being prepared.

● **6.11-60** Rea, D., Abedi-Hayati, S. and Takahashi, Y., **Dynamic analysis of electrohydraulic shaking tables,** *UCB/EERC-77/29,* Earthquake Engineering Research Center, Univ. of California, Berkeley, Dec. 1977, 65. (NTIS Accession No. PB 282 569)

The frequency response characteristics of two shaking tables have been determined experimentally. The lighter table, weighing 2,000 lb (900 kg), was used primarily to determine the effects of a resonant structure on a shaking table's frequency response. The heavier table, weighing 100,000 lb (45,300 kg), was used primarily to determine the effects of foundation compliance on a shaking table's frequency response. Mathematical models were formulated for both tables, and the models were refined by adjusting parameters to obtain the best correspondence between the computed and experimental frequency responses. The mathematical models were then used to study the effects of a resonant structure and of foundation compliance on the frequency responses of shaking tables and on the ability of shaking tables to reproduce earthquake-type motions.

It was found that the magnitudes of the peak and notch distortions in the frequency response of a shaking table are sensitive to the amount of force feedback employed by the control system. In addition, the magnitudes depend on the ratio of the mass of the structure to the mass of the shaking table and to the transmissibility function of the structure with respect to the table. Although the peak and notch effect may cause difficulties in determining the frequency response of structures by means of shaking tables, it has little effect on the accuracy to which a shaking table can reproduce earthquake-type motions. It was found that foundation compliance affects the frequency response of a shaking table only at low frequencies, and the magnitude of the effect is limited to an amount which depends on the transmissibility function of the foundation with respect to the table.

● **6.11-61** Kelly, J. M. and Tsztoo, D. F., **Earthquake simulation testing of a stepping frame with energy-absorbing devices,** *UCB/EERC-77/17,* Earthquake Engineering Research Center, Univ. of California, Berkeley, Aug. 1977, 53. (NTIS Accession No. PB 273 506)

Results are reported of earthquake simulation tests on a model frame with a partial base isolation system that includes energy-absorbing devices. The isolation system was modeled on a stepping bridge concept developed for New Zealand railways, and the energy-absorbing devices, based on the plastic torsion of rectangular mild steel bars, functioned only when the frame base lifted off the foundation. Two series of tests, using scaled accelerations from the El Centro N-S 1940 and Pacoima Dam 1971 earthquake ground motion records were used as input to the shaking table on which the tests were performed. Results from these tests are compared to those from earlier tests on an identical frame with the foundation (1) anchored as in conventional design, and (2) permitted to uplift freely. The response of the frame with the energy-absorbing devices was improved over that of both the fixed frame and the frame allowed to uplift freely for the El Centro accelerations. Although the results are not as favorable for the Pacoima Dam input, the feasibility of using the energy-absorbing devices associated with a partial base isolation system is established as an alternative to anchored frames and frames allowed to uplift freely.

● **6.11-62** Mihai, C. *et al.*, **On the static and seismic behaviour of a structural model of an industrial storied hall,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2735–2740.

The Romanian Ministry of Industrial Construction, in collaboration with the Polytechnic Inst. of Jassy, has been conducting research on multistory industrial buildings. Static and dynamic testing of a model was performed on the 140-ft shaking table at Jassy. The possibility of using lightweight concrete in some members of industrial structures is considered.

6.11-63 Tang, C. T. and McConnell, K. G., **A mathematical model of Hysol 8705 under impact loading,** *Experimental Mechanics,* **17,** *3,* Mar. 1977, 113–119.

The mechanical properties of Hysol 8705, a low-modulus viscoelastic material, were investigated experimentally under impact loading. The experimental results reveal that the material behavior under investigation is nonlinear. Empirical relationships between various important parameters, which characterize the mechanical properties of the material, were established. A three-parameter solid type of mathematical model consisting of a spring and Kelvin element in series was found to satisfactorily reproduce the experimental impact-property data of this material. The spring and dashpot elements were found to be nonlinear and strain-rate dependent.

6.11-64 Park, R. and Thompson, K. J., **Cyclic load tests on prestressed and partially prestressed beam-column joints,** *Journal of the Prestressed Concrete Institute,* **22,** *5,* Sept.-Oct. 1977, 84–110.

Tests were conducted on ten concrete beam-interior column frame assemblies subjected to static cyclic loading simulating the effect of severe earthquake loading. The frame members were nearly full-scale and contained a range of proportions of prestressed steel and nonprestressed steel. Information applicable to seismic design concerning inelastic flexural deformation capacity and degree of damage was obtained.

The behavior of the frames emphasized the need for transverse steel in the plastic hinge zones of flexural members and in beam-column joint cores to ensure ductile behavior and to avoid diagonal tension failure. The ductility of prestressed beams was enhanced by the presence of nonprestressed reinforcing steel in the compression zones of members. A central prestressing tendon at mid-depth in the beam passing through the joint was shown to be effective in contributing to joint core shear strength. Several design recommendations are proposed.

● **6.11-65** **Feasibility study on design and construction of foundation of large scale shaking table–analytical study** (in Japanese), Shimizu Construction Co., Tokyo, 1976, 147.

● **6.11-66** Ravara, A., Mayorga, A. and Carvalho, C., **Seismic tests of infilled reinforced concrete frames,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2772–2777.

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The purpose of this study is to investigate the earthquake performance of traditional low-cost housing structures composed of brick infilled reinforced concrete frames. The research program involves shaking table testing of eight models of three-story buildings scaled 1:4. The models are subjected to unidirectional horizontal movements. The models differed only in the geometry of the door and window openings and in the bracing around the openings, consisting of vertical and/or horizontal lintels. Each model was subjected to increasingly strong ground motions until different patterns of damage in the walls or in the frames occurred. Some of the models were then repaired and tested again until they underwent all probable seismic actions likely to occur in 50 years. The costs of construction and repair of the prototypes are estimates. Although this research program is in progress, preliminary results do indicate the most economical and efficient techniques for the confinement of the brick walls.

● **6.11-67** Umemura, H. *et al.*, **Experimental study on reinforced concrete columns with double spiral web reinforcements,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3113–3119.

This paper discusses experimental studies performed on reinforced concrete columns with double-spiral web reinforcements wound and welded on longitudinal bars. Thirty columns were tested under repeated alternating asymmetric and constant axial loads. The variables considered are shear span ratio, axial load level, the amount and diameter of web reinforcement, the angle between web and longitudinal reinforcements, and the variance of welding between the reinforcements. The tests reveal that double-spiral web reinforcements are more effective for reinforced concrete columns than conventional web reinforcements.

● **6.11-68** Focardi, F., **Experiments on the spatial behavior of plastic frames,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2785–2790.

Experimental tests of the plastic response of one-story steel frames subjected to horizontal static loads were conducted. The dynamic response of reinforced concrete plane frames of a building under demolition, and thereby subjected to forced vibrations, was also tested. In the testing of frames, asymmetry in the ground plan of the columns involves rotation and displacement normal to the load direction. Relationships between these movements, load direction, and structural geometry are discussed. In the dynamic tests, it was found that the brick wall panel can collapse without cracking as a result of separation from the concrete structure.

6.12 Deterministic Methods of Dynamic Analysis

● **6.12-1** Abdel-Ghaffar, A. M., **Dynamic analyses of suspension bridge structures,** *EERL 76-01,* Earthquake Engineering Research Lab., California Inst. of Technology, Pasadena, May 1976, 362.

A method of dynamic analysis for vertical, torsional, and lateral free vibrations of suspension bridges has been developed that is based on linearized theory and the finite element approach. The method involves two distinct steps: (1) specification of the potential and kinetic energies of the vibrating members of the continuous structure, leading to derivation of the equations of motion by Hamilton's principle, and (2) use of the finite element technique to (a) discretize the structure into equivalent systems of finite elements, (b) select the displacement model most closely approximating the real case, (c) derive element and assemblage stiffness and inertia properties, and (d) form the matrix equations of motion and the resulting eigenvalue problems. The stiffness and inertia properties are evaluated by expressing the potential and kinetic energies of the element (or the assemblage) in terms of nodal displacements. Detailed numerical examples are presented to illustrate the applicability and effectiveness of the analysis and to investigate the dynamic characteristics of suspension bridges with widely different properties. This method eliminates the need to solve transcendental frequency equations, simplifies the determination of the energy stored in different members of the bridge, and represents a simple, fast, and accurate tool for calculating the natural frequencies and modes of vibration by means of a digital computer. The method is illustrated by calculating the modes and frequencies of a bridge and comparing them with the measured frequencies.

6.12-2 Chon, C. T. and Martin, J. B., **A rationalization of mode approximations for dynamically loaded rigid-plastic structures based on a simple model,** *Journal of Structural Mechanics,* 4, *1,* 1976, 1–31.

A simple two degree-of-freedom viscoplastic beam is considered. The dynamic response of the model to impulsive and constant loading is plotted in the space in a manner which illustrates methods by which the response can be approximated. The generalization of the results to continuous structures is discussed.

● **6.12-3** Clough, R. W., **Predicting the earthquake response of reinforced concrete structures,** *Reinforced Concrete Structures in Seismic Zones, SP-53,* American Concrete Inst., Detroit, 1977, 59–79.

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Formulation of the dynamic response equations for structures subjected to earthquakes is summarized, and techniques to account for the nonlinear cyclic loading behavior of reinforced concrete are mentioned. The need for additional data concerning the actual performance of concrete structures subjected to earthquake actions is emphasized, and the test of a two-story R/C frame on the Univ. of California Earthquake Simulator Facility is described. Selected results of the test are presented, and the problem of obtaining similar results by computer analysis is discussed. The correlation obtained using a bilinear frame analysis program including a stiffness degradation mechanism is demonstrated to be reasonably good.

6.12-4 Hure, D. and Morysse, M., **Comparative methods for analysis of piping systems subjected to seismic motion,** *Nuclear Engineering and Design,* 38, 3, Sept. 1976, 511-525. (For an additional source, see Abstract No. 1.2-2.)

The dynamic analysis of a three-dimensional piping system of a nuclear power plant is conveniently performed with a finite element method. When the modal analysis is used, only the first few modes of vibration are computed for practical purposes. In this paper a method of residues is proposed which evaluates the neglected modes and combines them with the first calculated modes to estimate the total seismic response of the piping. This method emphasizes the importance of the selected modes. When the approach is made through a time history input function, this latter is usually characterized by a combination of several recorded accelerograms, e.g. El Centro, San Francisco and Taft. The response of a particular piping system has been evaluated by means of these two methods. The use of the modal approach is strongly recommended due to its inherent advantage of economy and also computation time and reliability.

6.12-5 Villasor, Jr., A. P., **Seismic analysis of a reactor coolant pump by the response spectrum method,** *Nuclear Engineering and Design,* 38, 3, Sept. 1976, 527-542. (For an additional source, see Abstract No. 1.2-2.)

Current nuclear steam supply systems are designed to remove the heat of fission by circulating coolant in closed loops from the reactor. For water reactors, this prime function is designated to the reactor coolant pump (RCP). The Westinghouse Type 93A RCP is analyzed for seismic response. Briefly described, this RCP is a vertical, single-stage, centrifugal pump designed to move 90,000 gpm ($568 m^3/sec$) of water and driven by a 6000 hp motor for use in the PWR primary system. The RCP assembly is generally axisymmetric and is modeled using three-dimensional finite elements of the types normally found in general-purpose computer programs such as ANSYS or NASTRAN. The structural frame and the rotating shaft are the principal branches of the model. Each consists of a series of pipe elements complemented by mass elements. Orthogonal sets of linear spring elements connect the branches at the bearings and possibly at each labyrinth. Fluid elements are added to include the interaction between the shaft and the pump case through the intervening water mass. Beam elements are used to account for unsymmetry of the motor stand. To complete the model, stiffness matrix elements representing the support structure and the neighboring loop piping are attached. It is impractical to idealize faithfully each geometric irregularity. Several adjacent sections are combined into one suitable element with total stiffness and mass equivalence. The number of elements in the model is thus minimized. Shear deflection of the pipe elements is considered; mass and mass inertia are lumped at nodal points, as needed, to compensate for the actual material distribution. The RCP model contains 82 nodes, 155 elements and 140 master dynamic degrees-of-freedom. A modal frequency analysis is first run to identify the mode shapes.

The seismic analysis is performed by the response spectrum method in ANSYS, with seismic velocity as the input excitation parameter. The model is excited by a set of three orthogonal spectra. For each load excitation, the modal displacements, forces and moments are computed at each node. A post-run subroutine calculates the absolute sum of nodal response quantities at each mode for one horizontal and the vertical seismic excitations. The resultant modal values are then combined using the square root of the sum of the squares to record the final values. Nodal stresses are computed; absolute displacements are reviewed for selected nodes along the model branches. The relative displacements at bearings and labyrinths are determined. Finally, the accelerations of nodes previously chosen are found.

This paper assesses the effects of a given seismic excitation on the overall structural integrity of an RCP. The in-depth analysis has found the RCP adequate to withstand the imposed seismic loading. All component stresses are within the applicable faulted criteria and the relative movements between closely mated parts fall inside their nominal clearance limits.

6.12-6 Penzes, L. E., **Seismic model of the gas cooled fast breeder reactor core support structure,** *Nuclear Engineering and Design,* 38, 3, Sept. 1976, 543-554. (For an additional source, see Abstract No. 1.2-2.)

The modeling technique for the seismic analysis of the core support structure of a gas-cooled fast breeder reactor (GCFR) is developed. The core support structure consists of the support cylinder and a perforated grid plate to which 265 fuel and blanket elements are clamped as cantilevered beams. The analysis of the core support structure consists of three steps: (1) analysis of the grid plate, (2) analysis of the core elements, and (3) modal synthesis.

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The first step in developing a solution to the problem is to assume that the core elements (fuel and blanket) are attached to the grid plate as rigid rods. In this case the influence of the rigid rods can be represented by their masses and rotary inertias. The solution of this problem was developed by applying the dynamic theory of grid plates. This was accomplished by generalizing the Reissner-Mindlin thick-plate theory with orthotropic constants and then modifying the formulations of the rotary inertia expressions to include the rotary inertia effects of the core elements. The numerical results showed that the grid plate's fundamental frequency is in the range of the fundamental frequencies of the core elements so that a dynamic coupling effect exists. Because of this dynamic coupling effect, the elastic properties of the grid plate must be included in the seismic analysis of the GCFR. The second step was to develop a mathematical model of the grid-plate core-element system using the method of Rayleigh-Ritz. In this model the elastic coupling effect of the core elements was included.

For the final application of the theory, the exact solution of the elastic plate with rigid rods was simulated on the computer by applying the elastic rotary inertias of the core elements to the model of the grid plate. With this technique, it is possible to model the grid plate with a reasonable number of fuel and blanket elements and to replace the missing core elements with their equivalent effective rotary inertias. The method includes the capability of modeling the different mass, damping and elastic properties of the fuel and blanket elements.

Comparing the results of the present analysis with the preliminary simple spring-mass core model, the amplitudes of vibration obtained, in some cases in the present analysis, are a factor of ten smaller than was previously computed. Applying this more elaborate analysis will lead to a simpler and less expensive design.

6.12-7 Morrone, A., Nahavandi, A. N. and Brussalis, W. G., **Scram and nonlinear reactor system seismic analysis for a liquid metal fast reactor,** *Nuclear Engineering and Design,* 38, *3,* Sept. 1976, 555-566. (For an additional source, see Abstract No. 1.2-2.)

Earthquake vibrations cause large forces and stresses that can significantly increase the scram time required for safe shutdown of a nuclear reactor. The horizontal deflections of the reactor system components cause impact between the control rods and their guide tubes and ducts. The resulting frictional forces, in addition to other operational forces, delay the travel time of the control rods. To obtain seismic responses of the various reactor system components (for which a linear response spectrum analysis is considered inadequate) and to predict the control rod drop time, a nonlinear seismic time history analysis is required. Nonlinearities occur due to the clearances or gaps between various components. When the relative motion of adjacent components is large enough to close the gaps, impact takes place with large impact accelerations and forces.

This paper presents the analysis and results for a liquid metal fast reactor system which was analyzed for both scram times and seismic responses such as bending moments, accelerations and forces. The reactor system was represented with a one-dimensional nonlinear mathematical model with two degrees-of-freedom per node (translational and rotational). The model was developed to incorporate as many reactor components as possible without exceeding computer limitations. It consists of 12 reactor components with a total of 71 nodes, 69 beam and pin-jointed elements and 27 gap elements. The gap elements were defined by their clearances, impact spring constants and impact damping constants based on a 50% coefficient of restitution.

The horizontal excitation input to the model was the response of the containment building at the location of the reactor vessel supports. It consists of a 10 sec safe shutdown earthquake acceleration-time history at 0.005 sec intervals and with a maximum acceleration of 0.408 g. The analysis was performed with two Westinghouse special purpose computer programs. The first program calculated the reactor system seismic responses and stored the impact forces on tape. The impact forces on the control rod driveline were converted into vertical frictional forces by multiplying them by a coefficient of friction, and then these were used by the second program for the scram time determination.

The results give time history plots of various seismic responses, and plots of scram times as a function of control rod travel distance for the most critical scram initiation times. The total scram time considering the effects of the earthquake was still acceptable but about four times longer than that calculated without the earthquake. The bending moment and shear force responses were used as input for the structural analysis (stresses, deflections, fatigue) of the various components, in combination with the other applicable loading conditions.

● 6.12-8 Werner, S. D., **Effects of traveling seismic waves on structure response: A research program,** *Proceedings of U.S.-Japan Seminar on Earthquake Engineering Research with Emphasis on Lifeline Systems,* Japan Society for the Promotion of Earthquake Engineering, Tokyo, Nov. 1976, 43-57.

Although not usually considered in most design applications, the spatial variation of seismic waves propagating across the length of a structure may, under certain conditions, have an important effect on the structural response. This paper describes a research program that has been initiated to investigate traveling-wave effects on structural

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response, with emphasis on bridges, tunnels, and pipelines—key elements of lifeline systems.

● **6.12-9** Goto, H., Iemura, H. and Nakata, T., **Aseismic displacement requirements at highway bridge girder supports,** *Proceedings of U.S.-Japan Seminar on Earthquake Engineering Research with Emphasis on Lifeline Systems,* Japan Society for the Promotion of Earthquake Engineering, Tokyo, Nov. 1976, 105–118.

In this study, the earthquake response of highway bridges with simply supported girders is analyzed to furnish useful information on aseismic displacement requirements at sliding hinges. Vibrational models of highway bridges are proposed by assuming the rocking motion of the piers and swaying motion of the girders. The earthquake response is calculated. Effects of the natural frequency of the piers, the mass ratio between the girders and piers, the frictional force at the sliding hinge, the phase delay of earthquake motion at the piers, and the nonlinear hysteretic restoring force of the supporting ground to the maximum relative displacement between the girders and piers are examined. It is concluded that the maximum relative displacement of linear and nonlinear hysteretic models can be predicted reasonably well from displacement response spectra. However, when nonlinearity of the supporting ground is strong, the effects of gravity acting on the girders may cause complete failure of a bridge system.

● **6.12-10** Iwan, W. D., **The earthquake design and analysis of equipment isolation systems,** *Proceedings of U.S.-Japan Seminar on Earthquake Engineering Research with Emphasis on Lifeline Systems,* Japan Society for the Promotion of Earthquake Engineering, Tokyo, Nov. 1976, 271–286.

A method is presented whereby the response spectrum may be used to predict the response of equipment mounted on an isolation system with nonlinear motion limiting constraints. The results of the approximate method are compared with results obtained by direct numerical integration for a representative piece of equipment.

● **6.12-11** Yamada, M. and Kawamura, H., **Resonance capacity criterion for evaluation of the aseismic capacity of reinforced concrete structures,** *Reinforced Concrete Structures in Seismic Zones, SP-53,* American Concrete Inst., Detroit, 1977, 81–108.

It is proposed that the aseismic capacity of a reinforced concrete structure is the steady state resonance capacity for an idealized one-mass model of that structure subjected to a sinusoidal forced vibration. That resonance capacity is determined using the known characteristics of the hysteresis loops and the stability in strength with cycling of typical reinforced concrete elements. For this evaluation reinforced concrete members are classified, according to their modes of deformation and failure under monotonic loading, into three types of aseismic elements: long columns, short columns, and shear walls. Structures are divided into two types: rigid and flexible. A diagram is developed showing the interaction surface in three-dimensional space between shear force amplitude, equivalent viscous damping, and the natural period of vibration. The range of aseismic characteristics for the different elements and structures are shown on that diagram. Response acceleration spectra for typical earthquake inputs are collated with the response indicated by that diagram. That interaction surface is shown to represent a reasonable basis for estimating the aseismic safety of reinforced concrete structures.

● **6.12-12** Hassan, F. M. and Hawkins, N. M., **Prediction of the seismic loading anchorage characteristics of reinforced bars,** *Reinforced Concrete Structures in Seismic Zones, SP-53,* American Concrete Inst., Detroit, 1977, 417–438.

A mathematical model is developed for predicting the relationship between the axial force on an anchored reinforcing bar and the displacement of the loaded end of that bar relative to the surrounding concrete. The model has different forms for monotonic and reversed cyclic loading. For reversed cyclic loading, the model takes account of yielding of the reinforcing steel, Bauschinger effects, the nonlinear nature of the bond stress distribution and the modification of that distribution caused by loading reversals.

The essential characteristics of the model are established from the results of 13 stub-cantilever tests on specimens containing a single No. 10 bar and reported in a companion paper. It is shown that in addition to correctly predicting the load-slip relationships for those stub-cantilever tests the model also predicts reasonably well the load-slip relationships obtained in stub-cantilever tests on specimens loaded monotonically to failure at the Univ. of West Virginia and the first inelastic cycle of the moment-rotation relationships of beam-column specimens tested at the Portland Cement Assn.

● **6.12-13** Nikiporets, G. L., **Utilization of response spectra for calculation of the earthquake response of structures extended over large areas (Ispolsovanie spektrov reaktsii dlya rascheta protyazhennykh v plane sooruzhenii na seismicheskie vozdeistviya, in Russian),** *Seismostoikoe stroitelstvo, 2,* 1977, 5–9.

The Kazakh Industrial Construction Research Inst. has formulated a practical method for calculating the earthquake response of structures extending over large areas. A short description of the method is given and a numerical example is presented.

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● **6.12-14** Nikiporets, G. L., **Seismic response spectra of structures extending over large areas** (Spektry reaktsii protyazhennykh v plane sooruzhenii pri seismicheskikh vozdeistviyakh, in Russian), *Seismostoikoe stroitelstvo, 1,* 1977, 22–26.

The behavior of structures extending over large areas and subjected to seismic excitations was investigated. Spectra of actual earthquake records were utilized. Structures of this type are distinguished by the presence of torsional vibrations, complex forms of excitation, the effects of seismic wave propagation velocity, and variations of stiffness parameters in the structure. The analytical model of the structures is an absolutely rigid beam on an elastic base with two degrees-of-freedom.

● **6.12-15** Ievenko, V. G., **Application of the finite element method to calculate the effects of crack formation and propagation on the behavior of brick walls** (Primenenie metoda konechnykh elementov pri raschete kamennykh sten s uchetom poyavleniya i razvitya treshchin, in Russian), *Seismostoikoe stroitelstvo, 1,* 1977, 9–15.

An algorithm is developed to calculate the effects of crack formation and propagation on the behavior of brick walls subjected to horizontal and vertical seismic loads. The effect of stresses on the variation of the stiffness of the wall is investigated. The effects of reinforced concrete cores are analyzed. Numerical examples are given.

● **6.12-16** Gould, P. L., Suryoutomo, H. and Sen, S. K., **Stresses in column-supported hyperboloidal shells subject to seismic loading,** *Earthquake Engineering and Structural Dynamics,* 5, *1,* Jan.-Mar. 1977, 3-14.

The computation of stresses within a finite element displacement method analysis of a shell of revolution is considered. The common procedure of applying the kinematic and constitutive laws to the displacement functions is examined and justified for models where the displacements are represented by high-order polynomial expansions. Two alternative computational formats within this technique are also explored.

The influence of the column-supported base condition on a hyperboloidal shell of revolution is studied with respect to the stresses calculated from a response spectrum analysis. These studies emphasize the importance of accurately modeling the base region of a column-supported shell such as a hyperbolic cooling tower.

● **6.12-17** Rutenberg, A., Tso, W. K. and Heidebrecht, A. C., **Dynamic properties of asymmetric wall-frame structures,** *Earthquake Engineering and Structural Dynamics,* **5**, *1,* Jan.-Mar. 1977, 41-51.

An approximate method is proposed for evaluation of the natural frequencies and mode shapes of uniform asymmetric wall-frame structures. An exact solution is first given for the case in which the coefficient matrix of the dynamic equilibrium equations satisfies certain conditions. Using perturbation analysis, the method is then applied to the more general case in which these conditions are only approximately satisfied. A numerical example is presented to illustrate the technique.

● **6.12-18** Emery, A. F. and Cupps, F. J., **Finite difference computation of the dynamic motion of cylindrical shells including the effect of rotatory inertia and transverse shear,** *Earthquake Engineering and Structural Dynamics,* **5**, *4,* Oct.-Dec. 1977, 323-335.

The accuracy of finite difference computations for the dynamic motion of cylindrical shells, including transverse shear and rotatory inertia, has been assessed by comparison with Fourier series solutions. The finite difference models were based upon either the differential equations or upon control volume concepts, with the results of the latter being less sensitive to mesh size and in better agreement with the Fourier series results. A frequency analysis of all the finite difference algorithms and the Fourier series demonstrated that for shells with clamped ends the finite difference spectra had a very considerable gap which closed slowly as the mesh size decreased, with the spectra being most complete for the best algorithm. A new algorithm was based upon these spectra and is shown to yield good results while permitting increases in the computational speed by the factor of mesh size/shell thickness.

● **6.12-19** Hilber, H. M., Hughes, T. J. R. and Taylor, R. L., **Improved numerical dissipation for time integration algorithms in structural dynamics,** *Earthquake Engineering and Structural Dynamics,* **5**, *3,* July-Sept. 1977, 283-292.

A new family of unconditionally stable one-step methods for the direct integration of the equations of structural dynamics is introduced and is shown to possess improved algorithmic damping properties which can be continuously controlled. The new methods are compared with members of the Newmark family, and the Houbolt and Wilson methods.

● **6.12-20** Warburton, G. B. and Soni, S. R., **Errors in response calculations for non-classically damped structures,** *Earthquake Engineering and Structural Dynamics,* **5**, *4,* Oct.-Dec. 1977, 365-376.

The classical normal mode method of determining response is extremely useful for practical calculations, but depends upon the damping matrix being orthogonal with respect to the modal vectors. Approximations that allow the method to be used when this condition is not satisfied have been suggested. The simplest approach is to neglect off-diagonal terms in the triple matrix product formed from the damping and modal matrices. In this paper, the errors in response caused by this approximation are determined for several simple structures for a wide range of damping parameters and different types of excitation. Based on these results, a criterion, relating modal damping and natural frequencies, is formulated. If this is satisfied, the errors in response caused by this diagonalization procedure are within acceptable limits.

●**6.12-21** Kubo, K., **Most important factors for aseismic characteristics of bridges,** *Proceedings of U.S.-Japan Seminar on Earthquake Engineering Research with Emphasis on Lifeline Systems,* Japan Society for the Promotion of Earthquake Engineering, Tokyo, Nov. 1976, 93–103.

The seismic strength of an existing bridge is usually determined using an equivalent static force method, the response spectrum method, or dynamic response analysis techniques. However, analyses using these methods are time consuming and not practical when many bridges are examined. The author presents a simplified method for evaluating the seismic resistance of bridges.

●**6.12-22** Belytschko, T., **Comparative evaluation of numerical methods for dynamic structural analysis,** *International Seminar on Extreme Load Conditions and Limit Analysis Procedures for Structural Reactor Safeguards and Containment Structures,* T 1/1, 17. (For a full bibliographic citation, see Abstract No. 1.2-2.)

The safety analysis of reactors has brought about a need for a large variety of catastrophic event simulations to assess the strength limits of reactor components in extreme loading situations. Problems included in this category are energy release events of various scales, transients due to pressure releases in the primary piping, shock analysis for airborne missiles and conventional weapons, and seismic analysis. Except in seismic analysis, economical reasons dictate that the strength of the material and the geometrically nonlinear domain be considered. Hence, there is often a need for completely nonlinear transient analysis.

In the U.S., several general purpose programs, such as MARC, ANSYS, and PISCES, and several special purpose programs, such as REXCO, DYNAPLAS, NONSAP, and WHAM (STRAW), have been employed for these problems. The methods of integration found in these programs will be described, particularly from the viewpoint of computational efficiency for the various classes of problems that are found in reactor safety analysis. The methods of time integration, in the usual manner, are subdivided into two categories: explicit and implicit. These methods will be summarized, along with their advantages and disadvantages and some recent results on the stability and accuracy of these methods. Also, customary practice in the use of these methods will be outlined.

Most of the available programs are limited to explicit or implicit integration. It is shown that the use of an integration method inappropriate to the problem can lead to hopelessly uneconomical computations and that the analyst should select a program suitable for his problem. In addition, the accuracy of these methods in nonlinear problems is poorly understood, so devices such as energy balance checks and residual force iteration in implicit methods are highly desirable.

Finally, some procedures that would be very useful in the solution of reactor technology problems are summarized. These include: (1) mesh and time partitions by which parts of the problem can be integrated implicitly and partly explicitly; (2) estimates for temporal discretization errors, including those due to approximations of the inertial response and those due to deviations from the stress history of the material; (3) apriori estimates on time steps for implicit integration procedures.

●**6.12-23** Mukherjee, S., **Matrix of transmission in structural dynamics,** *International Seminar on Extreme Load Conditions and Limit Analysis Procedures for Structural Reactor Safeguards and Containment Structures,* T 1/2, 18. (For a full bibliographic citation, see Abstract No. 1.2-2.)

In this paper, presented are a general outline and theoretical formulation of the transfer matrix method and the application of it to actual problems in structural dynamics related to seismic analysis. The natural frequencies of a freely vibrating elastic system can be found by applying proper end conditions. The end conditions will yield the frequency determinate to zero. By using a suitable numerical method, the natural frequencies and mode shapes are determined by making a frequency sweep within the range of interest. Results of an analysis of a typical nuclear building using this method show very close agreement with results obtained by using the ASKA and SAP IV programs. The method presented would be of interest to designers of nuclear power plants.

●**6.12-24** Nelson, F. C. and Greif, R., **On the incorporation of damping in large, general-purpose computer programs,** *International Seminar on Extreme Load Conditions and Limit Analysis Procedures for Structural Reactor Safeguards and Containment Structures,* T 1/4, 10. (For a full bibliographic citation, see Abstract No. 1.2-2.)

Damping plays an important role in the dynamic response of structures and equipment subjected to broadband excitation such as that caused by earthquakes. The incorporation of damping into general-purpose shock and vibration computer programs has been based on either a concern for the physical mechanisms of damping or on the

● *See* ***Preface,*** *page v, for availability of publications marked with dot.*

need for mathematical convenience. The various approaches will be compared; several large, general-purpose computer programs will be discussed; and suggestions are made for future work.

● **6.12-25** Svalbonas, V., **Transient dynamic and inelastic analysis of shells of revolution—a survey of programs,** *International Seminar on Extreme Load Conditions and Limit Analysis Procedures for Structural Reactor Safeguards and Containment Structures,* T 2/3, 25. (For a full bibliographic citation, see Abstract No. 1.2-2.)

Advances in the limits of structural use in the aerospace and nuclear power industries over the past years have increased the requirements upon the applicable analytical computer programs to include accurate capabilities for inelastic and transient dynamic analyses. In many minds, however, this advanced capability is unequivocally linked with the large scale, general purpose finite element programs. This idea is also combined with the view that such analyses are prohibitively expensive and should be relegated to the "last resort" classification. While this, in the general sense, may indeed be the case, if, however, the user needs only to analyze structures falling into limited categories, he may find that a variety of smaller special purpose programs are available, which do not put an undue strain upon his resources. One such structural category is shells of revolution.

This survey of programs will concentrate upon the analytical tools which have been developed predominantly for shells of revolution. The survey will be subdivided into three parts: (a) consideration of programs for transient dynamic analysis, (b) consideration of programs for inelastic analysis, and (c) consideration of programs capable of dynamic plasticity analysis. In each part, programs based upon finite difference, finite element, and numerical integration methods will be considered. The programs will be compared on the basis of analytical capabilities, and ease of idealization and use. In each part of the survey, sample problems will be utilized to exemplify the state-of-the-art.

● **6.12-26** Donea, J., Giuliani, S. and Halleux, J. P., **Prediction of the nonlinear dynamic response of structural components using finite elements,** *International Seminar on Extreme Load Conditions and Limit Analysis Procedures for Structural Reactor Safeguards and Containment Structures,* T 2/4, 19. (For a full bibliographic citation, see Abstract No. 1.2-2.)

The ability to predict the nonlinear dynamic response of complex structural components subjected to time and space dependent loads is a problem of considerable importance in the field of fast-reactor safety. The finite element method is ideally suited to the analysis of such problems since it can account for arbitrary geometries, loadings and material property variations. This method has been chosen for the development of the computer programs EURDYN for the transient dynamic analysis of large-displacement, small-strain problems with material nonlinearities. Since the EURDYN programs were conceived as tools for numerical investigations in the field of fast-reactor safety, their main options were dictated by the type of structural behavior and by the nature of loading encountered in this context. The EURDYN programs are described.

● **6.12-27** Stangenberg, F., **Dynamic ultimate load analysis using a finite difference method,** *International Seminar on Extreme Load Conditions and Limit Analysis Procedures for Structural Reactor Safeguards and Containment Structures,* T 3/3, 16. (For a full bibliographic citation, see Abstract No. 1.2-2.)

The method of numerical integration is used to evaluate a one degree-of-freedom system. Then, a generalization to systems with several degrees-of-freedom is given. The conditions for numerical stability and for achieving a sufficient approximation to the exact solution of the differential equations are dealt with. Not only a time discretization but also a geometric discretization is necessary. This may be anticipated by a lumped-mass dynamic model; or, with continuous bodies, it could be performed by a mesh pattern of finite coordinate differences. Examples are given for the numerical treatment of beams and plates. Using differential equations for the wave propagation process, the rotational inertia of single beam or plate elements as well as the transverse shear deformations are included.

By use of this numerical method of dynamic analysis which is suitable for computer programming, point-by-point time-history solutions are obtained for deterministic excitations and for material properties, both varying arbitrarily with time and space. Applications for practical dynamic problems of nuclear structural design taking into account a defined material ductility are discussed.

● **6.12-28** Wierzbicki, T., **Extremum principles in the dynamics of rigid-plastic bodies: a critical review of existing applications,** *International Seminar on Extreme Load Conditions and Limit Analysis Procedures for Structural Reactor Safeguards and Containment Structures,* T 3/5, 16. (For a full bibliographic citation, see Abstract No. 1.2-2.)

The present article discusses some of the limitations and clarifies difficulties in applying extremum principles to dynamic problems for rigid-plastic continua and structures with special reference to the development of analytical methods. In particular, an attempt will be made to answer the following questions: (i) To what extent can the existing methods be used in the analysis of transient problems for impulsively or pulse-loaded structures? (ii) Under what conditions can stationarity of appropriate functionals be proved so that the direct methods of the calculus of variation can be used? (*iii*) Can a formal approximation method be developed for mode response in which an exact solution is approached with any desired degree of accuracy?

The above questions are related to the problem of a choice of a class of admissible functions. If the admissible functions contain the true solution, then the extremal theorems are shown to be applicable for deriving solutions to both transient and fixed-shape problems. However, such a situation can be regarded as exceptional and in typical cases the exact solution is either unknown or cannot be expressed in terms of elementary functions. It is pointed out that in these circumstances solutions to transient problems, obtained by means of analytical methods, might not be correct. Examples from the available literature are cited. By contrast, approximate mode form solutions can still be derived from variational principles, but then the question of accuracy arises. Most of the existing solutions were obtained from the kinematic principle by considering very simple admissible fields of velocities or accelerations involving only a few free parameters. Because of the nonlinearity of the dissipation function and lack of the property of superposition, it becomes very tedious or even impossible to evaluate effectively integrals appearing in the functionals when more terms are considered. Thus, the Rayleigh-Ritz method for finding a stationary point of a functional does not seem to be of great help when improved accuracy is desired.

Two ways of overcoming this difficulty are suggested. In the first one, possibilities of using a dynamic rather than a kinematic principle are explored; the former is expressed entirely in stresses. As an alternative, a new definition of a set of approximate functions is introduced and its advantages over Ritz's coordinate functions are explained for the plate problem. Finally, conditions are examined for the extrema to be stationary. Various types of extremal behavior with analytic and nonanalytic extrema are demonstrated depending on the smoothness of the yield condition and continuity in the slope of the admissible velocity field.

● **6.12-29** Corradi, L., **Mathematical programming methods for displacement bounds in elasto-plastic dynamics,** *International Seminar on Extreme Load Conditions and Limit Analysis Procedures for Structural Reactor Safeguards and Containment Structures,* T 3/6, 26. (For a full bibliographic citation, see Abstract No. 1.2-2.)

In this paper, a study of several deformation-bounding techniques for analyzing elastoplastic structures is performed. The problem is formulated and previous results are outlined with reference to general continua made of hardening materials. A class of discrete structural models (such as some finite element discretizations) is considered and, on this basis, two categories of deformation-bounding techniques are described from the previous results. All these techniques, some of which are new, permit optimization of the upper bound by solving one or more mathematical programming problems of special forms.

Some of the bounding procedures are shown to have merely theoretical interest, since they lead to cumbersome numerical procedures or to very coarse bounds. The formulations that appear to have practical application are compared from various standpoints (type of loading history, different hardening rules, influence of second-order geometric effects, quantities to be bounded) and an assessment of their usefulness is attempted. Generalizations to second-order geometric and thermal effects and to situations in which the time history is not completely known are envisaged.

● **6.12-30** Melosh, R. J., **Transformation for solving linear equations,** *Journal of the Structural Division, ASCE,* **103,** *ST6,* Proc. Paper 13020, June 1977, 1289–1302.

This paper proposes an equation-solving adjunct for improving the optimality of the computer solution of many (thousands of) sparsely populated linear equations. The paper provides a formal and computer implementation description of the algorithm, illustrating solution steps, and comparing equation solving by the process with modified Gauss decomposition. While the process increases calculations over Gauss decomposition, it is expected to reduce computer costs by factors up to seven because of its superior data-handling potential. It permits imposing linear constraints and making equilibrium checks to be an integral part of the equation-solving process. In the finite element analysis, assembly of the system stiffness and evaluation of element generalized forces can also be integrated with the equation solution. When an N-step iterative method is used for solving the transformed equations, it forms with the adjunct a hybrid iterative method which requires about the same number of calculations as Gauss decomposition.

● **6.12-31** Menegotto, M. and Pinto, P. E., **Slender RC compressed members in biaxial bending,** *Journal of the Structural Division, ASCE,* **103,** *ST3,* Proc. Paper 12799, Mar. 1977, 587–605.

A method is presented for the "exact" nonlinear analysis of reinforced and prestressed concrete slender members subjected to biaxial bending. The main feature of the method is the procedure that analyzes the stress-strain state of sections, under given axial force and biaxial moment. The outlined procedure, applied to a number of selected cross sections in the structure, allows an overall iterative procedure to find the solution of the structural problem for given loads. In fact, by integrating the deformabilities, the coefficients for the compatibility equations at the freed end are built up, while the integration of the curvatures yields the deflections, i.e., the right-hand side of

● *See* ***Preface,*** *page v, for availability of publications marked with dot.*

the equations themselves. The analysis of a 165 m bridge pier (prestressed) under earthquake simulating forces is given as an example.

● **6.12-32** Grooms, H. R. and Rowe, J., **Substructuring and conditioning,** *Journal of the Structural Division, ASCE,* **103,** *ST3,* Proc. Paper 12788, Mar. 1977, 507–514.

In the numerical solution of any mathematical problem, the question of accuracy always arises. When a large system of equations is solved, no "exact" solution is usually available for making comparisons. Although various tests can be performed on the computed solution, these tests represent necessary rather than sufficient conditions. Thus, a computed solution can satisfy all the necessary conditions but be severely inaccurate. A problem that yields this type of solution is called ill-conditioned. Substructuring is a popular technique for solving certain types of structural problems. It has been suggested that by the judicious choice of substructures some conditioning problems might be alleviated. A numerical investigation was performed to evaluate this suggestion. The results indicate that substructuring does not significantly influence the solution accuracy of ill-conditioned systems.

● **6.12-33** Beavers, J. E. and Beaufait, F. W., **Higher-order finite element for complex plate structures,** *Journal of the Structural Division, ASCE,* **103,** *ST1,* Proc. Paper 12665, Jan. 1977, 51–69.

A higher-order triangular finite element is developed for the analysis of complex folded plate structures by combining a bending element whose normal displacement is represented as a restricted quintic polynomial with a membrane element whose in-plane displacements are represented as cubic polynomials. The discontinuities that occur along lines of plate intersections are minimized by setting forth various geometric and element force-displacement formulation criteria. To satisfy boundary conditions of the more complex plate structures, boundary constraint equations are developed and augmented to the stiffness equations. To show the applicability of the method, several examples of prismatic and nonprismatic folded plate structures are investigated and the resulting data compared with other theoretical and experimental data.

● **6.12-34** Morrone, A., **Instantaneous response spectrum in seismic testing of nuclear power plant equipment,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(b), Paper K 8/11, 9. (For a full bibliographic citation, see Abstract No. 1.2-5.)

Seismic response spectra, as used in seismic analyses, give the maximum responses of single degree-of-freedom oscillators without consideration of the different time in the seismic time history at which each of the maximum responses occur. For response spectrum seismic analysis, the use of time-independent maximum responses is appropriate. The time dependence is considered in a statistical manner, for multidegree-of-freedom systems, usually by combining the modal effects by the square root of the sum of the squares. For seismic testing of electrical equipment, IEEE Std. 344-1975 makes use of the response spectrum to define the input motion of the shaking table. One of the basic requirements is that the test response spectrum (TRS); that is, the response spectrum produced by the shaking table motion, should envelop the required response spectrum (RRS) calculated from the building analysis at the support point of the equipment being tested. This enveloping requirement is in part due to the desire of generating a test motion which simulates the seismic motion in frequency content, as well as amplitude, so that multiple frequencies of the equipment being tested are excited simultaneously. However, since the RRS enveloping with the TRS is to be obtained with standard response spectra, independent of time, it is not obvious that the enveloping is also satisfied at a particular time so that the multiple frequency effects are indeed simultaneous. This may be investigated by comparison of the enveloping of TRS and RRS derived at particular times obtained with time histories whose standard TRS envelops the RRS.

This paper presents the concept of an instantaneous response spectrum (IRS) as the response of single degree-of-freedom oscillators at a particular time. It demonstrates that a shaking table random motion whose standard TRS envelops the RRS does not necessarily satisfy the enveloping requirement instantaneously; that is, any one (or more) instantaneous required response spectrum (IRRS) is not enveloped by an instantaneous test response spectrum (ITRS). Response spectra from different time histories, including single frequency sine beat motion used in resonance testing, are compared for enveloping with maximum response and with the actual response at particular times. These comparisons are given for the enveloping of RRS and IRRS derived with a time history response calculated at a particular building elevation of a nuclear power plant. For the test motion, several of the most severe ITRS derived with a modified EL Centro motion and with a sine beat motion with ten cycles per beat were used. It is shown that although the TRS with the modified EL Centro motion enveloped the given RRS, the selected modified EL Centro ITRS did not envelop the corresponding IRRS. With the sine beat motion, even though the TRS did not fully envelop the given RRS, the resulting sine beat ITRS did not require a larger factor for full IRRS enveloping than those of the modified EL Centro motion.

● **6.12-35** Kost, G., Tsui, E. Y. W. and Krutzik, N., **Axisymmetric finite element analyses of the KKP-II containment and reactor pressure vessel structures,** *International Seminar on Extreme Load Conditions and Limit*

Analysis Procedures for Structural Reactor Safeguards and Containment Structures, U 4/2, 27. (For a full bibliographic citation, see Abstract No. 1.2–2.)

Two refined axisymmetric finite element models were used for the dynamic seismic analyses of the KKP-II containment and RPV structures, using a postulated ground motion time history. One model was established primarily for the response of the containment structure, whereas the other was used for the response of the reactor pressure vessel plus internals.

In the first model, refined axisymmetric thin shell and solid elements were used for the containment components, while fluid elements were employed to represent the hydrodynamic effects of the fluid entrapped between the steel shells. Effects of the reactor building, RPV and internals, foundation mat, as well as soil-structure interaction are included, using equivalent cylindrical or solid elements and suitable classical solutions of rigid plates on an infinite halfspace. Eigenvalues of the structural components of this model were generated and compared with those of similar but three-dimensional models established for the KKP-I. The correlations were satisfactory.

In the second model, both refined beam and shell elements were used to represent the internals and pressure vessel, while fluid elements were used for the water contained within the RPV. Whenever hinged boundary conditions were encountered, the corresponding rotational degree-of-freedom was suppressed. Effects of the reactor building, concrete and steel containments, foundation mat and soil-structure interaction were also considered in this model, using relatively coarser equivalent finite elements. To solve the resulting consistent mass matrices efficiently, the determinant search eigenvalue extraction iterative technique was utilized.

The modal superposition procedure was used to generate most of the solutions of the present analyses, applying the recently updated digital program ASHSD3. For the purpose of determining the stress distributions within the internals more critically, a beam element and several special boundary conditions have been incorporated in this program. It now accepts four different types of elements, i.e., shell, solid, fluid and beam, and is capable of producing desirable force resultants and stresses, either by the isotropic beam theory or the orthotropic shell or solid theory. To ensure that the dynamic characteristics of the present models are accurate enough to provide meaningful response, eigenvalues and vectors of these models have also been compared with those of similar three-dimensional beam models. It was found that the results show good agreement, especially on the fundamental frequencies.

● **6.12–36** Grooms, D. W., ed., **Finite elements in structural analysis: a bibliography with abstracts,** National Technical Information Service, Springfield, Virginia, July 1976, 166. (NTIS Accession No. NTIS/PS-76/0525)

Finite element analysis as applied to dynamic and static problems is analyzed as well as linear and nonlinear problems. Some computer programs for finite element analysis are also presented. The search period covered is 1972–June 1976. This updated bibliography contains 166 abstracts, 96 of which are new entries to the previous edition.

● **6.12–37** Dafalias, Y. F., Ramey, M. R. and Sheikh, I., **A model for rate-dependent but time-independent material behavior in cyclic plasticity,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. L, Paper L 1/8, 8. (For a full bibliographic citation, see Abstract No. 1.2–5.)

It is the purpose of this paper to present a model for rate-dependent, but time-independent, material behavior under cyclic loading in the plastic range. What is referred to as time-independent behavior here is the absence of creep and relaxation phenomena from the behavior of the model. The notion of plastic internal variables (piv) is introduced, as properly invariant scalars or second-order tensors, whose constitutive relations are rate-type equations not necessarily homogeneous of order one in the rates, as it would be required for independent plasticity. The concept of a yield surface in the strain space and a loading function in terms of the total strain rate is introduced, where the sign of the loading function defines a zero or nonzero value of the rate of piv. Thus rate dependence is achieved without time-dependent behavior (no creep or relaxation). In addition, discrete memory parameters associated with the most recent event of unloading-reloading in different directions enter the constitutive relations for the piv.

A particular form of the constitutive relations is assumed, where the rate of piv is a linear combination of the strain rate components, with coefficients depending on the second invariant of the strain rate tensor, which can be viewed as a scalar measure of the rate of deformation in the multiaxial case and a direct generalization of the uniaxial strain rate. This leads to a particularly simple form of the constitutive relations resembling the ones for rate-independent plasticity. The uniaxial counterpart would be a relation between the plastic strain rate (as one of the piv) and the total strain rate through a plastic modulus which depends on the strain rate, the piv, and the discrete memory parameters.

Within the above general framework, a generalization of a model proposed by Dafalias and Popov is suggested for cyclic rate-independent plasticity in order to incorporate rate dependence. The characteristic feature of this model, following from uniaxial experimental observations, was the concept of a "bounding surface" in stress space enclosing

● *See **Preface**, page v, for availability of publications marked with dot.*

the yield surface. The proximity of these two surfaces, in the course of their translation and deformation during plastic loading, determines the appropriate piv and discrete memory parameters on which the plastic modulus depends. Similar concepts can be introduced in strain space for the generalized model, where the difference now is that the size of the bounding surface depends also on the second invariant of the strain rate tensor; thus, the plastic modulus (different for strain space formulation) depends indirectly on the strain rate. This is a direct generalization of the uniaxial experimental observation that the stress-strain curves converge with bounds in stress-strain space, whose position depends on the strain rate. Besides this difference, the mechanism of the new model is identical to the model for rate-independent behavior, and the application in the uniaxial case is illustrated by simple examples, which show good agreement between theory and experiment.

● **6.12-38** Thulin, Jr., F. A., **Constructing mathematical models of cable tray and support systems to determine seismic response in nuclear plants,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(a), Paper K 3/11, 7. (For a full bibliographic citation, see Abstract No. 1.2-5.)

The problem is to construct a stick model with lumped masses representing the cable tray/support system in a nuclear plant subjected to seismic forces. The equivalent physical and geometric properties of the lattice subsystems to be changed into the theoretical stick components must be determined. The effective moment of inertia and shear area of the lattice structures are best ascertained by static load tests; damping ratios, by dynamic tests. Properties of nonlattice components are known from given data or can be readily calculated. Engineering judgment is required to fix the size and location of lumped masses.

A study of a steel ladder-type cable tray in the transverse direction considered as a plane frame serves to show how a lattice subsystem is changed to a mathematically equivalent stick component. Testing this tray at a span of 3426 mm with simple end conditions and equal concentrated loads of 1000 N at 912 mm and 2514 mm from the left-hand support yields deflections of 4.52 mm at the load points and 5.03 mm at mid span. The modulus of elasticity is 203400 MPa; the shear modulus, 77900 MPa. The solution of two equations, one for the load point deflection and one for the center deflection, gives the two unknowns, namely the moment of inertia and shear area. These are the transverse geometric properties of the equivalent stick. Similar testing procedures give the transverse properties of trapeze-type supports.

Results of forced vibration tests conducted on aluminum ladder-type cable trays loaded with cables yield, using "decay-rate" and "half-power" analysis, average damping ratios of 21% in the transverse direction, and 13% in the vertical direction. Trays and supports in the vertical and longitudinal directions are not lattice components. Hence, the sectional properties of these components are given or can be calculated using procedures for common geometric shapes. Torsional geometric properties are assumed as the sum of the properties of the elements at a cross-sectional cut. Assembling theoretically a series of plane frame stick models or one space frame stick model with the lumped masses artfully placed in either case provides the mathematical analog(s) needed for computer analysis. For the cable tray stick, the mass of about 300 mm of cable plus tray is one lump. The lumps on the cable tray stick are spaced approximately 300 mm on centers between supports with the end lump approximately 150 mm from each support. On the support sticks, the lumps are located where the cable trays are connected to the support. The quantity of each lump is the tributary mass of the support at the connection plus, in the case of a plane frame analysis only, the tributary mass of the cable and tray. Thus, combining these significant new test procedures to determine moment of inertia and shear area of lattice components with the conventional method of treating nonlattice components and lumping masses, a theoretical stick model with lumped masses representing a cable/tray support system is constructed and ready for mathematical excitation.

● **6.12-39** Takemori, T. *et al.*, **Analytical procedure in aseismic design of eccentric structure using response spectrum,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(a), Paper K 4/7, 12. (For a full bibliographic citation, see Abstract No. 1.2-5.)

In this paper, responses are evaluated by the following two methods with use of the typical torsional analytical models in which masses, rigidities, eccentricities between the centers thereof and several actual earthquake waves are taken as the parameters: (1) the root mean square of responses by using the response spectra derived from the earthquake waves, and (2) time history analysis by using the earthquake waves. The earthquake waves used are chosen to represent the different frequency content and magnitude of the response spectra.

The typical results derived from the study are as follows: (a) the response accelerations of mass center in the input earthquake direction by method (1) coincide comparatively well with those by method (2), (b) the response accelerations perpendicular to the input earthquake direction by method (1) are 2 to 3 times as much as those by the method (2), (c) the amplification of the response accelerations at arbitrary points distributed on the spread mass to those of center of the lumped mass by method (1) are remarkably large compared with those by method (2) in both directions. These problems with the response spectrum analysis for the above-mentioned eccentric structure

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are discussed, and an improved analytical method applying the amplification coefficients of responses derived from this parametric time history analysis is proposed for the actual seismic design by using the given design ground response spectrum with the root mean square technique.

● **6.12-40** Duff, C. G. and Biswas, J. K., **Floor response spectra from spectrum compatible motions,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(a), Paper K 4/12, 10. (For a full bibliographic citation, see Abstract No. 1.2-5.)

Floor response spectra are commonly used for the design of equipment, piping systems, control instruments, or other sub-systems located in nuclear power plants. An artificial or modified real time history of motion, compatible with a set of ground response spectra, is often used to generate these floor response spectra. This paper shows that even using a modified real time history, closely matching a given ground response spectrum, large variations of the floor response spectra can result. The paper discusses the cause of such variations, and aims at finding ways to deal with them.

A reactor building similar to that used in CANDU nuclear power plants is chosen as an example. The reactor building is idealized by a simple stick model with five lumped masses. The N/S component of the El Centro (1940) earthquake time history is modified by superposition of output waves to make it compatible with a given ground response spectrum. Floor response spectra are determined, at a number of locations, using the modified El Centro motion. To account for the variation of structural frequencies, due to uncertainties of material properties and approximations in the modeling technique used, the frequency is varied slightly and the floor response spectra are computed for each frequency. A comparison of the resulting floor response spectra shows that by changing the structural frequency slightly, not only can the frequencies of peak response be shifted, but also the value of the peak can be changed considerably. It is therefore necessary to smooth the floor response spectra and to broaden the peaks. Procedures are described indicating how the floor response spectra are smoothed and the degree of broadening that should be used.

● **6.12-41** Lasker, L. *et al.*, **OSCIL and OSCVERT: computer codes to evaluate the non-linear seismic response of an HTGR core,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(b), Paper K 7/2, 8. (For a full bibliographic citation, see Abstract No. 1.2-5.)

OSCIL and OSCVERT are FORTRAN codes which simulate the effects of seismic input on a high temperature gas-cooled reactor (HTGR) core. The analysis is to be used to determine safety standards and licensing regulations. OSCIL models the core as a one-dimensional nonlinear spring-mass system and OSCVERT is the two-dimensional extension, with each mass in OSCIL expanded to a vertical column of blocks. Either code can be used more generally for any system represented by such a model.

● **6.12-42** Buck, K. E., von Bodisco, U. and Winkler, K., **Dynamic analysis of electric equipment for nuclear power stations under seismic loads,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(b), Paper K 6/5, 12. (For a full bibliographic citation, see Abstract No. 1.2-5.)

The response spectrum method, generally accepted as the most practical method for linear seismic analysis of power station components, is here applied in conjunction with the finite element method to electrical components. The dynamic response of complex structures is governed by a number of natural modes of the multidegree-of-freedom system, where for seismic analyses the low-frequency end of the spectrum of natural frequencies usually is significant. The accuracy of the response spectrum method for a given response spectrum hinges on the accuracy by which the natural frequencies and modes of the structure are known. In contrast to published work on electrical equipment, frequencies and modes are determined using finite element idealizations. These mathematical models of switchgear and other electrical equipment may involve several hundreds of degrees-of-freedom. The numerical effort is justified by the ability to accurately model complicated structures as early as in the design phase.

The complete dynamic analysis based on the superposition of the natural modes as carried out for an electronics cabinet and for transmitter supports is outlined, and selected results are presented. Several different methods are in use for the superposition of the contributions of the different modes. Here, addition of absolute values, the square-root of the sum of squares, and a mixed method taking account of closely spaced modes are applied. For different structures, the degree of conservatism is thus demonstrated, the largest difference in the stresses computed by the different methods being approximately 30 percent. For structures whose natural frequencies are in the spectrum range with zero period response, a simplified response analysis using static loads is often carried out. This is demonstrated for the electronics cabinet and transmitter mountings, and the results are compared with the dynamic analyses. It is seen that this pseudo-dynamic analysis yields useful approximations for the distributions of stresses. Practical details of the structural models as well as results are presented for several switchgear and electronics cabinets.

The practicability of the response spectrum method based on detailed finite element modeling is thus demonstrated, and the data on computer run times for different types of structures allow a reliable estimate of the effort required.

● 6.12-43 Brusa, L., Ciacci, R. and Restelli, F., **A simplified procedure for evaluating modal damping factors in structures with widely varying damping capacities,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(a), Paper K 3/10, 9. (For a full bibliographic citation, see Abstract No. 1.2–5.)

The dynamic response of composite structures by the modal superposition method can be conveniently performed only if the motion equations are decoupled by proper coordinate transformations. The decoupling of motion equations can be exactly obtained by using the damped complex eigenmodes of the structure. This rigorous procedure results in an expensive computational effort for structures discretized with a large number of degrees-of-freedom. For this reason, some approximate solutions of the problem were proposed requiring only the computation of the undamped modal characteristics of the system. Such techniques are based on the assumption that the damping matrix is diagonalized by the undamped modes.

In this paper, a simplified procedure for the evaluation of the equivalent modal damping ratios is proposed taking into account to some extent the contribution of the off-diagonal terms of a damping matrix with undamped modes. This technique is based on the assumption that the complex eigenmodes can be expressed as a linear combination of the first few undamped modes. The coefficient of the linear combination and the approximate frequencies and damping modal factors are computed by solving a reduced generalized eigenvalue problem whose order is equal to the number of undamped modes considered.

To evaluate the efficiency of this procedure, a numerical experimentation was performed for the reinforced concrete containment and the internals of a typical pressure water nuclear reactor founded on different soil conditions. For this system, the exact complex modes, frequencies, and damping factors were computed, and the results were compared with those obtained by the proposed technique. Additional comparisons were also made with the results obtained neglecting the off-diagonal terms of the damping matrix. The numerical experimentation showed that a very small number of undamped modes were necessary to obtain modal damping factors nearly coincident with the exact ones. On the contrary, only in some cases, the results obtained using the damping matrix were reasonably accurate.

● 6.12-44 Shaw, D. E., **Seismic structural response analysis using consistent mass matrices having dynamic coupling,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(a), Paper K 3/4, 14. (For a full bibliographic citation, see Abstract No. 1.2–5.)

The classical treatment of seismic structural response has been primarily confined to simply coupled or close-coupled dynamic systems utilizing lumped-mass representation of the structural mass distribution. With the continued improvement of discrete or finite element methods of analysis, the use of consistent or equivalent mass matrices which are derived from the same displacement function used for the derivation of elemental stiffness matrices has become more popular, especially for equipment analysis and the analysis of continuum structures. One of the benefits of consistent mass matrices is off-diagonal terms in the mass matrix. These terms offer a better approximation to the actual mass distribution of continuum structures than lumped-mass matrices and lead to a dynamic coupling in addition to the static coupling inherent in the stiffness matrix. However, the dynamic coupling leads to a different seismic inertial loading vector than that obtained in the absence of off-diagonal terms. The basis for the theoretical development of this paper is the linear matrix equations of motion for an unconstrained structure subject to support excitation. The equations are formulated in terms of absolute displacement, velocity, and acceleration vectors. By means of a transformation of the absolute response vectors into displacements, velocities and accelerations relative to the support motions, the homogeneous equations become nonhomogeneous and the nonhomogeneous boundary conditions become homogeneous, with relative displacements, velocities and accelerations being zero at support points. The forcing function or inertial loading vector is shown to consist of two parts. The first part is comprised of the mass matrix times the support acceleration function times a vector of structural displacements resulting from a unit vector of support displacements in the direction of excitation. This inertial loading corresponds to the classical seismic loading vector and is indeed the only loading vector for lumped-mass sytems. The second part of the inertial loading vectors consists of the mass matrix times the support acceleration function times a vector of structural accelerations resulting from unit support accelerations in the direction of excitation. This term is not present in classical seismic analysis formulations and results from the presence of off-diagonal terms in the mass matrices which give rise to dynamic coupling through the mass matrix; thus, for lumped-mass models, the classical formulation of the inertial loading vector is correct. However, if dynamic coupling terms are included through off-diagonal terms in the mass matrix, an additional inertial loading vector must be considered. In general, off-diagonal terms may arise in mass matrices from two sources: (1) consistent mass matrix formulations whereby mass elemental matrices are derived from the same assumed displacement functions as used to derive stiffness matrices; and (2) dynamic condensation of systems whereby a reduced set of dynamic degrees-of-freedom are to characterize the dynamic behavior of a

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structural system. The presence of off-diagonal mass coupling terms in models using consistent mass matrices is obvious and a direct result of the mass matrix formulation. However, the presence of off-diagonal terms in a dynamically condensed reduced mass matrix is not obvious and off-diagonal terms may be present in the reduced mass matrix even though only lumped masses were used to formulate the dynamic model.

Several examples are included to show a comparison of the discrete element and closed-form solutions and the solution obtained using traditional methods. The result of consideration of the structural accelerations to unit support accelerations is shown to lead to larger modal participation factors than would be determined neglecting the dynamic coupling through the mass matrix.

● **6.12-45** Boulet, J. A. M. and Carley, T. G., **Response spectrum analysis of coupled structural response to a three component seismic disturbance,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(a), Paper K 3/5, 12. (For a full bibliographic citation, see Abstract No. 1.2-5.)

The work discussed herein is a comparison and evaluation of several response spectrum analysis (RSA) techniques as applied to the same structural model with seismic excitation having three spatial components. The structural model includes five lumped masses (floors) connected by four elastic members. The base is supported by three translational springs and two horizontal torsional springs. In general, the mass center and shear center of a building floor are distinct locations. Hence, inertia forces, which act at the mass center, induce twisting in the structure. Through this induced torsion, the lateral displacements of the mass elements are coupled. The ground motion components used for this study are artificial earthquake records generated from recorded accelerograms by a spectrum modification technique. The accelerograms have response spectra which are compatible with U.S. NRC Regulatory Guide 1.60. Lagrange's equations of motion for the system were written in matrix form and uncoupled with the modal matrix. Numerical integration (fourth-order Runge-Kutta) of the resulting modal equations produced time histories of system displacements in response to simultaneous application of three orthogonal components of ground motion, and displacement response spectra for each modal coordinate in response to each of the three ground motion components. Five different RSA techniques were used to combine the spectral displacements and the modal matrix to give approximations of maximum system displacements. These approximations were then compared with the maximum system displacements taken from the time histories. The RSA techniques used are the method of absolute sums, the square root of the sum of the squares, the double sum approach, the method of closely spaced modes, and Lin's method. The vectors of maximum system displacements as computed by the time history analysis and the five response spectrum analysis methods are presented. The absolute sum method, Lin's method and the closely spaced modes method are rather conservative; that is, they overestimate most or all the maximum displacements. The results of the square root of the sum of the squares method and the double sum method are closer to the time history results but underestimate several displacements. The double sum method, on the whole, gave more accurate results than did the square root of the sum of the squares approach.

A noticeable feature of these results is that in reading down the displacement vector, the pattern of variation of the absolute value of the percentage error is roughly the same for all methods. Since the various response spectrum methods are simply different ways of combining the same numbers, this is not surprising. The high percentage errors for some system displacements serve to emphasize the fact that system coordinate maxima are not functionally related to modal coordinate maxima. For any combination of structural model and excitation, one can expect to find certain displacements for which response spectrum analysis does not give good approximations.

● **6.12-46** Yu, I.-W., **A finite element method for seismic response analysis,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(a), Paper K 3/6, 12. (For a full bibliographic citation, see Abstract No. 1.2-5.)

In the mathematical modeling of a structural system or component, the support points of the structure can often be idealized by a single base point representation where the gross effect of the support motion is characterized by translational as well as rotational motions at the base point. For the response spectrum analysis of the structural system or component, the excitation due to the support motion is then expressed in terms of a response spectrum generated by the consideration of the translational base motion only. The effect of the base rotation is often neglected by assuming that it contributes only a secondary effect to the overall structural response. For some structural models, the effect of the base rotation may have significant contribution on the structural response. The neglecting of the rotational effect may underestimate the result.

The purpose of this paper is to present a complete finite element method for the response spectrum analysis of a structural system subjected to a base excitation expressed in terms of both translational and rotational response spectra. The major complexity involved in the procedure is the formulation of the inertial forces associated with a noninertial frame which is attached to and in motion with the base point of the structure. Throughout the discussion, the rotation of the base point is assumed to be small. The formulation is general and is applied to any type of finite element discretization. The inertial forces are formulated on an element-by-element basis. The mass matrix of the element can be formulated by either the lumped or the consistent mass approach. The procedure of condensation is also utilized to reduce the solution matrix of a large structural system. The basic principle used in the condensation is to preserve the system kinetic energy, potential energy, and the load potential. The concept of the rotational base excitation in terms of a response spectrum is introduced. The physical meanings of participation factors and modal masses associated with the rotational response spectrum are closely examined. The concept of the modal mass defined by the U.S. Naval Research Lab. for translational base motion is extended to the situation with base rotation; thus, modal masses with respect to rotations and translation-rotation couplings are introduced. A simple structural system is selected to illustrate the use of the translational as well as the rotational response spectra. Results of the structural response due to translation and rotation are presented. The accuracy of the solution due to condensation is also examined. Finally, the method described in this presentation is general and can be implemented into any general purpose finite element computer code.

● **6.12-47** Kawai, T., **New beam and plate bending elements in finite element analysis,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(a), Paper K 3/8, 11. (For a full bibliographic citation, see Abstract No. 1.2-5.)

From the viewpoint of limit analysis of three-dimensional plate structures, collapse load can be defined when the bending strength is exhausted. Based on such consideration, new beam and plate bending elements are proposed. A new beam element consists of rigid bars connected by a rotational spring, while a plate element consists of rigid plates with a rotational spring. The stiffness matrices of these bending elements are one-half as large as other beam bending elements and one-third of triangular plate bending elements. Therefore, considerable reduction of computing time can be expected in routine finite element analysis of structures with minor penalty in increase of mesh division. The spring constants in the matrices can be computed theoretically by using the finite difference formula for the curvature based on the second-order polynomial approximation. The result of limit analysis of a beam clamped at both ends under a single concentrated load and the results of collapse load analysis of a rectangular plate with unusual boundary conditions are shown. Agreement between analytical and experimental results was found to be extremely good. The author believes that the use of these new elements will be especially effective in nonlinear analysis of complex structures. Extended application of these models to in-plane, shell and three-dimensional problems was also successful. Dynamic collapse load analysis of simple beam and plate structures is also presented.

● **6.12-48** Hart, G. C. and Yao, J. T. P., **System identification in structural dynamics,** *Journal of the Engineering Mechanics Division, ASCE,* **103**, *EM6,* Proc. Paper 13443, Dec. 1977, 1089-1104.

The correlation between analytical structural model and experimental test results has become the subject of considerable recent research efforts. This area is known as system identification. The engineer has applied the tools of mathematical programming, statistics, and structural mechanics to seek solutions to this problem. This paper summarizes past work in system identification in the structural dynamics area.

● **6.12-49** Hutchinson, J. R. and Benitou, J. J., **Variable order finite elements for plate vibration,** *Journal of the Engineering Mechanics Division, ASCE,* **103**, *EM5,* Proc. Paper 13252, Oct. 1977, 779-787.

Rectangular finite elements with a variable number of degrees-of-freedom per element are developed for thin elastic plates. The displacement field for the element is described by a fixed series of polynomial terms plus a variable number of trigonometric terms. The polynomial terms alone are sufficient to describe the displacement field, and the additional trigonometric terms allow for better description of the displacement field within the element. If only the polynomial terms are used, the element becomes a standard type of finite element; whereas, if only one element is used, the procedure becomes the classical Rayleigh-Ritz method. A limited number of comparisons indicate that using one high-order element yields better frequencies than using many low-order elements with the same total number of system degrees-of-freedom. The low-order elements have computational advantages which may make them superior when considering overall computational efficiency.

● **6.12-50** Grooms, D. W., ed., **Structural mechanics software. Volume 2. May 1975-May 1977: a bibliography with abstracts,** National Technical Information Service, Springfield, Virginia, June 1977, 150. (NTIS Accession No. NTIS/PS-77/0525)

The use of computer programs in structural analysis and design problems is cited. Detailed analyses are included of structural problems—applied and theoretical—including stress analysis, vibration problems, shear stress analysis, deformation analysis, and others. The major computer programs cited in this report are NASTRAN, EPSOLA, SUPERSCEPTRE, and SINGER. This updated bibliography contains 151 abstracts, 74 of which are new entries to the previous edition.

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● **6.12-51** Wu, J. J., **Solutions to initial value problems by use of finite elements—unconstrained variational formulations,** *Journal of Sound and Vibration,* **53,** *3,* Aug. 8, 1977, 341-356.

This paper presents a variational formulation which treats initial value problems and boundary problems in a unified manner. The basic ingredients of this theory are (1) adjoint variable and (2) unconstrained variations. The theory is an extension of the finite element unconstrained variational formulation used previously in solving several nonconservative stability problems. The technique which makes this extension possible is described. This formulation thus enables one to adapt such numerical techniques as the finite element method, which has had great success and popularity for solution of boundary value problems, for solutions of initial value problems as well. These formulations are given here for a forced vibration problem, a heat (mass) transfer problem, and a wave propagation problem. Numerical calculations in conjunction with finite elements for two specific examples are obtained and compared with known exact solutions.

● **6.12-52** Ku, A. B., **Upper and lower bound eigenvalues of a conservative discrete system,** *Journal of Sound and Vibration,* **53,** *2,* July 22, 1977, 183-187.

The Timoshenko quotient and a lower bound formula, which are available for continuous systems, are presented here for discrete systems. With both the upper and lower bounds known, the estimation of eigenvalues becomes certain. Errors associated with these upper and lower bounds are discussed and an example given.

● **6.12-53** Rangacharyulu, M. A. V. and Srinivasan, P., **A note on the ultraspherical polynomial approximation method of averaging,** *Journal of Sound and Vibration,* **53,** *1,* July 8, 1977, 63-69.

The method of averaging based on ultraspherical polynomial approximation has been applied to several nonlinear oscillation problems. In this paper, it is shown that the above method can be interpreted as a Galerkin technique with an appropriate weight function. Even though the analysis is not rigorous enough mathematically, it does provide insight into this generalized method of averaging.

● **6.12-54** Tiwari, R. N. and Subramanian, R., **Multiple time scaling for analysis of third order non-linear differential equations,** *Journal of Sound and Vibration,* **52,** *2,* May 22, 1977, 165-169.

The two-time perturbation procedure has been employed for investigating the transient and the steady-state analysis of third-order nonlinear, nonautonomous differential equations.

● **6.12-55** McDaniel, T. J. and Eversole, K. B., **A combined finite element-transfer matrix structural analysis method,** *Journal of Sound and Vibration,* **51,** *2,* Mar. 22, 1977, 157-169.

A combined finite element transfer matrix (FETM) method is developed to study the dynamics of orthogonally stiffened structures. Finite element procedures are used in FETM to formulate transfer matrices for structural components which are not one dimensional. The resulting transfer matrices are used to reduce the large number of unknowns occurring in a standard finite element analysis by obtaining transfer matrix relationships over large units of the structure. A reduction in computer storage and computation time is obtained since the dimension of the final matrix equation to be solved is considerably reduced. The accuracy of the FETM results are verified by comparison with an exact solution to the limiting modes and frequencies of a periodically stiffened structure of infinite length. A reduction of computer time was demonstrated for the FETM by a comparison with the SAP IV finite element program. The efficiency of the FETM method was further demonstrated by computing the frequency response of a two row-five span orthogonally stiffened plate structure using ordinary transfer matrix procedures.

● **6.12-56** Brandt, K., **Calculation of vibration frequencies by a hybrid element method based on a generalized complementary energy principle,** *International Journal for Numerical Methods in Engineering,* **11,** *2,* 1977, 231-246.

Based on a generalized complementary energy principle, the derivation of the element matrices is presented for calculation of natural frequencies. The degrees-of-freedom are not defined on nodal points but in an abstract way. No restrictions about the number of interpolation functions in the interior and at the boundaries of the element have been introduced. The exact solution of the discretized element equations leads to the dynamic stiffness matrix, while the approximate solution results in a linear eigenvalue problem. Plate bending problems are used to study the convergence of frequencies, depending on the degrees of interpolation functions within the element and on its boundaries and on the number of elements.

● **6.12-57** Dong, S. B., **A Block-Stodola eigensolution technique for large algebraic systems with non-symmetrical matrices,** *International Journal for Numerical Methods in Engineering,* **11,** *2,* 1977, 247-267.

A Block-Stodola eigensolution method is presented for large algebraic eigensystems of the form $\mathbf{AU} = \lambda\mathbf{BU}$ where **A** is real but non symmetric. The steps in this method parallel those of a previous technique for the case when both **A** and **B** were real and symmetric. The essence of the technique is simultaneous iteration using a group of trial vectors instead of only one vector as is the case in the

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classical Stodola-Vianello interation method. The problem is then transformed into a subspace where a direct solution of the reduced algebraic eigenvalue problem is sought. The main advantage is the significant reduction of computational effort in extracting a subset of eigenvalues and corresponding eigenvectors. Theorems from linear algebra serve to underlie the basis of the present technique. Complex eigendata that emerge during iteration can be handled without doubling the size of the problem. Higher order eigenvalue problems are reducible to first-order form for which this technique is applicable. The treatment of the quadratic eigenvalue problem illustrates the details of this extension.

● **6.12-58** Petersson, H. and Popov, E. P., **Constitutive relations for generalized loadings,** *Journal of the Engineering Mechanics Division, ASCE,* **103,** *EM4,* Proc. Paper 13144, Aug. 1977, 611-627.

A multisurface approach in stress space is adopted as the most general means of describing incrementally the plastic flow phenomena. The inner surface is the yield surface and the outer one is the bounding surface. All other surfaces are intermediate. The size of the surfaces may change in a congruent manner due to changes in the state variables. The surfaces can only translate. For plastic flow, the two key parameters are the plastic strain at current time, and an increment in plastic strain from the current time to some future time for the case of monotonic outward loading. The locations of the center of a surface are a function of the preceding parameters. The examples given in the paper are for mild steel having a characteristic plateau and strain-hardening ranges. The proper balance between the influence of isotropic hardening compared to the kinematic one is achieved by using a variable weighing function.

● **6.12-59** Richards, T. H. and Leung, Y. T., **An accurate method in structural vibration analysis,** *Journal of Sound and Vibration,* **55,** *3,* Dec. 8, 1977, 363-376.

An important element in the dynamic analysis of structures is the computation of natural frequencies and modes. For complex systems, this inevitably requires automatic computation so that an accurate and reliable algorithm is essential. In this paper, a method in which subspace iteration is utilized in conjunction with a frequency-dependent mass and stiffness formulation is described and applied to framed structures.

● **6.12-60** Row, D. G., Powell, G. H. and Mondkar, D. P., **Solution of progressively changing equilibrium equations for nonlinear structures,** *Computers & Structures,* **7,** *5,* Oct. 1977, 659-665.

In the analysis of nonlinear structures by tangent stiffness methods, the equilibrium equations change progressively during the analysis. When direct methods of solving these equations are used, it may be possible to reuse a substantial part of the previously reduced coefficient matrix, and hence substantially reduce the equation-solving effort. This paper examines procedures for re-solving equations when only selected parts of the reduced matrix need to be modified.

The Crout and Cholesky algorithms are first reviewed for initial complete reduction and subsequent selective reduction of the coefficient matrix, and it is shown that the Cholesky algorithm is superior for selective reduction. A general procedure is then presented for identifying those parts of the coefficient matrix which remain unchanged as the structure changes. Finally, a general purpose in-core equation solver is presented, in which those parts of the previously reduced matrix which need to be modified are determined automatically during the solution process, and only these portions are changed. The equation solver is based on the Cholesky algorithm and is applicable to both positive-definite and well-conditioned nonpositive-definite symmetrical systems of equations.

● **6.12-61** Tsipouras, P., **Numerical stability of incompletely integrated shape functions,** *Computers & Structures,* **7,** *5,* Oct. 1977, 679-680.

Finite element stiffness matrices formed by "unstable" integration schemes regain positive definiteness with a sufficient number of imposed boundary conditions. Moreover, the feared decline in the condition of these matrices does not materialize; their spectral condition number being $O(h^{-2m})$ as for the strictly variational elements.

● **6.12-62** Caravani, P., Watson, M. L. and Thomson, W. T., **Recursive least-squares time domain identification of structural parameters,** *Journal of Applied Mechanics,* **44,** **Series E,** *1,* Mar. 1977, 135-140.

A method of identifying structural parameters such as damping and stiffness of a building from its time response under dynamic excitation is presented. A least-squares recursive computer algorithm which requires no matrix inversion is developed and tested with the response of a two degree-of-freedom structure including Gaussian white noise. The algorithm provides means to account for both the model uncertainty and the investigators' confidence in the initial guess of the parameters. These statistical quantities can be updated with passage of time. The study indicates that rapid convergence to the correct values of the parameters takes place even under severe noise in the response data.

● **6.12-63** Hsu, C. S., Yee, H. C. and Cheng, W. H., **Determination of global regions of asymptotic stability**

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for difference dynamical systems, *Journal of Applied Mechanics,* **44, Series E,** *1,* Mar. 1977, 147–153.

In this paper, certain global properties of dynamic systems governed by nonlinear difference equations are studied. When an asymptotically stable equilibrium state or periodic solution exists, it is desirable to be able to determine a global region of asymptotic stability in the state space. An effective method is presented for the determination of such a region. It will be seen that once certain features of the backward mapping have been properly delineated the development of the method becomes a rather simple one. The method is mainly presented for second-order systems but the basic ideas are also applicable to higher-order systems. Through the development of the theory and examples, one also sees that, in general, the region of asymptotic stability for a nonlinear difference system is of extremely complex shape.

● **6.12-64** McDaniel, T. J., Testa, F. J. and Babu, P. V. T., **Solution bounds for transient vibration problems,** *Journal of Applied Mechanics,* **44, Series E,** *1,* Mar. 1977, 159–164.

The theory of differential inequalities is applied to systems of nonlinear differential equations of the initial value type. Established bounding theorems are used to construct bounds on the solution of these equations and to develop an iterative procedure for improving these bounds. The methods are illustrated by constructing upper and lower solution bounds for one and two degree-of-freedom dynamic systems.

● **6.12-65** Gambhir, M. L. and Batchelor, B. de V., **A finite element for 3-D prestressed cablenets,** *International Journal for Numerical Methods in Engineering,* **11,** *11,* 1977, 1699–1718.

A finite element method is presented for the analysis of prestressed cablenets. The method is based on representing the prestressed cablenet as a series of finite length curved elements. The large displacement formulation used enables the evaluation of the static and dynamic response of 3-D cablenets. The development is general and the mathematical basis is explained at length. It is shown by means of comparison functions that for absolute continuity at nodes a cubic displacement field is sufficient for the prediction of the first frequency of shallow nets. For globally deep networks, the accuracy can be increased by employing a quintic displacement field for the normal component of displacement.

The application of the proposed model to cable structures of shallow and deep global geometry is presented. A variety of edge boundary shapes are employed in order to illustrate the versatility of the large displacement formulation. In all the example problems studied, gravity load has been taken as the initial load condition in calculating the equilibrium configuration. The stiffness and consistent mass matrices associated with the equation of motion are derived using Hamilton's variational principle.

● **6.12-66** Rao, N. S. V. K. and Das, Y. C., **A mixed method in elasticity,** *Journal of Applied Mechanics,* **44, Series E,** *1,* Mar. 1977, 51–56.

A mixed method for three-dimensional elastodynamic problems has been formulated which gives a complete choice in prescribing the boundary conditions in terms of either stresses or displacements or partly stresses and partly displacements. The general expressions for the responses of the elastic body have been derived in the form of transcendental partial differential equations of a set of initial functions, which can be evaluated in terms of the prescribed boundary conditions. The method so formulated has been illustrated by applying it to the theory of plates. Only plates subjected to antisymmetric loads have been considered for illustration. Some examples of free and forced vibration of plates have been presented. Results are compared with solutions from existing theories.

● **6.12-67** Kraus, H. D., **A hybrid stiffness matrix for orthotropic sandwich plates with thick faces,** *International Journal for Numerical Methods in Engineering,* **11,** *8,* 1977, 1291–1306.

A stiffness matrix is derived for a rectangular orthotropic sandwich plate element with thick faces by the assumed stress hybrid approach. It is shown that an independent polynomial can be used for the internal forces of the sandwich plate by the membrane theory (assumption of thin faces) and also for the additional moments and shear forces of the faces. The boundary displacements are approached solutions of the homogeneous differential equation of a sandwich beam.

● **6.12-68** Wood, W. L., **On the Zienkiewicz four-time-level scheme for the numerical integration of vibration problems,** *International Journal for Numerical Methods in Engineering,* **11,** *10,* 1977, 1519–1528.

The Zienkiewicz weighted residual method produces a very general (three-parameter) four-time-level scheme for the numerical integration of vibration problems. This paper obtains the values of these parameters, which minimize the incremental phase error, and gives a general method for testing the stability and artificial damping with any other choice of the parameters.

● **6.12-69** Trujillo, D. M., **An unconditionally stable explicit algorithm for structural dynamics,** *International Journal for Numerical Methods in Engineering,* **11,** *10,* 1977, 1579–1592.

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This paper presents an unconditionally stable explicit algorithm for the direct integration of the structural dynamic equations of motion. The algorithm is restricted to a diagonal mass matrix and positive definite symmetric stiffness and damping matrices. The algorithm is based on splitting the stiffness and damping matrices into strictly lower and upper triangular form. Unconditional stability is proven, but only for the undamped case and a completely symmetric splitting of the stiffness matrix. An alternate splitting method is also presented. Numerical examples indicate superior performance over the symmetric splitting, but only a conditional stability. A spring-mass-dashpot model is used to illustrate the algorithm.

● **6.12-70** Chugh, A. K., **Stiffness matrix for a beam element including transverse shear and axial force effects,** *International Journal for Numerical Methods in Engineering,* **11**, *11,* 1977, 1681–1697.

A simple and direct procedure is presented for the formulation of an element stiffness matrix on element coordinates for a beam member and a beam-column member including shear deflections. The resulting stiffness matrices are compared with those obtained using the alternative formulation in terms of member flexibilities. The relative effects of axial force and shear force on the stiffness coefficients are presented. The critical buckling loads, considering the effects of shear force, are computed and compared with those available in the literature. Only prismatic members are considered.

● **6.12-71** Kalev, I. and Gluck, J., **Elasto-plastic finite element analysis,** *International Journal for Numerical Methods in Engineering,* **11**, *5,* 1977, 875–881.

An elastoplastic finite element small displacement procedure to predict the behavior of structures under cyclic loads is presented. The solution combines incremental procedure with correction for equilibrium and iterative scheme. Applications are made for a rectangular strip and a notched bar under cyclic axial loads. Flow theory of plasticity with either isotropic hardening or kinematic linear hardening is used.

● **6.12-72** Lopez, L. A., **FINITE: an approach to structural mechanics systems,** *International Journal for Numerical Methods in Engineering,* **11**, *5,* 1977, 851–866.

FINITE is a general purpose structural mechanics system which runs under the POLO II supervisor. It is unique in that it provides users with a very flexible recursive structural modeling capability. Engineers can communicate with FINITE through a problem-oriented language similar to STRUDL. Significant savings in computer and data entry times can be achieved when its recursive substructuring and condensation features are used appropriately. The structure of FINITE permits engineers to enter new elements with a minimal knowledge of the system operation. Nonsymmetric elements, hybrid elements, and elements with different degrees-of-freedom at the various nodes are all permissible. Various forms of linear constraints may be applied to the generalized displacements of a structure. Since FINITE runs under the POLO II supervisor, FINITE derives many of its attributes from the unique data base management facilities of the POLO II system. Among them are a limited working set size in a virtual memory environment and automatic restart capabilities.

● **6.12-73** Kaul, M. K., **Determination of eigenvalues of large structural systems in an arbitrarily specified range,** *International Journal for Numerical Methods in Engineering,* **11**, *5,* 1977, 867–874.

For a large structural system of small bandwidth, a technique combining linear interpolation on the characteristic polynomial and suppression of its previously determined roots by deflation can be used to determine its eigenvalues. The eigenvalues, however, have to be found, beginning from either the lowest (or the highest) in order to obtain monotonic convergence to the polynomial roots. A method which eliminates this restriction is presented in this paper. If eigenvalues greater than a are desired, the method consists of modifying the deflation procedure to suppress all the eigenvalues smaller than a without actually determining them. Some consequences of the procedure developed in this paper are also presented, and it is shown that the standard deflation technique is a special case of this procedure. The method has been applied to a wide range of problems with success and considerable saving in computation time.

● **6.12-74** Mondkar, D. P. and Powell, G. H., **Finite element analysis of non-linear static and dynamic response,** *International Journal for Numerical Methods in Engineering,* **11**, *3,* 1977, 499–520.

The paper presents the theoretical and computational procedures which have been applied in the design of a general purpose computer code for static and dynamic response analysis of nonlinear structures. A general formulation of the incremental equations of motion for structures undergoing large displacement finite strain deformation is first presented. These equations are based on the Lagrangian frame of reference in which constitutive models of a variety of types may be introduced. The incremental equations are linearized for computational purposes, and the linearized equations are discretized using isoparametric finite element formulation. Computational techniques, including step-by-step and iterative procedures, for the solution of nonlinear equations are discussed, and an acceleration scheme for improving convergence in constant stiffness iteration is reviewed. The equations of motion are integrated using Newmark's generalized operator, and an algorithm with optional iteration is described. A solution

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strategy defined in terms of a number of solution parameters is implemented in the computer program so that several solution schemes can be obtained by assigning appropriate values to the parameters. The results of analysis of a few nonlinear structures are briefly discussed.

● **6.12-75** Minich, M. D. and Chamis, C. C., **Doubly-curved variable-thickness isoparametric heterogeneous finite element,** *Computers & Structures,* 7, 2, Apr. 1977, 295-301.

This paper describes an element streamlined for the analysis of doubly curved, variable-thickness structural components and illustrates its effective application to vibration and static problems. The element is isoparametric, doubly curved, thin-shell, and triangular with variable thickness and accounts for anisotropic, inhomogeneous elastic material behavior. The element has six nodes (three corner and three midside) with five degrees-of-freedom per node—three translations and two rotations. Quadratic isoparametric interpolation polynomials are used to express the element geometry and displacement variables in terms of corresponding nodal variables.

● **6.12-76** Petersson, H. and Popov, E. P., **Substructuring and equation system solutions in finite element analysis,** *Computers & Structures,* 7, 2, Apr. 1977, 197-206.

When dealing with large structural systems, it is convenient and often almost necessary to employ a substructure technique in order not to exceed the available computer capacity. Efficient solution of systems of equations is very essential in finite element analysis. In this paper, first, a procedure is described as to how to systematically reduce the number of assembled equations on different substructure levels. The size of a system of equations to be solved at a time is reduced by condensation of different types of unknowns at the element level. Second, a generalized Gaussian elimination scheme is presented for the solution of systems of equations in substructuring. The same scheme may be very useful in solutions of mixed finite element models, equation systems with Lagrangian multipliers, and moderately ill-conditioned problems. The system properties and the initially neatly banded forms of system equations are preserved during the solution process. Lastly, three examples illustrate solutions of some complex structural problems. The method presented is applicable to a wide range of physical problems and can achieve a substantial computer economy.

● **6.12-77** Park, K. C., **Practical aspects of numerical time integration,** *Computers & Structures,* 7, 3, June 1977, 343-353.

This paper examines some practical aspects of the numerical time integration of structural equations of motion. The concept of stability region in the complex frequency/time-step plane is proposed as the basis of stability evaluation, as opposed to the concept of conditional (or unconditional) stability. Stabilization of numerical computations via the introduction of artificial damping and the composition of existing schemes is discussed. It is shown that stabilization by artificial damping fails for any explicit scheme contrary to the notion that a frequency-proportional damping would suppress the high-frequency components. It is also shown that among explicit schemes examined in this paper the central difference scheme is preferred from the stability considerations. For nonlinear problems, the linearly extrapolated pseudo-force solution procedure is adopted to assess how nonlinearities affect the stability of implicit schemes. Finally, the impact of stability and accuracy characteristics upon numerical computation is discussed.

● **6.12-78** Stricklin, J. A. and Haisler, W. E., **Formulations and solution procedures for nonlinear structural analysis,** *Computers & Structures,* 7, 1, Feb. 1977, 125-136.

This paper presents a survey of the formulations and solution procedures for nonlinear static and dynamic structural analysis. The formulations covered include the pseudo-force method, the total Lagrangian method, the updated Lagrangian method, and the convected coordinate method. The relationship of each principle to the basic principle of virtual work is presented. For static analysis, the solution by direct minimization of the total potential, Newton-Raphson and modified Newton-Raphson, and the first- and second-order self-correcting method are reviewed and put in proper perspective. It is concluded that the most efficient methods for static problems are the modified Newton-Raphson and the first-order self-correcting methods. For dynamic nonlinear analysis, a new method based on modal analysis using the pseudo-force method is presented. Numerical results for the highly nonlinear dynamic response of a shallow cap under a step load at the apex shows the method to be five times faster than the Houbolt solution procedure. Other methods surveyed include the Newmark β method, the Wilson method, central differences, and the stiffly stable solution procedure of Park.

● **6.12-79** Witham, C. R. and Dubowsky, S., **An improved symbolic manipulation technique for the simulation of nonlinear dynamic systems with mixed time-varying and constant terms,** *Journal of Dynamic Systems, Measurement, and Control,* 99, 3, Sept. 1977, 157-166.

The time domain behavior of nonlinear dynamic systems often is obtained by numerical integration on the digital computer. These solutions are usually expensive and

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limit the scope of the dynamic study. The proposed improved technique results in a substantial increase in the computational efficiency by using automatic symbolic manipulation to generate explicit equations of motion algebraically prior to numerical integration. A model is presented which considers the effects of the presence of time-invariant and time-varying symbolic terms and of the sparsity of system elements to provide analytical guidelines for the use of this technique. A number of case studies including typical computational costs are presented.

● 6.12-80 Komaroff, N., **Performance standards in dynamics,** *Journal of Dynamic Systems, Measurement, and Control,* **99,** *2,* June 1977, 118–122.

A method to evaluate the performance of dynamic systems governed by ordinary differential equations is presented. It is based on averaging functions describing system behavior (e.g., velocities) over prescribed domains (e.g., surfaces) in phase space. Quantitative measures of motion are introduced to indicate how oscillatory or how monotone would be the response following a disturbance. Examples demonstrate how these measures serve as new design specifications, the role of which is to define, compare, and control system performance in a more comprehensive manner. Another application of the work is to qualitative studies in the analysis and synthesis contexts.

● 6.12-81 Kamel, H. A. and McCabe, M. W., **Applications of GIFTS III to structural engineering problems,** *Computers & Structures,* **7,** *3,* June 1977, 399–415.

The paper describes the latest version of the GIFTS system (Graphics Oriented Interactive Finite Element Package for Time-Sharing), due for release at the end of Mar. 1976. The paper gives a description of the program modules available in the GIFTS library and the options available within its framework. Examples are given to demonstrate the use of GIFTS in design-oriented applications. Some performance measurements are included. Among the GIFTS features described are: (1) automatic model and load generation; (2) generation and display of higher order elements; (3) display of model, loads, boundary conditions, deflections, and stresses; (4) suitability for use as a pre- and post-processor for large batch programs; (5) static analysis capabilities for trusses, frames, membranes, plates, and shells; (6) vibrational mode computation for complex structures; (7) transient response analysis using either direct integration or a modal analysis approach; (8) substructuring and constrained substructuring; (9) mixed boundary conditions; and (10) thermal stresses. Three examples have been chosen for inclusion in the paper.

● 6.12-82 Hopper, C. T. and Williams, F. W., **Mode finding in nonlinear structural eigenvalue calculations,** *Journal of Structural Mechanics,* **5,** *3,* 1977, 255–278.

The eigenvalue problems resulting from stiffness matrix formulations of structural vibration and buckling problems are nonlinear, if substructures are analyzed exactly, or if classical frequency (vibration problems) or load factor (buckling problems) dependent member equations are used. This makes rapid calculation of accurate free vibration or buckling modes difficult. This paper presents several techniques which might overcome this difficulty, examines them theoretically and experimentally, and gives some of the ways in which the more successful techniques can be incorporated in mode finding methods. Coincident eigenvalues (i.e., natural frequencies or critical load factors) are included.

● 6.12-83 Gourlay, A. R., McLean, J. M. and Shepherd, P., **Identification and analysis of the subsystem structure of models,** *Applied Mathematical Modelling,* **1,** *5,* June 1977, 245–252.

Two possible approaches to the problem of determining a suitable subsystem breakdown for a dynamic model are described. They are based on the manipulation and subsequent processing of a system interaction matrix which indicates the relationships which exist between the model variables. Examples of the application of both techniques to existing models are presented and discussed.

● 6.12-84 Felippa, C. A., **Numerical experiments in finite element grid optimization by direct energy search,** *Applied Mathematical Modelling,* **1,** *5,* June 1977, 239–244.

A series of numerical experiments is conducted to assess the feasibility and practical value of finite element grid optimization based on direct minimization of the total potential energy of the discrete model with respect to the node locations. An implementation relying upon nonlinear programming techniques is found to be numerically reliable and to lead to improved grids in accord with engineering intuition. This rigorous approach is hampered, however, by the excessive computational effort required by the energy minimization process. A combination of related techniques is therefore proposed to make dynamic node distribution a useful tool within the framework of large-scale finite element analysis. The combined strategy involves use of substructuring methods, application of a local energy-balancing optimality criterion for fast node distribution, and automatic refinement of previously improved coarse grids.

● 6.12-85 Fraser, D. J., **Equivalent frame method for beam-slab structures,** *Journal of the American Concrete Institute,* **74,** *5,* Title No. 74-24, May 1977, 223–228.

Codes of practice for reinforced concrete buildings permit the use of an equivalent frame for the analysis of real beam-and-slab buildings. The success of the idealization requires that realistic data be available concerning the

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effective size of the equivalent beams and the initial joint moments corresponding to the type of load on the beam-slab structure. Many different rules appear in codes and other references for the evaluation of this information. From an extensive investigation of beam-slab structures, based on finite element analyses, simple rules are proposed for the evaluation of the effective flexural stiffness of the equivalent beams. Also, initial joint moments for three types of floor-beam loads may be evaluated.

● **6.12-86** Allen, F. and Darvall, P., **Lateral load equivalent frame,** *Journal of the American Concrete Institute,* 74, 7, Title No. 74-32, July 1977, 294–299.

Theoretical effective width coefficients for flat plate structures are presented for a wide range of plate aspect ratios and column cross-section aspect ratios. Experimental results are given which confirm the theoretical values of the effective width coefficients. A new lateral load equivalent frame is proposed that uses the theoretical effective width coefficient. Three hypothetical buildings under lateral loads are analyzed for story sways using the various equivalent frames permitted by the codes and are compared to the results for the proposed lateral load equivalent frame. It is shown that the full-panel-width equivalent frame may seriously underestimate lateral load sways if the column stiffnesses are not reduced.

● **6.12-87** Cassis, J., **An efficient approach for the dynamic sensitivity analysis,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1034–1039.

An efficient new approach for the dynamic sensitivity analysis of structures is introduced. The method computes expressions of exact derivatives of structural response functions with respect to design variables. The proposed method is compared with the traditional approximate finite differences approach and with another exact method previously introduced. Examples applied to building frames subjected to ground motion show the efficiency of the proposed solution.

● **6.12-88** Mondkar, D. P. and Powell, G. H., **General purpose computer program for dynamic nonlinear analysis,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1154–1159.

A brief description of the concepts, features, organization, and usage of a general purpose finite element computer program, ANSR (Analysis of Nonlinear Structural Response), for the static, dynamic, and earthquake response analysis of nonlinear structures is presented. The program is intended ultimately to form a practical analysis tool for use in design studies as well as to satisfy research needs in various aspects of nonlinear analysis.

● **6.12-89** Nathan, N. D., Cherry, S. and McKevitt, W. E., **Structural response to simultaneous multi-component seismic inputs,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1160–1165.

A study of the influence of simultaneous multicomponent seismic inputs on structural response is reported. Rectangular and cruciform plan ten-story reinforced concrete buildings were subjected to a combination of input acceleration components, including torsional ground accelerations, corresponding to several different earthquakes. The structures were analyzed as elastic three-dimensional space frames, and time-history studies were made of the external and re-entrant corner column seismic forces developed along the heights of the structures. It was found that multicomponent inputs resulted in axial forces up to the sum of the maximum single-component values. Shears and moments were increased as a result of torsional ground motion, eccentricity, and "tube" action. Preliminary design recommendations for dealing with multicomponent inputs are included.

● **6.12-90** Ozden, K., **Analysis of shearwall-frame systems,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1191–1194.

This paper shows that a shear wall-frame system can be substituted by a virtual system consisting of a shear wall and infinitely rigid floor beams jointed to the shear wall at its axis on one end with springs defined by their bending stiffness and supported by sliding supports on the other end. The internal forces of this virtual system can be calculated by using a set of equations written for the moments on the shear wall sections just above the floor beams. This set of equations is similar to the three moment equations. The internal forces of the real system can easily be found afterwards.

● **6.12-91** Tso, W. K. and Biswas, J. K., **Seismic analysis of asymmetrical structures subjected to orthogonal components of ground acceleration,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1217–1222.

A procedure to compute the seismic responses of asymmetric structures subjected to two orthogonal components of ground motion is presented. The procedure is an extension of the response spectrum technique for structures under unidirectional excitation. This method is applied to a realistically proportioned asymmetric L-shaped building subjected to two horizontal components of the 1940 El Centro earthquake. Its accuracy is checked with results obtained using time-history dynamic analysis.

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● **6.12-92** Monahenko, D. V., **Principles of physical modelling of structures resistant to earthquakes,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2856–2860.

Presented are basic principles and an algorithm for physically modeling linear boundary value problems. Given are examples of the application of the theory for solving structural seismic stability problems by model tests. The conditions which allow simplified modeling of seismic impacts, material properties, and structural geometry are described.

● **6.12-93** Tang, D. T. and Clough, R. W., **Mathematical modeling of a steel frame structure,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2741–2744.

By comparing the computed dynamic response with the performance of a three-story steel frame structure observed during shaking table tests, three different mathematical models of the structure are evaluated. The testing was done in two phases: first, with underdesigned joint panel zones and, second, after reinforcing the panel zones. The study demonstrates that a rationally formulated mathematical model can predict adequately the linear and nonlinear seismic response of steel frames.

● **6.12-94** Takizawa, H., **Biaxial and gravity effects in modeling strong-motion response of R/C structures,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1022–1027.

A mathematical formulation is presented for modeling the process of dynamic failure of R/C flexural (flexural-shear) columns subjected to lateral biaxial deformation. The model considered is a two-dimensional extension of the degrading quadrilinear hysteresis concept that accounts for the prominent effects of cracking, yielding, crushing and spalling, and stiffness degradation. By including another factor of gravity (the P-delta effect) responsible for destabilization, studies are conducted to clarify the role of particular factors in determining the capacity to resist intense seismic shaking in the horizontal plane. Significance of the combined effects of biaxial deterioration and gravity is emphasized.

● **6.12-95** Hisada, T. and Igarashi, K., **Evaluation of the effects of earthquake motions on structures based on elasto-plastic response envelope spectrum with time-domain,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 477–482.

For the evaluation of the effects of earthquake motions on buildings and structures, the effects of repeated cyclic loading need to be considered. Assessment of cyclic loading effects is essential for reinforced concrete structures which collapse as a result of shear failure. The engineering characteristics of earthquake motions are usually represented by the structural response spectra. However, no information of the effects of the earthquake motion on structures along a time domain is furnished by these spectral figures. In this paper, the authors evaluate the effects of earthquake motions by presenting an elastoplastic response envelope spectrum with time domain.

● **6.12-96** Sabnis, G. M. and White, R. N., **Small scale models of concrete structures subjected to dynamic loads,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2766–2771.

The use of small-scale models to predict the behavior of reinforced concrete structures has become increasingly important in recent years because of the complexity of many modern structures and the need to understand their behavior in the inelastic range. A well-known example is the prestressed concrete pressure vessel for nuclear applications, where the only feasible method for investigating the effects of severe loading combinations is with scaled models. This paper presents the state-of-the-art in the field of models of concrete structures in dynamic loading applications. The major problems met in dynamic modeling include (a) materials that meet similitude conditions, (b) proper simulation of mass, (c) loading methods, and (d) strain measurements in the reinforcing and model concrete. These topics are discussed, along with selected applications of dynamic and pseudostatic modeling. It is concluded that small-scale structural models are highly useful for studying certain types of dynamic response, both elastic and inelastic.

● **6.12-97** Wang, W. Y. L. and Goel, S. C., **Prediction of maximum structural response by using simplified accelerograms,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1004–1009.

Maximum displacements of single degree-of-freedom systems in the elastic and inelastic ranges caused by earthquake motion are calculated by an economical method. The efficiency of this method is achieved by constructing a four-rectangular pulse model accelerogram of short duration. This model accelerogram retains the intensity and the frequency contents of the actual ground motion. Examples of this method are given by constructing model accelerograms for two parent ground motions–El Centro 1940 NS magnified twice and Pacoima Dam 1971 S14W. Both elastic and inelastic displacement response spectra calculated from these model accelerograms are close to those

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calculated from the parent accelerograms. One application of this method is presented to construct a model accelerogram from an elastic design spectrum. The inelastic response obtained using this design model accelerogram are compared with those obtained using current methods.

6.12-98 Monforton, G. R. and Ibrahim, I. M., **Modified stiffness formulation of unbalanced anisotropic sandwich plates,** *International Journal of Mechanical Sciences,* **19,** *6,* 1977, 335–343.

An approximate formulation is presented for the analysis of sandwich plates consisting of an orthotropic core and two anisotropic face plates of unequal thickness. The method uncouples the membrane and bending actions, thereby significantly reducing the effort involved in accurately predicting displacements and stresses. In the formulation, modified stiffnesses are defined. The formulation is valid for relatively thick anisotropic laminated face plates and can be combined with a variety of analysis techniques. Using a series solution, results are generated for simply supported sandwich plates with unbalanced cross-ply and angle-ply faces.

● 6.12-99 Schiff, A. J., Feil, P. J. and Newsom, D. E., **Computer simulation of lifeline response to earthquakes,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3298–3303.

A method is described for using digital computer simulation to evaluate the response of lifelines to earthquakes. Information gained from the simulation enables seismic specifications of equipment and facilities to be evaluated in a system context. This capability is of particular importance for power, communication, and other lifeline systems since these systems are highly redundant. Information is also gained for improving the postearthquake recovery process. While the general methodology could be applied to all ground-based lifelines, parts of the computer code are tailored to model electrical power systems.

● 6.12-100 Zilch, K. and Lappas, G., **Response and stresses of structures subjected to two-directional earthquake excitation,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 2079–2084.

Earthquake ground motions propagate in more than one horizontal direction; they are recorded in two orthogonal axes. Maximum resulting stresses from a two-dimensional earthquake response are derived by integrating recorded time histories and comparing them with combinations of one-dimensional results. It can be shown that the shape of the cross section is an essential factor. As a model of a building, a linear elastic clamped beam with a lumped mass is used. Several cross-sectional shapes, representing the possible arrangement of horizontal load-resisting elements, are considered.

● 6.12-101 Blockley, D. I., **Analysis of structural failures,** *Proceedings, The Institution of Civil Engineers,* Part 1, **62,** Feb. 1977, 51–74.

In Part I of this article a classification of basic types of structural failure is presented. This classification is expanded into a set of parameter statements which could be assessed subjectively in a prediction process. This process is intended to account for a structure failing due to causes other than stochastic variations in load and strength. The parameters are assessed for twenty-three major structural accidents and one existing structure, and are analyzed using a simple numerical interpretation. The accidents are ranked in order of inevitability. Human error proved to be the dominant reason for the failures considered. A simplified form of the proposed procedure for predicting the likelihood of structural accidents is outlined in Part II. This is then applied to the twenty-four accident parameter assessments made in Part I. The concept of fuzzy sets is used.

● 6.12-102 Schapiro, S. M. and Sethna, P. R., **An estimate for the small parameter in the asymptotic analysis of nonlinear systems by the method of averaging,** *International Journal of Non-Linear Mechanics,* **12,** *3,* 1977, 127–140.

The method of averaging is an asymptotic method that can be used to obtain approximate solutions for many parameter dependent nonlinear systems. The resulting approximate solutions are as accurate as desired provided the system parameter is sufficiently small. In this paper, a general technique is developed to obtain a relationship between the magnitude of the small parameter and the permissible error in the approximate solution. The technique is then demonstrated by its application to two examples.

● 6.12-103 Singh, M. P., Singh, S. and Ang, A. H.-S., **Extended applications of response spectra curves in seismic design of structures,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1747–1752.

Methods are presented for obtaining accurate response of structures with closely spaced frequencies, generation of floor spectra, and analysis of secondary systems with multiple supports for seismic design. The methods are based on stochastic concepts, but no stochastic seismic input is required; only structural quantities, such as system frequencies, mode shapes, participation factors, and site spectra curves are required.

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6.12-104 MacLeod, I. A., **Structural analysis of wall systems,** *The Structural Engineer,* **55,** *11,* Nov. 1977, 487–495.

Significant developments in techniques for analyzing complex wall systems have occurred over the past 15 years. These developments are reviewed and the relative importance of various parameters is discussed. The behavior and analysis of both plane and 3-dimensional wall systems is considered.

● **6.12-105** Lukkunaprasit, P. and Kelly, J. M., **Dynamic plastic analysis using stress resultant finite element formulation,** *UCB/EERC-77/21,* Earthquake Engineering Research Center, Univ. of California, Berkeley, Sept. 1977, 59. (NTIS Accession No. PB 275 453)

A stress-resultant finite element formulation is developed for the dynamic plastic analysis of plates and shells of revolution undergoing moderate deformation. A nonlinear elastic-viscoplastic constitutive relation simulates the behavior of rate-sensitive and insensitive materials. A local time-step subdivision procedure is developed to stabilize the direct numerical integration of the system of nonlinear dynamic equations; satisfactory accuracy is obtained with large time steps. The simple nonlinear viscoplastic constitutive model approximates the nonlinear dynamic behavior of metals over a wide range of strain rates and has the advantage that the need to identify the state of the material during deformation is eliminated and the numerical algorithm thereby simplified. Direct step-by-step integration techniques are used to solve the system of equations governing the motion of a structure under dynamic loading. An implicit Runge-Kutta scheme, in conjunction with a Newton-Raphson iteration technique, is used in solving systems of first-order ordinary differential equations.

● **6.12-106** Quang Le, D., Petersson, H. and Popov, E. P., **SUBWALL: a special purpose finite element computer program for practical elastic analysis and design of structural walls with substructure option,** *UCB/EERC-77/09,* Earthquake Engineering Research Center, Univ. of California, Berkeley, Mar. 1977, 94. (NTIS Accession No. PB 270 567)

An efficient and refined special purpose finite element computer program has been developed for the linear structural analysis and design of complex reinforced concrete walls subjected to arbitrary in-plane static loadings. A substructuring technique has been implemented, along with several practical user's options, which contribute to the computational efficiency and economy unavailable in many general purpose computer programs.

Large structural walls with multiple openings, nonplanar coupled walls, and staggered wall-beam systems can be analyzed. Openings and offsets in structural walls are represented by special "hole" elements with no structural stiffness. Preparation of input is simple. Few cards are necessary for a relatively large-sized structure. Joint coordinates, element connectivities, and boundary conditions are automatically generated.

Output can be requested in terms of tabular printout, element by element printer plots and/or regular plotter plots of displacements, stresses, and reactions. Section forces such as shear, moment, and normal forces in connecting beams can be obtained. The main emphasis of the report is on the practical and economical application of the finite element method rather than on the theoretical aspect of the program development. Throughout the report, linearly elastic behavior of the materials is assumed.

● **6.12-107** Guendelman-Israel, R. and Powell, G. H., **DRAIN-TABS: a computer program for inelastic earthquake reponse of three-dimensional buildings,** *UCB/EERC-77/08,* Earthquake Engineering Research Center, Univ. of California, Berkeley, Mar. 1977, 146. (NTIS Accession No. PB 270 693)

A computational procedure and computer program for the inelastic dynamic response analysis of three-dimensional buildings of essentially arbitrary configurations are described. A building is idealized as a series of independent plane substructures interconnected by horizontal rigid diaphragms. Each substructure can be of arbitrary geometry and include structural elements of a variety of types. It is not necessary for all substructures to connect to all diaphragms so that structures with independent diaphragms at some levels can be idealized (as, for example, two towers connected only at the top floors). The analysis makes use of substructuring techniques to improve computational efficiency. The major limitation is that the coupling of the substructures through common columns is not fully taken into account so that the idealization is not suitable for tube-type buildings.

The program consists of a base program which reads and prints data for the structure and its loading, allocates storage, carries out a variety of bookkeeping operations, assembles the substructure and building stiffnesses and loadings, solves the equilibrium equations, and determines the displacement response. This base program is then combined with a library of element subroutines to produce the complete program. Subroutines for new elements can be developed independently and added to the element library with relative ease. Subroutines for truss, beam column, shear panel, semirigid connection, and beam elements are currently included.

The structure idealization is explained, and the computational procedure and computer program logic are described. Instructions to be followed when adding new

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elements to the program are presented. A computer program user's guide and an illustrative example are included.

6.12-108 Iesan, D., **Reciprocal theorems and variational theorems in nonlocal elastodynamics**, *International Journal of Engineering Science*, **15**, *12*, 1977, 693–699.

The paper is concerned with the linear theory of nonlocal elastodynamics for inhomogeneous and anisotropic solids. Reciprocal and variational theorems are established.

● 6.12-109 Norrie, D. H. and De Vries, G., comps., **Finite element bibliography**, IFI/Plenum Data Co., New York, 1976, 686.

The bibliography covers the period 1956–1975 primarily, although some earlier publications of historical interest are included. The citations are not restricted to the English language and documents are listed in many languages and from diverse places of origin. All publication formats were accepted, so that references will be found to books, monographs, journal papers and articles, theses, dissertations, reports, surveys, and the like. Although this bibliography is not complete, it does include 7115 citations and is believed to reference the larger portion of the literature published in this area. Some references are included which are not specific to the finite element method but are closely related, such as matrix structural analysis, computational procedure, and automatic mesh generation.

6.12-110 Peters, K. A., Schmitz, D. and Wagner, U., **Determination of floor response spectra on the basis of the response spectrum method**, *Nuclear Engineering and Design*, **44**, *2*, Nov. 1977, 255–262.

In this paper a method to determine floor response spectra is proposed which is based on the modal analysis of a support structure with an interaction-free, one degree-of-freedom system attached. The time-consuming methods using real or artificial soil accelerations are avoided as well as some of the arbitrariness in the approaches of Biggs or Kapur-Shao.

6.12-111 Gupta, A. K. and Chu, S. L., **Probable simultaneous response by the response spectrum method of analysis**, *Nuclear Engineering and Design*, **44**, *1*, Oct. 1977, 93–95.

A new method is proposed by which any response at any location in a structure which is expected to occur simultaneously with a maximum probable response at a given (or reference) location can be calculated. The application of this method is illustrated for vertical displacement calculations in a base slab. It is shown that a clear displacement pattern emerges by the use of the proposed method.

6.12-112 Gupta, A. K. and Chu, S. L., **Equivalent modal response method for seismic design of structures**, *Nuclear Engineering and Design*, **44**, *1*, Oct. 1977, 87–91.

A new method is proposed in which the response in several modes of vibration under the three components of earthquake is replaced by the response in a small number of equivalent modes. The SRSS (square root of the sum of the squares) combination of the response in the equivalent modes yields the same response as the original modes. For the design of a building cross section and the base slab, it is shown that only three equivalent modes are needed. For the design of shear walls, three equivalent modes are needed for the story axial force and the overturning moments (which are common to all the shear walls), and an additional mode is needed for each wall for shear force. The main advantage of the equivalent modal response method is that one can see a relationship between the applied force and moments in any equivalent mode and their effect, thus reducing the chances of calculation errors.

6.12-113 Jeanpierre, F. and Livolant, M., **Direct calculation of floor response spectra from the Fourier transform of ground movement—application to the Superphenix fast reactor project**, *Nuclear Engineering and Design*, **41**, *1*, Mar. 1977, 45–51.

This paper describes a method for the direct calculation of floor response spectra from ground response spectra. The procedure utilizes the Fourier transform of the ground movement. The mathematical derivations are given in detail, and the method is applied to the calculation of the floor response spectra for a structure which is a simplified model of the Superphenix fast breeder reactor power plant.

6.12-114 Wright, W., **The use of conversational computer programs in the structural engineering office**, *Canadian Journal of Civil Engineering*, **4**, *4*, Dec. 1977, 417–435.

As a result of improved facilities, reduced computing costs, and the availability of better programs, conversational computer programs are becoming effective design aids in structural engineering offices. The author outlines his work in writing conversational computer programs and the techniques he has developed to make them successful. Some of the topics given attention are: the relationship between the engineering and the computer science aspects of the work; a technique for free-format data input; the planning of the dialog to take place between the program users and the computer; the provision of technical support for the program users; the selection of the correct computer and the correct terminal for conversational programs; procedures for error checking, error correction, and input data editing; electronic data transmission from program to program; development of the programs in a time-sharing

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environment; the use of modular programming; and trade-offs between the costs of the software and the hardware to support conversational programs. A limited amount of attention is given to the role of computer graphics in support of conversational computer programs.

6.12-115 Firmin, A., Gilmor, M. I. and Collins, R. A., **Computer programs for structural analysis and design—a Canadian view,** *Canadian Journal of Civil Engineering,* **4,** *1,* Mar. 1977, 72–85.

In recent years there has been a rapid expansion in the number of structural analysis and design computer programs commercially available to the design engineer. This paper summarizes the current state of the art of this software and outlines some of the major features. A broad coverage of the practical considerations surrounding the use of these programs is followed by a description of individual programs and their uses. Only those programs known to be readily available through Canadian computer bureaus are included in the review.

● **6.12-116** Felippa, C. A. and Park, K. C., **Computational aspects of time integration procedures in structural dynamics: Part I: implementation. Part II: error propagation,** *LMSC D556247,* Lockheed Palo Alto Research Lab., Palo Alto, California, Jan. 1977, 116.

A unified appr:ach for the implementation of direct time integration procedures in structural dynamics is presented. Two key performance assessment factors are considered: computational effort and error propagation. It is shown that these factors are strongly affected by details in the reduction of the second-order equations of motion to a system of first-order equations and by the computational path followed at each time step. Part I is primarily devoted to the study of the organization of the computational process. Specific implementation forms derived from the unified approach are studied in detail and rated accordingly.

● **6.12-117** Newmark, N. M., Hall, W. J. and Morgan, J. R., **Comparison of building response and free field motion in earthquakes,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 972–978.

Structures on large foundations generally suffer less damage from earthquakes than those on smaller foundations. Studies of records suggest that accelerations imparted to large structures approach an average of the free field excitation over a "transit time" related to the size of the building. A numerical technique for averaging the acceleration is presented. The resulting structural response may be reduced by a factor of two or three from the free field spectral response in the high-frequency region. This reduction can significantly affect structural design. Torsional motion is also discussed.

● **6.12-118 Seismic analysis by computer,** Structural Engineers Assn. of Southern California, Los Angeles, 1977, var. pp.

Dynamic seismic analysis of structures is now required by some building codes for hospitals, vital community facilities, irregular or unusual structures, and major high-rise buildings; and some builders are interested in greater protection of their investment than can be provided by a design utilizing current static code criteria. These guidelines have been developed in an attempt to clarify the more complex, computer-generated dynamic seismic analysis process and to define its documentation requirements. Much of the material presented is equally applicable to computer-generated static seismic analysis.

● **6.12-119** Wohlen, R. L., **Expansion and improvement of the FORMA system for response and load analysis. Vol. IIIA-explanations, dense FORMA subroutines,** *MCR-76-217,* Martin Marietta Corp., Denver, May 1976, 300.

Techniques are presented for the solution of structural dynamic systems on an electronic digital computer using FORMA (FORTRAN Matrix Analysis). FORMA is a library of subroutines coded in FORTRAN IV for the efficient solution of structural dynamics problems. The subroutines are in the form of building blocks that can be assembled to solve a large variety of structural dynamics problems. The advantage of the building block approach is that the programming and the checkout time is limited to that required for assembling the blocks in the proper order. Other advantages of the FORMA method include (1) the extensive use of the subroutines; (2) the applicability of the method to any computer with a FORTRAN IV compiler; (3) the incorporation of new subroutines without problem; and (4) the basic FORTRAN statements, which allow extreme flexibility in program writing. Two programming techniques, dense and sparse, are used.

● **6.12-120** Curiskis, J. I. and Valliappan, S., **A solution algorithm for linear constraint equations in finite element analysis,** *UNICIV Report No. R160,* School of Civil Engineering, Univ. of New South Wales, Kensington, Australia, Sept. 1976, 21.

A general solution algorithm is presented for the incorporation of a general set of linear constraint equations into a linear algebraic system. Implementation of the algorithm, without the need for pre-arranging the equations, into an equation solving method using Gauss elimination is developed.

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● **6.12-121** Bathe, K.-J. and Wilson, E. L., **Numerical methods in finite element analysis,** Prentice-Hall, Inc., Englewood Cliffs, New Jersey, 1976, 528.

The aspects of finite element analysis are examined in order to provide a basis for the understanding of the complete solution process. The numerical aspects of the method are emphasized, but whenever possible physical explanations are given. The book is divided into three parts. In the first part, important concepts of matrix and linear algebra are presented. The second part of the book comprises the formulation of the finite element method and the numerical procedures used to evaluate the element matrices and the matrices of the complete element assemblage. General principles are established and those elements that are believed at present to be most effective are described. In the last part of the book, procedures for the effective solution of the finite element equilibrium equations in static and dynamic analysis are presented.

Principles and procedures are illustrated with over 100 examples. Short computer programs are also included to demonstrate the numerical procedures. These programs can be used directly as subroutines in finite element codes.

● **6.12-122** Perrone, N. and Pilkey, W., eds., **Structural mechanics software series, Vol. I,** Univ. Press of Virginia, Charlottesville, 1977, 630.

The primary objective of the Structural Mechanics Software Series is to provide the technical community with information on structural analysis and design computer programs on a continuing basis. The first part of this volume contains information on directly callable programs, and the second part deals with program surveys. A subject index of programs and an alphabetical index are included. Here follow the contents of the volume:

Part I: Software Series Library of Computer Programs. *Library of available software series programs. BOSOR4: program for stress, buckling, and vibration of complex shells of revolution,* Bushnell, D.—*GIFTS: graphics oriented interactive finite element time-sharing system,* Kamel, H. A. and McCabe, M. W.—*Interactive and data mode preprocessor for SAP,* Kaldjian, M. J.—*TOTAL: interactive graphics system for the two-dimensional analysis of linear elastic solids,* Beaubien, L. A.—*BEAM: a program for the static, stability, and dynamic response of beams,* Pilkey, W. D. and Chang, P. Y.—*BEAMSTRESS: a program for determining the cross-sectional properties and stresses of a bar,* Pilkey, W. D., Chang, P. Y. and Thasanatorn, C.—*SHAFT: a program for the unbalanced response and critical speeds of rotating shafts,* Pilkey, W. D. and Chang, P. Y.

Part II: Reviews and Summaries of Available Programs. *Computer-aided building design,* Goel, S. K., Waddick, R. K. and Beck, C. F.—*Curved girder bridge systems,* Schelling, D. R., Heins, C. P. and Sikes, G. H.—*Symbolic and algebraic manipulation languages and their applications in mechanics,* Jensen, J. and Niordson, F.—*Floor analysis and design,* Vanderbilt, M. D.—*Three-dimensional gross-motion crash victim simulator,* Huston, R. L.

6.13 Nondeterministic Methods of Dynamic Analysis

6.13-1 Karimova, R. Kh., **Calculation of the reliability of suspended systems subjected to seismic excitations,** (Otsenka nadezhnosti visyashchikh sistem pri seismicheskikh vozdeistriyakh, in Russian), *Trudy Frunzenskogo politekhnicheskogo instituta, 87,* 1976, 89–97.

In order to determine the probabilistic characteristics of suspended systems the stochastic method is employed to construct and solve the Fokker-Planck-Kolmogorov equations. Using the probabilistic amplitude and phase characteristics obtained, the author has calculated the regular component of the axial force and its variance. Using the techniques of sampling theory, the author has calculated the probability of the system reaching the limiting state during the period of its operation.

● **6.13-2** Petrov, A. A. and Bazilevskii, S. V., **Calculation of seismic loads taking account of the finite duration of earthquakes** (Opredelenie seismicheskikh nagruzok s uchetom konechnoi prodolzhitelnosti zemletryasenii, in Russian), *Seismostoikoe stroitelstvo, 2,* 1977, 23–27.

Nonstationary vibrations of linear systems subjected to nonstationary random excitation are investigated. V. V. Bolotin's model of seismic excitation is employed. Dynamic parameters and seismic forces are calculated.

● **6.13-3** Ozaki, M. and Ishiyama, Y., **An evaluation method for the earthquake resistant capacity of reinforced concrete and steel reinforced concrete columns,** *Wind and Seismic Effects,* VI-10–VI-27. (For a full bibliographic citation, see Abstract No. 1.2-4.)

An evaluation method is proposed for the earthquake-resistant capacity of reinforced concrete and steel reinforced concrete columns by utilizing the force deflection relationship of column specimens subjected to axial force and repeated and reversed lateral loading of considerable intensity. An approximate response analysis for a nonlinear structural system was developed based on random vibration theory and was applied to models represented by a single-degree-of-freedom system subjected to a constant white noise acceleration. The mean expected maximum response values of the models with two different natural periods 0.1

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and 0.5 sec were calculated. Each model has a degrading stiffness system and various hysteretic envelope slopes after four different yield point levels. The viscous damping ratio is considered to be 5% of the critical damping for the entire processes of the models, and the hysteretic damping ratio after yielding is assumed to increase according to the increase of the ductility factor. The ductility factors of the models calculated by the nonlinear response analysis are shown in tables and figures. The maximum strength of the linear model having the equivalent earthquake-resistant capacity of a column specimen can be assumed if the yield point, hysteretic envelope slope tangent, coefficient of hysteretic damping ratio, and ductility factor of the specimen are measured by testing.

A facility was designed for testing large models of reinforced concrete and steel reinforced concrete columns under action simulating gravity loads and ground motion in order to standardize the testing techniques and to forestall possible errors that may be induced by the use of different types of testing facilities. An example of evaluation of the earthquake-resistant capacity of a reinforced concrete column is presented by utilizing the force deflection relationship of the specimen obtained during tests. It is confirmed that yield point level, hysteretic envelope slope, hysteretic damping ratio, and ductility factor are the most important components of earthquake-resistant capacity.

● 6.13-4 Wen, Y.-K., **Statistical combination of extreme loads,** *Journal of the Structural Division, ASCE,* **103,** *ST5,* Proc. Paper 12930, May 1977, 1079–1093.

As most loads on buildings and structures are stochastic in nature, an analytical method for studying load combination is presented in which the loads are treated as random processes. Factors considered include load occurrence rate, intensity variation, duration of each occurrence, and simultaneous occurrence of different loads. The probability distribution of the maximum combined load effect over a given time interval is derived and the distribution parameters as functions of the first two moments and the occurrence rate of individual loads are also obtained in close form. Numerical examples are given and the results are compared satisfactorily with Monte Carlo simulation. The reduction factors for load combination as given in current building codes are examined and their inadequacy pointed out.

● 6.13-5 Schneeberger, B. and Breuleux, R., **Comparison between time-step-integration and probabilistic methods in seismic analysis of a linear structure,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(a), Paper K 4/6, 11. (For a full bibliographic citation, see Abstract No. 1.2–5.)

Assuming that earthquake ground motion is a stationary time function, the seismic analysis of a linear structure can be accomplished using such probabilistic methods as the power spectral density function (PSD), instead of applying the more traditional time-step integration using earthquake time histories (TH). A given structure was analyzed by the PSD and TH methods, computing and comparing floor response spectra. For the purpose of having "equivalent" TH and PSD to use in the comparison, both were derived artificially from the U.S. Nuclear Regulatory Commission (USNRC) response spectra. The TH were readily available; the PSD was derived directly from the USNRC response spectra by iteration. The analysis using TH was performed for two different TH and different frequency intervals for the floor-response spectra. The analysis using PSD first produced PSD functions of the responses of the floors and these were then converted into floor-response spectra. Plots of the resulting floor-response spectra show: (1) The agreement of TH and PSD results is quite close. (2) The curves produced by PSD are much smoother than those produced by TH and mostly form an envelope of the latter. (3) The curves produced by TH are quite jagged with the location and magnitude of the peaks depending on the choice of frequencies at which the floor-response spectra were evaluated and on the choice of TH.

● 6.13-6 Sato, H., Komazaki, M. and Ohori, M., **An extensive study on a simple method for estimating response spectrum based on a simulated spectrum,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(a), Paper K 4/9, 11. (For a full bibliographic citation, see Abstract No. 1.2–5.)

The response spectrum is useful for estimating the seismic coefficient in dynamic aseismic design. It was demonstrated in a previous work that the response spectrum for earthquake motions could be simulated by an artificial earthquake with two ground predominant periods. However, the simulation procedure took much time, and it was practically impossible to assume more than two ground predominant periods because of the complexity of the computation. In order to overcome these difficulties, a simple method for estimating the response spectrum of a building and a building-appendage structure was developed. The method is extended to a multistory building structure and appendage. Analysis is made as for a two-story structural system, the mode of which for the first natural frequency is that the amplitude ratio of the upper mass to the lower is 2 to 1, so that the mode shape is a reversed triangle. The behavior of the system is dealt with by the normal coordinate. The amplification factors due to two ground predominant periods are estimated for the system with the first natural frequency. In this procedure, the method developed for the single degree-of-freedom system is directly applicable. The same method is used for the system with the second natural frequency. Thus, an

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estimated amplification factor for the mode of the respective natural frequency is summed using the principle of the square root of sum of squares after multiplying the excitation coefficient of each mode by the corresponding factor. The estimate of the spectrum using this method shows good agreement with the earthquake motions.

The estimate of the amplification factor of the appendage to this building structure is also made. The method used for the simple building-appendage system and that for the multistory building system are combined. The results give very good agreement for this case, too. These investigations show that the response spectrum can be estimated from a statistically evaluated simple spectrum even for multiground predominant periods, if the ground predominant periods, the natural periods of the structure, and the rate of the contribution of the respective components are predicted. The El Centro earthquake motion is used throughout the paper.

● 6.13-7 Scanlan, R. H. and Sachs, K., **Development of compatible secondary spectra without time histories,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(a), Paper K 4/13, 7. (For a full bibliographic citation, see Abstract No. 1.2-5.)

This paper describes a process for developing compatible secondary spectra without time histories. When the process is used for this purpose, it gives results which compare well with those obtained using time histories, though the present results are smoother and more consistent.

● 6.13-8 Fardis, M. N., Cornell, C. A. and Meyer, J. E., **A probabilistic seismic analysis of containment liner integrity,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(a), Paper K 4/16, 12. (For a full bibliographic citation, see Abstract No. 1.2-5.)

After determining the dynamic response of a containment to seismic events well beyond the safe-shutdown-earthquake, the integrity of the liner under the induced stresses is assessed. In a containment composed of a concrete structure with a steel liner plate attached to its interior face, the liner's function is to isolate the containment atmosphere from the environment, even in case the concrete is cracked. To perform this function, the liner must be absolutely leak-tight. The response to earthquake ground motion is found by representing the containment as a distributed mass beam, and considering only the first vibrational mode. Depending on the extent of cracking, the response will be between that of a shear beam and that of a cantilever beam. The stresses at any point are found then from internal forces, as a combination of the stresses from a shell model (with mainly membrane action) and the stresses from a beam model, in which horizontal planes remain plane. The resulting stresses and strains in the liner reveal that although the liner might yield under very high ground accelerations its high ductility will prevent it from tearing.

As the average performance of the liner will be satisfactory, the authors seek potentially localized weak points in it. They identify the seam welds between adjacent liner panels as such points, and weld defects as potential crack initiators. Statistics concerning the size and frequency of occurrence of weld defects in structures similar to the liner plate are used. Assuming that the provisions of the ASME-ACI code, regarding sampling and radiograph examination of welds, are applied, the authors find the frequency of occurrence of various sizes of defects after examination and the cold-pressure test. A defect of given size may become a crack initiator by either brittle fracture or ductile rupture. By using: (1) linear elastic fracture mechanics, (2) experimental results regarding the dynamic fracture toughness of similar steel plates, (3) a probabilty distribution of the liner temperature under operating conditions, and (4) a probability distribution of the liner nil-ductility-transition temperature, the authors find the probability that a given size defect will fracture under given stress. They also find the probability of ductile rupture under given strain, which is found to be negligibly small in comparison to the probability of brittle fracture. By combining the results above, the authors find the probability that a weld point will become a crack initiator under given stress. Assuming independence of weld occurrence, and under given state of stress on the entire liner, the number of crack initiators is Poisson distributed. Statistics regarding the distribution of the peak ground acceleration over different horizontal directions are used. Uncertainty in the structural model is also included. By combining these, the probability distribution of various states of stress over the entire liner, for given recorded peak ground acceleration, is found. The result is the probability distribution of the number of crack initiators, and the probability distribution of the total crack length as a function of peak ground acceleration.

● 6.13-9 Sundararajan, C. and Gangadharan, A. C., **Seismic analysis of structures by simulation,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(a), Paper K 3/2, 11. (For a full bibliographic citation, see Abstract No. 1.2-5.)

The paper presents a state-of-the-art survey and recommendations for future work in the area of stochastic seismic analysis by Monte Carlo simulation. First, the Monte Carlo simulation procedure is described, with special emphasis on a "unified approach" for the digital generation of artificial earthquake motions. Next, the advantages and disadvantages of the method over the power spectral method are discussed; and, finally, an efficient "hybrid Monte Carlo power spectral method" is developed. The Monte Carlo simulation procedure consists

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of the following tasks: (1) digital generation of artificial earthquake motions, (2) response analysis of the structure to a number of sample motions, and (3) statistical analysis of the structural responses.

The Monte Carlo method, though versatile, is rather costly, and hence it is employed mostly for nonlinear response analyses, for which the power spectral method is not applicable. Because of the cost, even for large, nonlinear systems, it is seldom used. The method proposed in this paper is cheaper than the Monte Carlo method, and is applicable to a wider class of problems than the power spectral method. It is particularly suited for the nonlinear, stochastic seismic analysis of nuclear plant structures, when material nonlinearities, fluid-structure interaction nonlinearities, or nonlinearities due to "gaps" are included in the analysis.

● **6.13-10** Singh, M. P. and Wen, Y.-K., **Nonstationary seismic response of light equipment,** *Journal of the Engineering Mechanics Division, ASCE,* **103,** *EM6,* Proc. Paper 13419, Dec. 1977, 1035-1048.

A method is developed to obtain floor spectra curves that include the effect of nonstationarity of seismic motions. Such spectra are usually required for design or qualification of light secondary systems such as piping and equipment. The response of a simple structure-equipment system is examined for excitations modeled by Gaussian shot-noise and filtered shot-noise. The Markov vector approach is used to obtain the time-dependent second moments of the response in closed form. The nonstationarity due to time-varying intensity of the excitation and also due to the zero-starting condition is considered. The effects of various seismic intensity modulation functions and structural parameters on the response are evaluated. Useful results of practical importance are obtained in terms of response ratios of nonstationary-to-stationary responses, and their application in the generation of floor spectra curves for a structure is illustrated.

● **6.13-11** Rosenblueth, E. and Contreras, H., **Approximate design for multicomponent earthquakes,** *Journal of the Engineering Mechanics Division, ASCE,* **103,** *EM5,* Proc. Paper 13289, Oct. 1977, 881-893.

Current design criteria are intended to provide values of structural responses to a single component of a design earthquake at a fixed exceedance probability. It is desirable to know whether a structure is safe, at that reliability level, under the combined effect of several simultaneous earthquake components. If the component accelerograms are idealized as zero-mean Gaussian processes and the structure behaves linearly, then structural responses associated with a given exceedance probability define an ellipsoid in the response space. In this space, points falling inside the failure surface (interaction surface) correspond to survival; those outside, to failure. A structure is safe at the specified reliability level, if the corresponding ellipsoid falls entirely within the survival region. Computations to verify this condition are, however, awkward. An approximate method is developed which replaces computation of the ellipsoid coordinates with that of a linear combination of responses to individual components.

● **6.13-12** Iyengar, N. R. and Dash, P. K., **Highest peak distribution in time varying systems,** *Journal of the Engineering Mechanics Division, ASCE,* **103,** *EM5,* Proc. Paper 13285, Oct. 1977, 869-879.

Upper and lower bounds have been developed for the probability distribution function of the highest absolute response of a linear time varying system under random inputs in a given interval of time. The system itself can be either deterministic or stochastic. Explicit expressions have been derived for a single degree-of-freedom system with typical deterministic and stochastic process coefficient terms and excited by white noise. Numerical results are presented and are compared with digitally simulated highest peak statistics. For small perturbations around the deterministic time invariant system, the method leads to very sharp bounds.

● **6.13-13** Valadares Tavares, L., **Extremes of autocorrelated load model,** *Journal of the Engineering Mechanics Division, ASCE,* **103,** *EM4,* Proc. Paper 13162, Aug. 1977, 717-723.

Usually, a sequence of independent and equally distributed random variables is the statistical model adopted to study the design extremes of geophysical loads. However, the autocorrelation function of most loads has significant nonzero values and then a stochastic model should be used. Unfortunately, major conclusions of the stochastic theory of extremes are obtained assuming that the studied process is Gaussian, and that several geophysical loads are non-Gaussian processes. Furthermore, most of these results are only acceptable under asymptotic conditions and their validity limits are not well known. In this paper, a non-Gaussian stationary stochastic process is developed to model geophysical loads in order to obtain the exact distribution of their extremes. A sensitivity analysis of the expected return periods and of their variances with the coefficients of the autocorrelation of the loads is also presented. As an example, this model is successfully applied to deseasonalized hourly mean wind velocities recorded in Lisbon.

● **6.13-14** Takemiya, H. and Lutes, L. D., **Stationary random vibration of hysteretic systems,** *Journal of the Engineering Mechanics Division, ASCE,* **103,** *EM4,* Proc. Paper 13133, Aug. 1977, 673-687.

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A more rigorous derivation of the modified power balance method is given for general yielding systems. It is demonstrated that the physical meaning of the equivalent linearization criteria derived by the mean-square minimization (Krylov-Bogoliubov) method are the equivalency of the dissipative and potential energies of the linear and nonlinear systems. Thus, linearization by power balance can be the same as by mean-square minimization. Simple gradient-stiffness approximations for the amplitude-dependent average frequency of hysteresis cycles and the overall average frequency of random response are presented for systems of Masing's type. In addition to the previously studied bilinear hysteretic system, the method is applied to compute rms response levels of trilinear hysteretic and Ramberg-Osgood type systems.

● **6.13-15** Corotis, R. B. and Marshall, T. A., **Oscillator response to modulated random excitation,** *Journal of the Engineering Mechanics Division, ASCE,* **103,** *EM4,* Proc. Paper 13107, Aug. 1977, 501–513.

A closed-form expression for the response power spectral density function is derived for the case of exponentially modulated stationary excitation as an extension of the heaviside case. A simply analytical expression is obtained for the mean-square response due to broad-band input when the excitation modulation can be expressed as the sum of exponential terms. Examples are presented to graphically compare the response for oscillator parameters typical of structures and two modulations approximating earthquake excitation. Numerical calculation of the first-passage probabilities based on the two-state Markov approach permits a comparison of the stationary, heaviside, and exponential excitation envelopes. For the examples presented, stationary or quasistationary approximations do not represent the reliability accurately.

● **6.13-16** Gersch, W. and Yonemoto, J., **Synthesis of multivariate random vibration systems: a two-stage least squares AR-MA model approach,** *Journal of Sound and Vibration,* **52,** *4,* June 22, 1977, 553–565.

A parametric time series model procedure for the synthesis of multivariate stationary time series random vibrations is shown. The vibrations are assumed to be the outputs of a regularly sampled, random noise excited, differential equation model of a vibration system. The procedure is a two-stage least squares method for realizing a multivariate discrete time mixed autoregressive-moving average (AR-MA) model from a given stationary process matrix covariance function. The synthesis procedure and the problem of the minimal representation of multivariate output systems and the overparameterization of AR-MA models are discussed and illustrated.

● **6.13-17** Elishakoff, I., **On the role of cross-correlations in the random vibrations of shells,** *Journal of Sound and Vibration,* **50,** *2,* Jan. 22, 1977, 239–252.

Theoretical and numerical results are presented for the investigation of the role of cross correlations in random shell vibration problems. The conditions of negligibility of the cross terms are derived for a shell with hysteretic damping. It is shown that, for a spherically curved panel, omission of cross correlations leads to an incorrect estimate of the mean square of displacements. This is due to closeness of the eigenfrequencies at the low end of the frequency domain and the consequent high modal density.

● **6.13-18** Roberts, J. B., **Stationary response of oscillators with non-linear damping to random excitation,** *Journal of Sound and Vibration,* **50,** *1,* Jan. 8, 1977, 145–156.

For oscillators with nonlinear damping, excited by white noise, an approximation to the stationary joint density function of the displacement and velocity response is derived. This involves reducing the basic two-dimensional Fokker-Planck equation for the transition density function to a one-dimensional equation relating to the energy envelope of the response. Results obtained from this approximation are compared with other theoretical predictions, and also with digital simulation results in typical cases.

● **6.13-19** Sidwell, K., **Analysis of the response of linear dynamic systems to product random processes,** *Journal of Sound and Vibration,* **55,** *1,* Nov. 8, 1977, 55–64.

A method is presented for the analysis of the response of linear dynamic systems to product random processes, with application to processes with random amplitude modulation. The Fokker-Planck equation is used to develop equations for the statistical moments of the system response, including relations for the moments of all orders of the response of systems which have linear time-invariant state equations. The quasisteady approximation, which omits the dynamic properties of the random amplitude modulation, is developed as a limiting case of the exact solution for the moments of the system response. The quasisteady approximation is generally accurate for well-damped systems, but can be inaccurate for lightly damped systems.

● **6.13-20** Baratta, A., **Plastic adaptation of structures under stochastic excitation,** *Journal of Structural Mechanics,* **5,** *4,* 1977, 421–450.

Dynamic adaptation of plastic structures is considered, assuming that the loading process is described by a random function of time. Since classical shakedown theory fails under stochastic loading, the need to follow the evolution of plastic deformations and displacements is recognized. To this end, bounding theorems for plastic displacements are

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extended to treat random loading. The theory is explained with reference to a simple structural pattern, i.e., an elastic-plastic work-hardening shear frame. A practical example concerning damage to a structure in a seismic site is numerically treated, and the results are evaluated against aseismic standard codes.

● **6.13-21** Badrakhan, F., **Approximate study of a hysteretic system** (Étude approchee d'un systeme a hysteresis, in French), *International Journal of Non-Linear Mechanics*, **12**, *1*, 1977, 1-7.

On the basis of a geometrical interpretation of the method of the first harmonic of Haag, Krylov, Bogolioubov, and Mitropolski, a new method for the dynamic study of a hysteretic oscillator is presented. This consists of replacing the equation of motion by another approximate equation which conserves the nonlinearity of the original equation. This method is valid in the case of sinusoidal excitation, but it can be extended to the case of random excitation. When this excitation is white noise, the Fokker-Planck equation for the system can be written with the help of certain assumptions which are physically realizable. Hence, all useful information concerning the response can be derived.

6.13-22 Ahmadi, G., **On the stability of systems of coupled partial differential equations with random excitation**, *Journal of Sound and Vibration*, **52**, *1*, May 8, 1977, 27-35.

The stability of the null solution of a class of systems of coupled partial differential equations with stationary random coefficients is studied. Criteria for the asymptotic stability of the equilibrium state are established. Some examples of applications of the results to engineering problems are presented. A variational scheme for improvement of the stability criteria is also discussed.

● **6.13-23** Pereira, J., Oliveira, C. S. and Duarte, R. T., **Direct and indirect conversion from power spectra to response spectra**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. I, 1977, 624-630.

The earthquake ground motion is considered as a time parameter Gaussian stationary stochastic process defined by a power spectral density function. Based on the properties of the stochastic processes and on the behavior of linear systems, the response spectra of a one degree-of-freedom system for the given power spectral density function is obtained. An iterative technique allows the conversion of a response spectrum into the corresponding power spectral density.

● **6.13-24** Balasubramonian, S. and Iyer, K. S. S., **Stochastic response analysis of structures to earthquake forces**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. II, 1977, 1089-1093.

The stochastic response analysis of a single degree-of-freedom system is discussed, using a mathematical nonstationary model to describe the ground acceleration. The approach makes use of only the statistical properties of the model and the system characteristics. In addition to evaluating the variance of the response process, the paper also analyzes, using approximate methods, the statistical distribution of the peaks of response and the upper bound probability of failure within time-dependent barriers.

● **6.13-25** Rasskazovsky, V. T., **Response spectra for calculating seismic loads**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. II, 1977, 1350-1355.

The response spectrum technique for aseismic design of structures is widely used but a theoretical approach to the definition of the real shape of the response curve meets with difficulties. This is because spectral densities are random functions which cannot represent the entire seismic process. In this paper, a probabilistic approach is presented for the derivation of the upper estimate of the response spectra for design purposes. If additional information about the predominant earthquake frequency in a given site is available, regional response curves can be plotted, leading to more precise and economic design. Some examples of such curves are given.

● **6.13-26** Duarte, R. T., **A probabilistic approach to the study of linear response of structures under multiple support non-stationary ground-shaking**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. II, 1977, 1376-1381.

Structural behavior is modeled by a multidegree-of-freedom linear dynamic system. An equation of motion representing earthquake loading as a multiple support input, and/or propagating ground motion, is presented. Earthquake vibration is idealized as a nonstationary Gaussian stochastic process. Magnitude, source mechanism, distance to fault, and local soil conditions are considered. Under those assumptions, the structural response is also a nonstationary Gaussian stochastic process, for which the most important probabilistic properties are derived.

● **6.13-27** Ermutlu, H. E., **Probabilistic dynamic earthquake analysis of nuclear power plants**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. III, 1977, 2578-2586.

To satisfy the requirements of probabilistic design, a modal frequency response analysis method incorporating stationary Gaussian random vibration theory is developed

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for the dynamic analysis of elastic structures subjected to three-dimensional earthquake excitation. A direct random analysis approach to the floor response spectrum problem is achieved such that the need for analysis in the time domain is completely avoided. Because of the practical difficulties met in determining the site-dependent acceleration power spectrum of ground motion and to increase the applicability of the method, the possibility of a direct conversion between the acceleration power spectrum and the velocity response spectrum is established. The feasibility of using a generated average power spectrum is demonstrated using a sample structure.

● **6.13-28** Kobori, T. and Minai, R., **Application of stochastic differential equations to seismic analyses of nonlinear structures,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1065-1070.

This paper deals with the methods of solving the Fokker-Planck-Kolmogorov equations with an appropriate initial or end condition and a mixed type boundary condition on a bounded domain, paying particular attention to the seismic reliability analyses of nonlinear structures. First, as the mathematical bases of this problem, a generalized Green's formula for nonself-adjoint partial differential systems and the associated stationary functional theorems are presented. Then, defining a reliability function in terms of Green's function of the Fokker-Planck-Kolmogorov partial differential system with a bounded boundary surface, some kinds of solution techniques based on the boundary value problem as well as the variational method are discussed.

● **6.13-29** Matsushima, Y., **Stochastic response of structure due to spatially variant earthquake excitations,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1077-1082.

The cross-spectral density function between spatially variant earthquake excitations is assumed such that the statistical correlation decreases in an exponential manner as the distance increases. A system in which a rigid slab is supported by many columns, the bottom ends of which are subjected to multiple inputs which have the above-mentioned characteristic, is supposed. Frequency response functions of the absolute acceleration and of the relative displacement are mathematically derived. Stochastic responses are analytically evaluated when excitations are band-limited white noises with such spatial variation. Responses resulting from simulated noises of typical earthquakes are numerically estimated.

● **6.13-30** Hadjian, A. H. *et al.,* **Variability in engineering aspects of structural modeling,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2729-2734.

The state-of-the-art in modeling of structures has not kept pace with the advances made in computational techniques. The modeling of structures is still an exacting art. The engineering judgment that goes into the modeling process is of extreme importance as regards to the outcome of the calculations. This human aspect of structural modeling is explored by two separate experiments involving an idealized structure and a seven-story steel frame structure. The results of these experiments are statistically evaluated and a probabilistic model is suggested to deal with the effect that engineering judgment has on frequency calculations.

● **6.13-31** Sato, H. *et al.,* **On a simple method estimating the appended system response spectrum from a statistically simulated spectrum,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3280-3285.

Response spectra of single degree-of-freedom systems subjected to earthquake motions show the behavior with peaks and troughs as the natural period varies. In a previous work, it was shown that this behavior could be simulated by assuming two ground predominant periods for an artificial earthquake. However, this required complicated computations. A new simple method of the estimate based on a basic spectrum with a single ground predominant period and on the method of the square root of sum of squares is proposed.

● **6.13-32** Ozaki, M. and Ishiyama, Y., **An evaluation method for the earthquake resistant capacity of reinforced concrete and steel reinforced concrete columns,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3107-3112.

An evaluation method is proposed for the earthquake-resistant capacity of reinforced concrete and steel reinforced concrete columns by utilizing the force deflection relationship of column specimens subjected to axial force and repeated and reversed lateral loading of considerable intensity. An approximate response analysis for a nonlinear structural system was developed based on random vibration theory and was applied to models represented by a single degree-of-freedom system subjected to a constant white noise acceleration. It is confirmed that yield point level, hysteretic envelope slope, hysteretic damping ratio, and ductility factor are the most important components of earthquake-resistant capacity.

● **6.13-33** Vanmarcke, E. H., **A simple procedure for predicting amplified response spectra and equipment response,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3323-3327.

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This paper presents a new procedure to evaluate secondary system response directly from the specified ground response spectra. It is assumed that a modal analysis of the primary structure has been made. The maximum acceleration of a one-degree secondary system is expressed as the square root of a sum (over all significant primary system modes) of contributions which depend on (1) S_{Ak} = the pseudo-acceleration (ground) response spectrum for the period and damping of primary system mode k, and (2) S_{Ae} = the pseudo-acceleration (ground) response spectrum for the equipment period and damping. The amplification factors multiplying S_{Ak} and S_{Ae} are easy to evaluate and have been derived from a nonstationary random vibration study of secondary system response.

● **6.13-34** Duarte, R. T., **An engineering assessment of the influence of source mechanism on firm ground shaking,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 895-900.

Nonstationary earthquake ground motion is generated by the superposition of a stationary vibration originating from the foci of lower magnitude earthquakes, closely spaced along a fault. This vibration is modeled by a stationary Gaussian stochastic process restricted to a time interval and the power spectral density of acceleration which takes into account magnitude, focal distance, and geologic conditions at the site of interest. The intensity of the resulting ground motion is assessed through the maximum acceleration, maximum velocity, maximum displacement, duration, and response spectra.

● **6.13-35** Friedman, M. and Yavin, Y., **Computation of functionals over discontinuous sample paths of a stochastic van der Pol oscillator,** *International Journal of Non-Linear Mechanics,* 12, 5, 1977, 307-314.

This paper deals with a random van der Pol oscillator. It is assumed that the oscillator is subjected to two types of perturbation. The first type is represented by the standard Wiener process and the second type by a homogeneous process with independent increments, finite second-order moments, mean zero, and no continuous sample functions. In order to measure quantitatively the stochastic stability of the oscillator, two functionals are defined over its phase plane sample paths. It is shown that each of these functionals is a solution to a corresponding partial integro-differential equation. A numerical procedure for the solution of these equations is suggested, and its efficiency and applicability are demonstrated with examples.

● **6.13-36** Ahmadi, G., **On the stability of nonstationary stochastic differential equations,** *Iranian Journal of Science and Technology,* 6, 2, 1977, 75-80.

The stability of linear differential equations with nonstationary random coefficients is studied via the Liapunov method. A new Liapunov function is employed and the criteria for mean square stability are established. Several examples are considered. It is shown that for a special class of nonstationary random coefficients the null solution is stable if it is a stable solution of the force-free system.

● **6.13-37** Ifrim, M. and Zorapapel, T., **A new semiprobabilistic concept concerning the codification of the seismic actions,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1753-1759.

A more realistic estimate of the ratio of maximum ground acceleration during an earthquake to the RMS of the realization is proposed. First passage techniques are used based on the assumption that the model for ground acceleration is taken as an ergodic, normally distributed stationary process, and the half number of crossings, at a reasonably high threshold in the duration of the earthquake, is distributed using the Poisson method. The normalized maximum acceleration is a function of the ratio of the predominant period to the earthquake duration. On this basis, a simple formula is proposed to replace the constant value used in the international standard CEB-CECM-CIB-FIP-IABSE.

7. Earthquake-Resistant Design and Construction and Hazard Reduction

7.1 General

● **7.1-1** Ichihara, K., Kuribayashi, E. and Tazaki, T., **Retrofitting of vulnerability in earthquake disaster mitigation problems,** *Wind and Seismic Effects,* VI-69–VI-84. (For a full bibliographic citation, see Abstract No. 1.2-4.)

This paper discusses a criterion for retrofitting existing structures that are vulnerable to disastrous earthquakes. It would be ideal if all structural damage could be avoided during earthquakes, however, to completely strengthen structures is not practical because of the limitation of resources. It is necessary to classify the structures by functional importance and structural vulnerability. The first category of classification deals with the structural load, and the second category deals with the type of structure.

● **7.1-2** Lee, L. T. and Collins, J. D., **Engineering risk management for structures,** *Journal of the Structural Division, ASCE,* **103,** *ST9,* Proc. Paper 13182, Sept. 1977, 1739–1756.

A systematic methodology has been developed to determine risk to structures from four hazards: fire, flood, earthquake, and wind. This methodology combines seven major elements: the hazard model to provide a probabilistic description of each hazard, the exposure model to describe the assets exposed, the vulnerability model to describe the condition and strength of each structure, the damage algorithm to provide a relationship between hazard intensity and degree of damage expected, a set of alternative mitigations, cost data for the mitigations, and a risk equation. The risk equation produces an estimate of the average annual loss or the risk of mission loss. A number of possible mitigations are considered for each structure and each hazard. The average annual loss (i.e., risk) is computed for each hazard for the structure as is, and with each mitigation applied separately. This information is used to select an optimum investment program that minimizes the risk from the hazards. The hazard models and damage algorithms are based on information available in technical literature. The exposure and vulnerability data are project dependent.

● **7.1-3** Gasparini, D. A., **On the safety provided by alternate seismic design methods,** *Evaluation of Seismic Safety of Buildings, No. 9, R77-22,* Dept. of Civil Engineering, Massachusetts Inst. of Technology, Cambridge, July 1977, 215.

A method is developed for obtaining distributions of responses of elastic multidegree-of-freedom systems subjected to earthquake excitation. Uncertainty in both dynamic model and earthquake excitation parameters is accounted for. Random vibration theory and an approximate first-passage problem solution are utilized. Distributions of responses are computed for two 4-DOF dynamic models. Sensitivity of such distributions to earthquake and dynamic model parameters is quantified. Factors contributing to uncertainty in the strength measures used for rigid frame buildings are examined. A simple frame is designed by elastic criteria and a second-moment description of the story strength measures is given. Probabilities of exceeding limit-elastic response levels are computed for three models by utilizing the derived load effect and strength distributions. Probabilities conditional on peak ground accelerations are first obtained; then seismic risk information is incorporated to arrive at unconditional failure probabilities. Sensitivity of such safety estimates to seismic risk

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information and strength distribution parameters is examined. Comparisons with the safety assessments of other investigators are also made. Alternate elastic seismic design strategies are reviewed and their interrelationships are clarified. The relative conservatism of resultant designs is compared by using the methodology developed in the report.

● 7.1-4 Vinck, W. and Maurer, H. A., **Measures taken in the member countries of the European Communities, for anti-seismic design compared to present U.S. practice,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(a), Paper K 1/2, 9. (For a full bibliographic citation, see Abstract No. 1.2-5.)

Most countries of the European Communities base their aseismic design parameters on specific U.S. earthquake characteristics. There are, however, important discrepancies in the basic data reported on the two continents as well as in their design application. This was one of the topics under discussion within a European working group on methodologies, criteria, and standards in nuclear safety. The contribution is based on an inventory of the applied national practices, the existing specifications, regulations, and guidelines applied in the design, the manufacture, and the safety assessment of structures, systems, and components to withstand potential earthquake consequences in the countries of the European Communities. In a comparison of these national specifications and guidelines, the common points of agreement are identified and the divergencies discussed with reference to the U.S. practice. Special attention is given to the divergencies for definition and determination of the reference earthquakes. In European countries, the definitions of the reference earthquakes are largely analogous to definitions in the U.S. federal regulations but expressed in a different way. Unlike U.S. practice, in some European countries, the maximum earthquake that can be envisaged (corresponding to the safe shutdown earthquake in U.S. practice) is defined by adding a margin of safety to the maximum probable earthquake (corresponding to the operating basis earthquake in the U.S. for which statistical data exist). Differences exist also in the design parameters to be taken into account in the different European countries, especially in the evaluation of the maximum acceleration and in the relationship of the acceleration versus earthquake intensity. For design purposes, in the U.S. as well as in European countries, the assumption is made that seismic waves basically approximate a sustained simple harmonic motion. Under this assumption, the Neumann correlation which gives the relationship between the modified Mercalli intensity, the wave period, and the ground acceleration is applied. While in the U.S. a whole spectrum of wave periods (from 0.33 to 6.0 sec) in function of the type of foundation (soil, bedrock) and the distance of the epicenter are considered, the European countries base their investigations on shorter wave periods (approximately 0.3 sec). Mention is made of the existing differences in the relationship of horizontal to vertical acceleration levels. These differences in the evaluation of the earthquake characteristics influence the design to protect power plants against seismic effects, especially as far as stress and strain limits for structures and components within the elastic range and in the excess of yield are concerned.

The report discusses the instrumentation installed in newer nuclear power plants to record horizontal and vertical acceleration of earthquakes so that the seismic response of nuclear power plant features important to safety can be determined promptly to permit comparison of such response with that used as the design basis. For European countries, threshold values are proposed to guarantee safety for nuclear power plants.

● 7.1-5 Mroz, Z., **Mode approach to rational synthesis of structures under impulsive and dynamic pressure loading,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. L, Paper L 2/4, 8. (For a full bibliographic citation, see Abstract No. 1.2-5.)

An important problem in structural dynamics is concerned with the analysis of structural behavior under dynamic pressure or impulsive loading in the inelastic range. To simplify this analysis, models of rigid-plastic, nonlinear elastic, or nonlinear viscous materials have been used. For such simplified models, the permanent mode motions exist, and correspond to solutions of nonlinear eigenvalue problems. It has been shown that modal approximations provide a reasonable prediction of finite deflections, even in cases when initial transient behavior precedes the mode motion.

The aim of this work is to study the optimal synthesis of beams and plates which are subjected to initial impulses of given kinetic energy or dynamic pressure loading. Such form of a structure is to be determined for which mean or local final deflections are minimized for a given material volume and prescribed kinetic energy of the initial impulse. Starting from modal approximation, it was shown that maximization of the eigenvalue of modal solution leads to minimization of the mean deflection, and the problem of maximizing this eigenvalue can be used as a major criterion in generating optimal design solutions. On the other hand, for minimization of a local final deflection, the numerical search technique must be applied. In particular, it was shown that for perfectly plastic structures, the static criteria of optimal design, based on a concept of a simultaneous failure mode, are not directly applicable in the case of dynamic loading and the significance of one degree-of-freedom modes was demonstrated. In the case of dynamic pressure loading, the piecewise modal solutions were applied in the range of moderate pressures in order to determine optimal designs. A direct variational approach and optimal control theory are useful tools in deriving optimality conditions and relative numerical methods.

Examples of beams and circular plates are considered in detail in order to illustrate general results reached in the first part of the work. Designs with continuously varying and piecewise constant cross sections were considered and their effectiveness with respect to uniform designs was studied. Nonunique modal solutions were found to exist in rigid-plastic structures, and coexistence of several modes was theoretically observed for some ranges of design parameters. The present work constitutes the first step towards rational synthesis of inelastic structures, and the presented approach may prove too simplified since no constraint was imposed on the acceleration of motion, which is of essential importance in damping of impact in vehicle structures. However, there is a large class of problems (for instance, in nuclear technology), where plastic damping capacity of a structure should be utilized without necessary constraints on accelerations. For such cases, the present formulation may be useful in determining rational designs of flexural structural elements.

● **7.1-6** McConnell, R. D., **Least weight structures for threshold frequencies in a seismic environment,** *Computers & Structures,* 7, *1,* Feb. 1977, 157–160.

Conventional component seismic design procedures fall into two categories: a table of component forces (which often result in grossly conservative or grossly underdesigned structures); or sophisticated computer analyses using detailed models which undergo dynamic time-history force simulations. Aerospace engineers have designed many component structures to a minimal-fundamental, or "threshold," frequency specification. Although it does not appear as generally applicable for ground-supported "base structures," the design of substructures for earthquake-induced forces can similarly adopt such specifications. For example, radio towers, water tanks, electrical gear, etc., which are supported by larger structures will be subjected to the filtered natural frequencies of the base structure regardless of the earthquake spectrum. By knowing approximate amplification factors for the principal contributing frequencies (and corresponding base-excitation levels) of the base structure, a substructure specification for the (1) dynamic amplification factor, (2) arbitrary safety-factor, and (3) threshold design frequency can be determined. This paper describes adjustment of a structure's fundamental frequency to a prescribed minimum level by a modified Rayleigh method which simultaneously converges to the fundamental mode while being proportioned accordingly to a fully stressed optimum design.

● **7.1-7** Skinner, R. I., Heine, A. J. and Tyler, R. G., **Hysteretic dampers to provide structures with increased earthquake resistance,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1319–1324.

Hysteretic dampers are being used in combination with special provisions which increase sidesway to reduce earthquake forces and deformations throughout structures and to reduce or eliminate ductility requirements. Present or possible future applications to buildings, bridges, nuclear power plants, and other structures are described, together with new developments in damper technology.

● **7.1-8** Bertero, V. V., Mahin, S. A. and Herrera, R. A., **Problems in prescribing reliable design earthquakes,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1741–1746.

Earthquake ground motion characteristics, which control the elastic and inelastic response of structures, are shown to be fundamentally different. Limitations of present methods for specifying design earthquakes for buildings located near potential sources of a major earthquake are examined. In particular, the reliability of inelastic design response spectra derived directly from linear-elastic design response spectra is evaluated on the basis of the nonlinear dynamic response of single and multiple degree-of-freedom systems to near-fault accelerograms obtained during the San Fernando earthquake. Guidelines for prescribing inelastic design response spectra for near-fault sites and recommendations for further research are presented.

● **7.1-9** Borges, J. F., **Principles of earthquake resistant design,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1735–1740.

A set of principles to be used as a checklist in code writing and structural design is presented. Since earthquake-resistant design should be viewed within the general framework of planning and design, most of the principles are of a general character and express modern tendencies in structural design.

● **7.1-10** Krishna, J. and Chandrasekaran, A. R., **Problems in earthquake resistant design,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1729–1734.

In earthquake-resistant design, judgment has to be exercised by the designer at various stages. This paper discusses the assumptions involved in the evaluation of site seismicity, determination of ground motion parameters, soil-structure interaction effects, and modeling of a structure. It is concluded that, at present, earthquake-resistant design is an art even though progress has been made in recent years to give it a scientific basis. The paper highlights the unanswered questions.

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● 7.1-11 Meltsner, A. J., **Seismic safety of existing buildings and incentives for hazard mitigation in San Francisco: an exploratory study,** *UCB/EERC-77/28,* Earthquake Engineering Research Center, Univ. of California, Berkeley, Dec. 1977, 83. (NTIS Accession No. PB 281 970)

The seismic safety of existing buildings is not solely a matter of engineering design and assessment of seismological risk. Within any community there are likely to be different perceptions of the public interest. It seems clear that what is needed, in addition to engineering studies to develop appropriate procedures for evaluating and coping with the hazard from existing buildings, is knowledge about the range of appropriate incentives which might prove useful for governments to use to facilitate implementation of these procedures. The main purpose of this report is to document an exploratory study of a number of possible incentives and options which have been discussed in the past by public officials and conceivably might be adopted in the future.

● 7.1-12 Dowrick, D. J., **Structural form for earthquake resistance,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1826-1834.

For earthquake resistance, the form of a structure is of fundamental importance. This paper provides a set of guidelines which should be observed by designers in planning the form of a structure.

● 7.1-13 Housner, G. W. and Jennings, P. C., **Earthquake design criteria for structures,** *EERL 77-06,* Earthquake Engineering Research Lab., California Inst. of Technology, Pasadena, Nov. 1977, 52.

● 7.1-14 Nakano, K., Aoki, Y. and Watanabe, K., **A proposal for a rational aseismic structural design process,** *BRI Research Paper No. 68,* Building Research Inst., Tokyo, Sept. 1976, 45.

This paper presents a proposal for a new aseismic structural design process based on a rational, synthesized design concept. Structural planning and evaluation are defined to establish a more comprehensible design process. Traditional design processes are analyzed, and it is shown that actual design processes are governed and controlled by three principles. The most important principle, that of public safety, is thoroughly examined, and an example of the design process is shown.

● 7.1-15 Drenick, R. F., **Minimax procedures for specifying earthquake motion,** Polytechnic Inst. of New York, May 31, 1976, 88.

7.2 Building Codes

● 7.2-1 Celasun, H., **Interpretation of the 13th section of new earthquake code** (Yeni deprem yonetmeligi 13, bolumunun aciklanmasi, in Turkish), *Deprem Arastirma Enstitusu bulteni,* 3, *12,* Jan. 1976, 1-15.

In the design of structures to withstand earthquakes, it is difficult to predict the character and intensity of the ground motion. The number of actual measurements of strong earthquakes is low. Therefore, the design must be based only on the estimated ground motion. A realistic analysis of multidegree systems should also take into account the plastic behavior of the structures. For this reason, the problem is sophisticated and tedious and a rigorous solution is quite impossible.

An introduction to the subject and basic concepts are summarized. The relation between the theory and the new design code formula is presented. The code recommendations are based on the dynamic distribution of lateral forces, but it is also admitted that for design purposes a building may be analyzed as though these forces were applied statically. The main formula of the code is based on the assumption that the characteristic shape of the fundamental mode is a straight line from the foundation to the top of the building. This approximation is reasonable for typical buildings. The code considers only one mode response and damping is ignored.

● 7.2-2 Elms, D. G., **Seismic torsional effects on buildings,** *Bulletin of the New Zealand National Society for Earthquake Engineering,* 9, *1,* Mar. 1976, 79-83.

This paper gives the background to the seismic torsional provisions of the New Zealand Loadings Code. These provisions attempt to deal with three effects: accidental eccentricity, torsional ground motion, and coupling between torsional and translational modes. The first two effects lead to an effective eccentricity of one-tenth of the width of a building, while the third leads to a parabolic function of the calculated eccentricity, which was obtained by applying existing results for torsional amplification to a number of typical structures.

● 7.2-3 Ersoy, U. and Gulkan, P., **Interpretation of the section of the code for reinforced concrete structures built in earthquake zones** (Afet bolgelerinde yapilacak yapilar hakkindaki yonetmelikte betonarme yapilarla ilgili bolumun temel ilkeleri ve aciklamalar, in Turkish), *Deprem Arastirma Enstitusu bulteni,* 3, *10,* July 1975, 5-44.

Taking into account the behavior of reinforced concrete structural elements subjected to an earthquake loading, the authors interpret a section of the earthquake code and suggest some changes in it.

● 7.2-4 Glogau, O. A., **Code philosophy,** *Bulletin of the New Zealand National Society for Earthquake Engineering,* 9, *1,* Mar. 1976, 43–55.

This lecture presents philosophical background information on the decisions leading to the final proposal for the seismic provisions for New Zealand NZS 4203 *Code of Practice for General Structural Design and Design Loadings.* Ductility factors and seismic design coefficients for various types of structures are discussed to show how the new code covers the necessary design requirements. The design principles lying behind the code requirements are discussed, particularly for concrete frame and shear wall type structures. The interaction between the motion of the subsoil and motion of the structure is also discussed. A number of unresolved problems are highlighted, particularly those concerned with short-period structures, long acceleration seismic pulses, the damping-amplification relationship, and mathematical modelling. Japanese and California codes are compared by showing the latest developments and illustrating the wide range of values used for the basic requirement. The paper concludes with consideration of likely future development and the manner in which the New Zealand code is likely to be updated in the future.

● 7.2-5 Skinner, R. I., **Dynamic aspects of the code for the design of earthquake resistant buildings,** *Bulletin of the New Zealand National Society for Earthquake Engineering,* 9, *1,* Mar. 1976, 63–67.

A brief review is given of the most important aspects of the dynamic building behavior during elastic and inelastic deformations under earthquake loads. These aspects must be considered for the effective utilization of the NZS 4203 *Code of Practice for General Structural Design and Design Loadings.*

● 7.2-6 Taylor, P. W., **Code provisions related to soils and foundations,** *Bulletin of the New Zealand National Society for Earthquake Engineering,* 9, *1,* Mar. 1976, 68–73.

In the new *Code of Practice for General Structural Design and Design Loadings* (NZS 4203), the basic seismic coefficient, in addition to being related to the period of the structure, is related to the deformability of the site. The requirement for foundation interconnection has been carried forward from the earlier code with little change. A new section on foundation design has been introduced in which the principles of capacity design are applied, where appropriate, to the foundation substructure.

● 7.2-7 Armstrong, I. C., **Design of shear walls for seismic resistance,** *Bulletin of the New Zealand National Society for Earthquake Engineering,* **10,** *1,* Mar. 1977, 17–36.

Ductile coupled shear walls, ductile cantilever shear walls, and less ductile "category 6" shear walls comprise three basic shear wall types included in categories 3, 4, 5 and 6 of Table 5 of the earthquake provisions of the New Zealand Loadings Code. Seismic resistance of these shear wall types, when fully cracked with reinforcement at or near yield where applicable, is discussed relative to the New Zealand Loadings Code provisions. Reinforcement requirements are outlined, using capacity design criteria for ductile walls, and load-factor methods for category 6 shear walls requiring distributed vertical and horizontal reinforcement to control cracking.

● 7.2-8 Priestley, M. J. N., **Seismic resistance of reinforced concrete-masonry shear walls with high steel percentages,** *Bulletin of the New Zealand National Society for Earthquake Engineering,* **10,** *1,* Mar. 1977, 1–16.

Test results of six heavily reinforced concrete masonry shear walls are summarized. The test program was designed to investigate the necessity for the low ultimate shear stress specified by masonry codes. Care was taken to accurately model good but realistic design practice in detailing, and variables investigated in the series included steel percentage, influence of vertical load, and confinement of potential crushing areas by mortar bed confining plates. Results clearly indicate that the maximum current code allowance for ultimate shear stress is unreasonably low. No wall suffered diagonal shear failure despite maximum shear stresses exceeding four times the maximum code level. All walls displayed stable hysteresis loops at a displacement ductility factor of 2, and the less heavily reinforced walls (designed to approximately twice code levels) were satisfactory at DF = 4. Degradation was never catastrophic and occurred due to slip of the entire wall along the foundation beam. Methods for reducing the degradation are discussed. Confining plates did not significantly reduce the degradation of the hysteresis loops, but they substantially reduced damage to the walls at high ductility factors. Values of required ductility for walls designed to the New Zealand Loadings Code are investigated, and on the basis of these and the experimental results, recommendations are made for relaxation to the ultimate shear provisions of the masonry code.

● 7.2-9 Burns, R. J. *et al.*, **Papers resulting from deliberations of the Society's discussion group on seismic design of ductile moment resisting reinforced concrete frames,** *Bulletin of the New Zealand National Society for Earthquake Engineering,* **10,** *2,* June 1977, 69–105.

The seismic design of frames according to the New Zealand Loadings Code, NZS 4203:1976, is discussed. Here follow the paper titles and authors' names; none of these papers is abstracted in this volume of the *AJEE: Section A: Introduction and philosophy,* Burns, R. J.—*Section E: Beam flexure and hinge zone detailing in reinforced concrete*

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ductile frames requiring beam sway mechanisms, Smith, I. C. and Sidwell, G. K.—*Section F: Shear strength requirements,* Paulay, T.—*Section G: Columns–evaluation of actions,* Paulay, T.—*Section H: Columns subjected to flexure and axial load,* Park, R.—*Section K: Parts, portions and secondary elements,* Allardice, N. W.

● **7.2-10** Shah, H. C. and Sexsmith, R. G., **A probabilistic basis for the ACI code,** *Journal of the American Concrete Institute,* 74, *12,* Title No. 74-54, Dec. 1977, 610–611.

Development of probability-based second moment codes of several types is currently taking place. The paper describes the general philosophy of developing such codes. Various probability-based improvements are suggested for the current ACI code. Some of the problems associated with the introduction of probabilistic concepts in current codes are described.

● **7.2-11** ACI Committee 318, **Proposed revisions to: building code requirements for reinforced concrete (ACI 318-71),** *Journal of the American Concrete Institute,* **74,** *1,* Title No. 74-1, Jan. 1977, 1–21.

A number of revisions to the ACI Building Code are presented. The revisions deal with definitions of terms, materials, concrete quality, formwork, reinforcement details, analysis and design, strength and serviceability, flexure and axial loads, shear and torsion, development of reinforcement, prestressed concrete, and working stress design. Some reorganization of the code is proposed.

● **7.2-12** Newmark, N. M. *et al.,* **Seismic design and analysis provisions for the United States,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1986–1992.

The seismic design and analysis requirements presently used in U.S. building codes were largely developed in the *Recommended Lateral Force Requirements and Commentary* published by the Seismology Committee of the Structural Engineers Assn. of California in 1959-1960. Since that time, improvements and modifications have been made at frequent intervals. Several years ago, it became apparent that a comprehensive review should be made of the provisions, the latest state of knowledge should be evaluated, and a coordinated set of provisions should be developed. This paper describes the basic concepts and summarizes some of the details of the structural design provisions of a new code developed by the Applied Technology Council.

● **7.2-13** Goldberg, A. and Sharpe, R. L., **Provisions for seismic design of non-structural building components and systems,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3274–3279.

Requirements for the design of architectural, electrical, and mechanical systems and components in buildings are included in nationally applicable seismic design provisions now being developed for the United States. The design requirements are based on the use or occupancy of the building and the possible life hazard that may result in the event of failure. The requirements for architectural elements involve force resistance and provisions for interstory drift and out-of-plane deflections. The design of mechanical and electrical systems includes provisions for fixed and resiliently mounted equipment and the utility or service interface with the building. A detailed commentary is included.

● **7.2-14** Shibata, H., **The way of setting the aseismic design code of oil refineries and petro-chemical industries,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3347–3355.

The way in which aseismic design codes for oil refineries and petrochemical facilities are developed is considered. The aseismic design of both types of structures should be based on their modes of failure.

● **7.2-15** Glogau, O. A., **Some comments on the New Zealand earthquake loading provisions for buildings,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1780–1788.

New Zealand's *Code of Practice for General Structural Design and Design Loadings of Buildings* (NZS 4203) is discussed with attention given to the philosophy and to unusual features of the earthquake provisions. Seismic loads are derived from a function which includes terms reflecting the expected manner of dissipation of seismic energy and the performance of a structural system in general. The code recognizes that the design load level must be intimately linked with other structural designs and detailing criteria. Consequently, all ductile structures are required to conform to a capacity design procedure which includes the concept of concurrency. The code establishes design criteria for architectural and service components for avoiding damage in moderate earthquakes and minimizing direct and indirect risk to lives in severe events.

● **7.2-16** Krimgold, F., **Seismic design decisions for the Commonwealth of Massachusetts state building code,** *Seismic Design Decision Analysis Report No. 32, MIT-CE R77-27,* Dept. of Civil Engineering, Massachusetts Inst. of Technology, Cambridge, June 1977, 116.

This report reviews the background for the work of the American Society of Civil Engineers/Boston Society of Civil Engineers Joint Committee on Seismic Design Criteria and the considerations taken by the committee in

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formulating its recommendations. Attention is focused on the questions of estimation of seismic risk in Massachusetts, determination of a design earthquake, development of appropriate soil factors, and the determination of acceptable risk. Aside from documenting the decision process of the committee, this report also attempts to evaluate the role of the Seismic Design Decision Analysis project methodology and the loss estimates provided by the research project in the formulation of recommended seismic design criteria.

● **7.2-17** Borges, J. F. and Ravara, A., **The new Portuguese seismic code,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1789–1794.

This paper presents the main provisions in the chapter on seismic actions of the new Portuguese "Safety and Loading Code for Building and Bridge Structures." A draft of the document, issued in January 1976, was circulated for criticism and suggestions until the end of 1976, when the final version became official. The main problem in preparing the new code was that of including changing modern concepts of earthquake engineering while maintaining simplicity. The criteria adopted to overcome this difficulty are presented and briefly discussed.

● **7.2-18** Estrada-Uribe, G., **The proposal for the Colombian antiseismic building code,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1801–1806.

Given the record of seismic activity in Colombia, an aseismic building code is necessary to guarantee a minimum standard of public safety. In November 1972, the author presented before the National Engineering Council the first proposal for a Colombian Antiseismic Building Code. Although it is still under discussion and consideration, city authorities in Bogota and other metropolitan areas of the country have already approved tall buildings designed according to the code. The document has also served as a source for the corresponding proposals of other Latin American countries. A brief summary of its contents is presented.

● **7.2-19** Fuller, G. R. and Gerich, A., **Seismic design and construction for single-family dwellings,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1993–1998.

This paper presents the results of research conducted by the Applied Technology Council of San Francisco, under the sponsorship of the U.S. Department of Housing and Urban Development. The project developed a manual for seismic design and construction of single-family dwellings. The manual contains architectural layouts, design guides, types of construction, and features recommended or to be avoided. It also contains a cost impact study for dwellings located in seismic zones 2 and 3. The manual was prepared for engineers, builders, building officials, field inspectors, and house designers. It emphasizes the advantages of shear walls in single-family dwellings as prime elements to resist lateral forces.

● **7.2-20** Corderoy, H. J. B. and Warner, R. F., **Slender column design,** *UNICIV Report No. R-156,* School of Civil Engineering, Univ. of New South Wales, Kensington, Australia, June 1976, 24.

The slender column design procedures in the current Australian concrete code, AS 1480 - 1974, are found to be quite conservative when compared with those of the ACI and other national codes. Various factors are also found to be ignored or inadequately treated in AS 1480. It is suggested that the additional eccentricity approach to slender column design be seriously considered for inclusion in any future revision of AS 1480.

7.3 Design and Construction of Buildings

● **7.3-1** Bertero, V. V., **Identification of research needs for improving the aseismic design of building structures,** *Bulletin of the New Zealand National Society for Earthquake Engineering,* 9, *1,* Mar. 1976, 1–31.

The purpose of this paper is to present and discuss problem areas that exist in the earthquake-resistant design of buildings and to determine the most effective methods for obtaining the necessary information to resolve them. The identification of research needs is done by reviewing some of the general aspects involved in achieving an economical, serviceable and safe design for buildings located at sites with high seismic risk.

● **7.3-2** Englekirk, R. E., **Seismic design of reinforced concrete buildings from a designer's viewpoint,** *Reinforced Concrete Structures in Seismic Zones, SP-53,* American Concrete Inst., Detroit, 1977, 47–57.

Two approaches to seismic-resistant design, the building code approach as outlined in the Uniform Building Code and the spectral dynamic approach, are compared. The elements in each approach which define the amount of force to which a system will be subjected and the factors which define material capacities are given. Each method is then used to design and analyze two types of structures, an elastic system and an elastoplastic system. The results are examined and compared.

● **7.3-3** Fintel, M., **Ductile shear walls in earthquake resistant multistory buildings,** *Reinforced Concrete Structures in Seismic Zones, SP-53,* American Concrete Inst.,

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Detroit, 1977, 117–126. (Also in *Journal of the American Concrete Institute,* **71,** *6,* June 1974, 296–305.)

Slender shear walls in multistory buildings are discussed in this paper and answers are given to such questions as "Why do we need them?" and "What do we know about their design?". Also discussed are the historical development of the use of shear walls, their performance in earthquakes of the past 10 years (both good and bad), and finally, the available design information and our future needs in the area of design of shear walls for strength, stiffness, and ductility, as well as needs in the area of analysis for the dynamic response of shear wall structures.

● 7.3-4 Hart, G. C., **Active and passive instrumentation of hospitals,** *Proceedings of U.S.-Japan Seminar on Earthquake Engineering Research with Emphasis on Lifeline Systems,* Japan Society for the Promotion of Earthquake Engineering, Tokyo, Nov. 1976, 441–458.

The role and application of active and passive instrumentation systems in the improvement of hospital safety is discussed. A description of floor peak acceleration and response spectra as related to ground peak acceleration and response spectra is also presented.

● 7.3-5 Gallegos, H. and Rios, R., **Earthquake–Repair–Earthquake,** *Reinforced Concrete Structures in Seismic Zones, SP-53,* American Concrete Inst., Detroit, 1977, 463–477.

This paper analyzes the authors' experience in repairing reinforced concrete buildings damaged by earthquakes. Buildings damaged by a first earthquake were repaired and subsequently those repaired buildings underwent the effects of second and third earthquakes. In some cases, the new seismic actions caused further damage which also had to be repaired.

The initial repair of the buildings was carried out according to a planned "repair philosophy," with defined objectives, limitations, and methodology. That philosophy was tested by the subsequent earthquakes. The need for new repairs pinpointed the positive aspects and made it possible to modify and broaden the philosophy. The fact that many repaired buildings did not suffer any additional damage during subsequent earthquakes bore witness to the conceptual accuracy of the repair philosophy for the types of structures on which it was used.

● 7.3-6 Zelenskii, G. A. and Nazin, V. V., **Vibration damping in buildings with gravitational seismic dampers on kinematic foundations** (Dempfirovanie kolebanii zdanii s gravitatsionnoi seismoisolatsiei na kinematicheskikh fundamentakh, in Russian), *Seismostoikoe stroitelstvo, 2,* 1977, 27–33.

Under certain types of seismic conditions, seismic loads on a structure may be lowered by adjusting the geometrical dimensions of the supporting structures to lower vibration frequencies. This may cause extremely large displacements of the building relative to its foundation during the low-frequency phase of the seismic excitation. To solve this problem, an experimental apartment building was constructed in Sevastopol employing vibration dampers which utilize sand friction. Design parameters of this building are described and its behavior analyzed.

● 7.3-7 Sardzhaev, Ch. K., **Cost calculation of the reconstruction of buildings after earthquakes** (Raschet zatrat na vosstanovitelnyi remont stroitelnykh konstruktsii posle zemletryaseniya, in Russian), *Seismostoikoe stroitelstvo, 5,* 1977, 34–36.

Calculations performed during the period of reconstruction following the 1966 Tashkent earthquake show that the average reconstruction cost of brick buildings without aseismic design features was 54 percent of their appraised value, but for aseismic brick buildings the average cost was 10 to 15 percent of the appraised value. This points up the necessity of including the cost of reconstruction following a major earthquake in the construction and operation expenses of buildings in seismic areas. The comparative economic advantages arising from the application of interchangeable structural members for aseismic buildings constructed in regions with varying seismicity are analyzed.

● 7.3-8 Saburov, V. S., **Design of series 135 precast panel buildings for sites along the Baikal-Amur railroad** (Proektirovanie krupnopanelnykh zdanii serii 135 dlya vslovii stroitelstva BAM, in Russian), *Seismostoikoe stroitelstvo, 2,* 1977, 3–5.

Most of the territory along the projected route of the Baikal-Amur railroad is covered by permafrost up to a depth of 150 m. There are several seismically active regions within the area. Living quarters for construction crews, recreation, and office buildings are mass produced at centrally located factories. The foundations and supporting structures of these buildings must be designed for complex geological conditions. Design specifications of the series 135 precast panel building designed at the Kucherenko Construction Research Inst. are presented.

● 7.3-9 Aizenberg, Ya. M. and Chachua, T. L., **Optimization of parameters of a tall building with disengaging joints** (Optimizatsiya parametrov mnogoetazhnogo zdaniya s vyklyuchayushchimisya svyazami, in Russian), *Seismostoikoe stroitelstvo, 3,* 1977, 11–15.

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The effectiveness of rigid disengaging joints for increasing the earthquake resistance of buildings is investigated. An example of a 14-story building is used to illustrate the optimization technique developed by the authors.

● **7.3-10** Pukhovskii, A. B. and Gordin, V. A., **Earthquake-resistant arch spanning 62 m with prestressed steel panels** (Seismostoikoe arochnoe pokritie proletom 62 m s predvantelno napryazhennymi stalnymi panelyami, in Russian), *Seismostoikoe stroitelstvo, 3,* 1977, 3–7.

An experimental design for an earthquake-resistant skating rink roof is analyzed. The supporting structure of the roof consists of parabolic steel arches with a span of 62 m and a height of 22 m. Three-layered lightweight prestressed panels were used to reduce seismic loads on the structure. An approximation method was used to calculate the first five vibration mode shapes of the structure. The method applied led to an eight percent reduction of the required arch cross section area compared to traditional methods.

● **7.3-11** Aizenberg, Ya. M. and Chachua, T. L., **Application of disengaging joints to increase earthquake resistance of a tall building with stiffening diaphragms and core** (Primenenie vyklyuchaushchikhsya svyazei kak elementa seismozashchiti mnogoetazhnogo zdaniya s diafragmami i stvolami zhestkosti, in Russian), *Seismostoikoe stroitelstvo, 3,* 1977, 7–11.

The special problem of seismic processes consists of the great variation of intensity and spectral characteristics (dominant frequencies, effective duration of ground motion, etc.) of earthquakes. Therefore, structures must be designed on the basis of unreliable information concerning seismic excitation parameters. The effectiveness of adaptive structures is investigated. One of the features of such systems is the utilization of disengaging joints. A minimax problem is formulated to search for the most destructive seismic excitation among those physically possible and for the minimum cost of aseismic design features.

● **7.3-12** Makhviladze, R. L. and Malaniya, M. G., **Attachment of wall panels to columns** (Kreplenie peregorodochnykh panelei k kolonnam, in Russian), *Seismostoikoe stroitelstvo, 6,* 1977, 3–5.

A method for attaching wall panels to columns by means of plate elements is discussed. The method simplifies installation and reduces the cost of construction. The economic implications of the method are analyzed. The technique is suitable for construction in seismic areas.

● **7.3-13** Mardzhanishvili, M. A. and Chanukradze, G. Sh., **Prefabricated reinforced concrete frames with cross beams hidden in floor slabs** (Sbornyi zhelezobetonnyi karkas so skrytymi v perekritii rigelyami, in Russian), *Seismostoikoe stroitelstvo, 6,* 1977, 5–8.

Earthquake-resistant framed structures with wide cross beams incorporated in the floor slabs are investigated. Such cross beams increase the plane stiffness of floor slabs. Experimental investigations to determine the stiffness parameters of this type of structure were carried out. The application of such frames in various structures leads to reduced construction time in the prefabrication and installation of frame elements.

● **7.3-14** Kilimnik, L. Sh., **Performance evaluation of various designs of multistory buildings for seismic regions** (Otsenka sovershenstva konstruktivnykh reshenii pri proektirovanii mnogoetazhnykh zdanii dlya seismicheskikh raionov, in Russian), *Seismostoikoe stroitelstvo, 6,* 1977, 20–23.

A method is presented to evaluate the performance of various designs of multistory buildings for seismic regions. The method is based on the analysis of the strength, stiffness, natural frequencies, and the economic and technological characteristics of more than 200 designs carried out at the Kucherenko Inst.

● **7.3-15** Cherkashin, A. V., Konovodchenko, V. I. and Fetisova, V. I., **Hollow ceramic bricks for construction of ordinary or earthquake-resistant buildings** (Keramicheskie pustotelye kamni dlya konstruktsii obychnykh i seismostoikikh zdanii, in Russian), *Seismostoikoe stroitelstvo, 5,* 1977, 7–12.

Applications of hollow ceramic bricks in earthquake-resistant construction are discussed. The advantages include thinner walls and higher supporting structures. The main disadvantage is the brittleness of ceramic bricks subjected to dynamic loads. Various methods to improve the plasticity properties of ceramic brick structural members are discussed.

● **7.3-16** Kilimnik, L. Sh., **Design of frame buildings with elastic friction joints on high-strength pins for seismic regions** (Proektirovanie karkasnykh zdanii dlya seismicheskikh raionov s uprugo-friktsionnymi soedineniyami na vysokoprochnykh boltakh, in Russian), *Seismostoikoe stroitelstvo, 5,* 1977, 12–17.

The utilization of elastic friction joints on high-strength pins in earthquake-resistant steel frame buildings is discussed. Joints of this type may substantially reduce seismic loads and increase energy dissipation in buildings. Results of experimental investigations carried out at the Dept. of Strength and New Types of Steel Structures of the Kucherenko Inst. in Moscow are presented.

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● 7.3-17 Pukhovskii, A. B., **Investigation of the effectiveness of using prestressed steel trusses in earthquake-resistant construction** (Issledovanie effektivnosti primeneniya prednapryazhennykh stalnykh ferm v seismostoikom stroitelstve, in Russian), *Seismostoikoe stroitelstvo, 5*, 1977, 30-34.

The behavior of various types of prestressed steel trusses used for the roofs of earthquake-resistant industrial buildings is analyzed. Discussed are the parameters of the type of roof truss found most suitable for application in earthquake-resistant construction.

● 7.3-18 Polyakov, S. V., Kotov, Yu. I. and Alekseenkov, D. A., **Effects of the thickness of composite separating walls on their carrying capacity under compression** (Vliyanie shirini kompleksnykh prostenkov na ikh nesushchuyu sposobnost pri szhatii, in Russian), *Seismostoikoe stroitelstvo, 5*, 1977, 26-30.

To strengthen brick walls of buildings constructed in seismic regions, composite separating walls containing brickwork and reinforced concrete elements are used. The effectiveness of such walls was investigated, using full-scale models subjected to horizontal loads and central compression.

● 7.3-19 Eisenberg, I. M., **Safety of structural systems with reserve elements for earthquake protection**, *Earthquake Engineering and Structural Dynamics*, **5**, *3*, July-Sept. 1977, 305-318.

Some results of approximate analyses of the safety of earthquake protection systems with reserve elements are presented. Systems with one and several reserve elements are considered. The overshoot random vibration approximation is used for analysis. A numerical example is given. It is shown that the failure probability of structures with reserve elements is considerably lower, and the safety is much higher, when compared with similar characteristics of structures without reserve elements.

● 7.3-20 Fintel, M. and Schultz, D. M., **A philosophy for structural integrity of large panel buildings**, *Wind and Seismic Effects*, VI-220-VI-251. (For a full bibliographic citation, see abstract No. 1.2-4.)

The paper reviews the various methods to reduce risk from abnormal loads. To limit the occurrence of progressive collapse in large panel residential structures, a philosophy for establishing general structural integrity is developed to assure bridging of local damage while maintaining overall stability. This eliminates the need to design for any particular abnormal load. In this approach, tensile continuity and ductility of the elements, and their connections as well as that of the overall structure, is stressed. The rationale for a minimum tie system consisting of transversal, longitudinal, vertical, and peripheral ties to establish this general structural integrity is developed. The objective of this approach is not to afford absolute safety in regard to any exceptional event in any part of every building; rather, the intention is to limit and substantially reduce the general risk of collapse, as compared to that existing if no such measures were taken.

● 7.3-21 Culver, C. G., **Comprehensive seismic design provisions for buildings—A status report**, *Wind and Seismic Effects*, VI-61-VI-68. (For a full bibliographic citation, see Abstract No. 1.2-4.)

A review of the first draft of recently developed U.S. seismic design provisions for buildings is presented. The draft includes regulatory provisions suitable for inclusion as part of a building code and technical criteria for earthquake-resistant design. The provisions are intended for implementation by standards organizations, model code groups, federal agencies, and other regulatory groups. Technical criteria for structural design, architectural and mechanical-electrical design, and existing buildings are discussed.

● 7.3-22 Gosain, N. K., Brown, R. H. and Jirsa, J. O., **Shear requirements for load reversals on RC members**, *Journal of the Structural Division, ASCE*, **103**, *ST7*, Proc. Paper 13090, July 1977, 1461-1476.

Considerable experimental research on the influence of shear on the performance of flexural hinges under reversed loadings has been reported in the literature. Using existing data, a means of making comparisons between test results is devised and design requirements for shear in members under reversed loading are suggested. Results of 65 tests from six investigations are compared on the basis of a modified work index, a measure of the hysteresis exhibited by the test specimens under load reversals. Factors that significantly influenced the work index were level of axial force, shear span to depth ratio, and maximum applied shear stress. Design recommendations for maximum shear stress and minimum transverse reinforcement are developed for satisfactory performance of hinging regions.

● 7.3-23 McCue, G. M. and Kost, G., **The interaction of building components during earthquakes**, McCue Boone Tomsick, San Francisco, Jan. 1976, 209. (NTIS Accession No. PB 258 326)

Under certain conditions buildings must endure dynamic as well as static forces. For seismic action, a model of four elements provides a frame of reference for considering the dynamic role of building components: the ground, the integrated structure, interdependent elements, and dependent elements. The components of the enclosure, finish, and service systems which are not part of the

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integrated structure must be considered as either dependent elements or interdependent elements, in accordance with their degree of dynamic interaction with the integrated structure. Significantly different design criteria pertain, depending upon which of these dynamic roles they perform.

● 7.3-24 Robinson, Jr., J. H., **Inelastic dynamic design of steel frames to resist seismic loads,** *Evaluation of Seismic Safety of Buildings, No. 11, R77-23,* Dept. of Civil Engineering, Massachusetts Inst. of Technology, Cambridge, July 1977, 142.

This report details an investigation designed to determine the reliability of an inelastic design procedure based upon elastic modal analysis using an inelastic response spectrum proposed by Newmark and Hall. This was accomplished by computing the inelastic response, in terms of local member ductilities, of three steel moment-resisting frames to simulated earthquake motions derived from the design response spectrum. The effects of P-Δ forces, earthquake motion details, assumed damping level, and earthquake intensity were examined. The results indicate that the unmodified inelastic seismic design procedure is unconservative, most likely because individual modal responses, based on elastic modal properties, cannot be combined according to modal responses to compute total inelastic response with sufficient accuracy. However, it is possible to achieve the desired inelastic response, with respect to level of yielding and distribution of yielding throughout the structure, by increasing member strength or design forces. It is felt that in spite of the design difficulties demonstrated herein there is a great deal of promise in the use of inelastic spectra in the seismic design of structures. More research is necessary to determine necessary modifications to the inelastic design spectrum.

● 7.3-25 Sherbourne, A. N. and Krishnasamy, S., **Design of mild steel structures under unequal cyclic loads,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. L, Paper L 5/2, 12. (For a full bibliographic citation, see Abstract No. 1.2-5.)

Nonstatic loads applied to engineering structures are generally the rule rather than the exception. However, regardless of the method of estimating these loads, the ultimate interest of the design engineer is to design and predict accurately the behavior of a structure or component. It is therefore necessary to establish a relation between loads and deformations. Such relations, in some cases, could be highly nonlinear and quite complex. Since the response of a structure to excitation or nonstatic loads could also be a function of its deformation history, the need and value of a simplified approach is quite evident.

In this paper, a method is proposed to investigate the behavior and life of structural components under unequal cyclic loading conditions. Appropriate cyclic moment-curvature relations and life information, in the form of life versus extreme fiber strain, are developed from tests on beams under pure bending conditions. Theoretical predictions of behavior are based on structural geometry and the cyclic moment-curvature relations used in association with the simple curvature-area method. Structural life is also predicted using the life information developed and the theoretical strain history at the critical section in conjunction with a linear damage summation criterion. Theoretical predictions of behavior and life compare reasonably well with the experiments. Based on this study, a design procedure is proposed for mild steel components subjected to unequal cyclic loading conditions. The loads on the tested components were such that they failed due to low-cycle fatigue.

● 7.3-26 Gupta, A. K., Fang, S.-J. and Chu, S.-L., **A rational and economical seismic design of beam columns in steel frames,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(b), Paper K 9/7, 12. (For a full bibliographic citation, see Abstract No. 1.2-5.)

Beam columns in steel frames are required to resist axial forces together with biaxial moments when subjected to earthquake excitation in three directions. Design of a section of such a beam column depends on the axial force and the two moments on the section and also on the distribution of the two moments along the axis of the member. The seismic analysis is often performed using the response spectrum method. Various responses (e.g., forces and moments) in several modes of vibration under the three components of an earthquake are combined probabilistically by the SRSS (square root of the sum of the squares) method. In case of modes with closely spaced frequencies, a modified double sum approach is used. In either case, one obtains the maximum probable values of individual responses, which do not occur simultaneously. In the conventional design of beam columns, maximum probable values of the axial force and the moments at various sections are used as though they were acting simultaneously. Such a procedure yields a conservative design.

In the present study, a new rational procedure is used in which simultaneous variation in various response quantities is predicted. To design the beam column section according to the AISC Manual of Steel Construction, one has to know the values of the axial force, the moments about the axes at the two ends, and the maximum moments about the axes near the center of the beam column, which constitutes seven response quantities of interest for each beam column element. Normally, seven equivalent modes will be required to represent the response. However, by designing the two end sections and the intermediate section independently, one can consider three equivalent modes for each section, thus simplifying the problem a great deal.

An existing computer program is used for the implementation of the proposed method. Results for typical example problems have been presented. It is shown that savings up to 42% in the steel cross-sectional area can be obtained depending upon combination of various forces and moments. The proposed method is "exact" within the existing assumptions of the SRSS or the double sum method.

● 7.3-27 Feng, T.-T., Arora, J. S. and Haug, Jr., E. J., **Optimal structural design under dynamic loads,** *International Journal for Numerical Methods in Engineering,* **11,** *1,* 1977, 39-52.

This paper presents an algorithm for optimal design of elastic structures subjected to dynamic loads. Finite elements, modal analysis, and a generalized steepest-descent method are employed in developing a computational algorithm. Structural weight is minimized subject to constraints on displacement, stress, structural frequency, and member size. Optimum results for several example problems are presented and compared with those available in the literature.

● 7.3-28 McDonough, J. F., Baseheart, T. M. and Ringo, B. C., **Space frame simulated for structural design,** *Computers & Structures,* **7,** *6,* Dec. 1977, 747-750.

This paper presents a unique cost-saving analysis procedure for a complex space frame structure subjected to earthquake and wind loadings. The analysis is simplified by simulation of a structural model for a multistory unsymmetrical reinforced concrete building subject to substantial lateral loads. The concept combines two computer analysis procedures and reduces the time and cost incurred in the overall analysis. From the building's Y-shaped plan, multistory rigid frames were selected to resist motion transverse to the plane of each wing of the Y-shaped configuration. These so-called cross frames, by means of the first analysis program, provided the properties of transverse resistance needed to simplify the three-dimensional behavior for analysis by the second program. Significant savings in calendar time and costs were realized by the simulation.

● 7.3-29 Lepik, U. and Mroz, Z., **Optimal design of plastic structures under impulsive and dynamic pressure loading,** *International Journal of Solids and Structures,* 13, *7,* 1977, 657-674.

Optimal design of rigid-plastic stepped beam and circular plate is considered in the first part of the paper assuming the mode form of motion. The form of optimal mode is sought for which a structure of constant volume attains a minimum of local or mean deflection. It is assumed that the constant kinetic energy is attained by the structure through impulsive loading. Differences between optimal static and dynamic solutions are discussed. Nonuniqueness of modes is demonstrated and significance of stable mode motions is emphasized. In the second part of the paper, an optimal design of a rigid-plastic stepped beam loaded by a uniform pressure over a time interval is considered, assuming constant beam volume and searching for a design that corresponds to a minimum of local deflection. The solution presented is valid for moderate dynamic pressures when mode motion occurs during consecutive time intervals and no travelling plastic hinges exist.

● 7.3-30 Davidson, J. W., Felton, L. P. and Hart, G. C., **Reliability-based optimization for dynamic loads,** *Journal of the Sructural Division, ASCE,* **103,** *ST10,* Proc. Paper 13297, Oct. 1977, 2021-2035.

A general formulation is presented for weight optimization of indeterminate structures subject to transient dynamic loads and reliability constraints. Two distinct methods of structural analysis are examined and compared for use in the optimization algorithm: (1) numerical integration of equations of motion and (2) shock spectra. Details of the essential computation of standard derivation of response quantities associated with each analysis technique are also examined. The formulations are illustrated by design examples of a rigid frame subjected to an acceleration impulse applied to its base.

● 7.3-31 Fenwick, R. C. and Irvine, H. M., **Reinforced concrete beam-column joints for seismic loading—Part I: theory,** *Bulletin of the New Zealand National Society for Earthquake Engineering,* **10,** *3,* Sept. 1977, 121-128.

The basic theory relating to the design of reinforced concrete beam-column joints is reviewed and extended. In particular, the theory of an alternative joint detail using bond plates is developed.

● 7.3-32 Andrews, A. L., **Slenderness effects in earthquake resisting frames,** *Bulletin of the New Zealand National Society for Earthquake Engineering,* **10,** *3,* Sept. 1977, 154-158.

This study discusses lateral flexibility and displacement ductility controls which, if observed, will ensure that frame P-delta effects never become significant and so can be ignored. The controls are shown to be not too dissimilar from controls applied for other purposes; therefore, they are tolerable. The study is linked to the New Zealand design rules. An appendix extends it to U.S. rules.

● 7.3-33 Priestley, M. J. N., Crosbie, R. L. and Carr, A. J., **Seismic forces in base-isolated masonry structures,** *Bulletin* *of the New Zealand National Society for Earthquake Engineering*, **10**, 2, June 1977, 55–68.

Dynamic analyses of four-, eight-, and twelve-story masonry shear walls supported on base-isolation systems are described. Each wall was modeled as a multidegree-of-freedom cantilever. Results are presented which indicate that behavior is more complex, due to the significance of higher mode effects, than was previously believed on the basis of simple single degree-of-freedom models. The influence of different types of base isolation, and the influence of the stiffness of the gravity supporting system, are discussed. Tentative design recommendations are proposed which provide a substantial reduction in design forces for short-period structures while still maintaining an adequate margin of safety against the formation of wall hinges.

● 7.3-34 Delfosse, G. C., **The Gapec system: a new highly effective aseismic system**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. II, 1977, 1135–1141.

The Gapec system is a new aseismic system of the shock-absorbing soft-story type developed at the Centre National de la Recherche Scientifique in Marseille, France. With this system, a building is standing on energy-absorption isolators set between the first story and the basement. In addition, tall buildings are fitted with wind stabilizers located at the same level as the isolators. Experiments performed on a shaking table and dynamic designs of typical buildings show that using the Gapec system divides the acceleration response, shears, and overturning moments by a factor of five to eight.

● 7.3-35 Nishikawa, T., **Relation between yield strengths and response displacements of structures**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. II, 1977, 1100–1105.

To judge the safety of low- and medium-height framed-type reinforced concrete structures during strong earthquakes, the ductility evaluation concept is most commonly used. The relation between the amount and distribution mode of strength for the weak-column- and strong-girder-type structures and the response ductility factors of each story level are presented; and the required strength ratio and its distribution mode for the seismic-resistant design of such structures are discussed based on the response results for artificial earthquakes.

● 7.3-36 Murakami, M. and Penzien, J., **Nonlinear response spectra for probabilistic seismic design of reinforced concrete structures**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. II, 1977, 1046–1051.

Twenty each of five different types of artificial earthquake accelerograms were generated for computing nonlinear response spectra of five structural models representing reinforced concrete buildings. To serve as a basis for probabilistic design, mean values and standard deviations of ductility factors were determined for each model having a range of prescribed strength values and having a range of natural periods. Adopting a standard design philosophy, required strength levels were investigated for each model. Selected results obtained in the overall investigation are presented and interpreted in terms of prototype behavior.

● 7.3-37 Wang, P. C. and Drenick, R. F., **Critical seismic excitation and response of structures**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. II, 1977, 1040–1045.

The practicality of the critical excitation method for the design of structures is investigated. In the terminology of this paper, an excitation is critical for a structure if it generates a larger response peak in one of the design variables than any other possible excitation in some given class. The basic idea of the method is to draw up the design in such a way that the structure has sufficient reserve strength to sustain its own critical excitations up to a certain maximum intensity. This paper investigates more specifically a modification of the idea. It is shown, by an analysis of several existing and planned structures, that the modification leads to fairly realistic, if somewhat conservative, designs. The results encourage the conclusion that the modified method, or a similar one, may become a useful design tool for structures whose importance justifies conservative design.

● 7.3-38 Okamoto, T. and Yokoyama, M., **Earthquake-resistant design theory for prefabricated structures using checkered shear wall frames**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. II, 1977, 1917–1925.

The authors are engaged in research of an earthquake-resistant prefabricated building system allowing much freedom in architectural planning and backed by a wide range of experimentation with primary application to multistoried apartment house construction. The basis of this system is an earthquake-resistant element structured by shear walls reinforced around their peripheries by frames and positioned three dimensionally in a checkered pattern. The paper describes a practical and highly accurate elasto-plastic analysis method for this checkered shear wall frame and the method of earthquake-resistant design for the frame illustrated by an actual case of a 14-storied building.

● 7.3-39 Igarashi, S. *et al.*, **A study on shear-type structural model for aseismic design**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. II, 1977, 1185–1190.

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Earthquake response analysis of framed structures has various purposes, ranging from obtaining the aseismic design data for the preliminary structural design to examining the aseismic safety of the framed structure in the final form. This analysis has mainly been carried out by using a shear-type structural model in which a mass is concentrated on each floor level and connected with adjacent masses by shear springs having simplified hysteretic characteristics. In such a simulation, however, there remain such problems as whether a shear-type structural model may take into account the comprehensive responses (e.g., relative story displacements or story shear forces) of an actual structure and whether the real aseismic safety of a structure may be guaranteed by the comprehensive appraisal with a shear-type structural model. This paper attempts to clarify these problems by examining the accuracy and suitability of a shear-type structural model. The response of this model to an earthquake is compared with that of the more detailed structural model.

● **7.3-40** Muto, K. *et al.*, **Simulation analysis of a highrise reinforced concrete building in two different earthquakes,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1058-1064.

Past legal prohibition in Japan of reinforced concrete buildings exceeding 20 m and/or 6 stories in height was broken by the erection of the new 18-story Shiinamachi Building by Kajima Corp. The building was constructed after an unprecedented special approval by the construction minister. Following the building's completion, earthquake observations have been continued as a postconstruction study. Of the many recorded, two earthquakes whose accelerations registered remarkably the same as the building have been simulated. Using an idealized vibration model, excited motions are computed and compared with the observed behavior. It was found that they are identical, corroborating the reliability of the aseismic design of the building.

● **7.3-41** Freeman, S. A., **Modification of structures to satisfy new seismic criteria,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2511-2518.

An existing hospital facility was evaluated for earthquake resistance in accordance with new criteria, more severe than the original design criteria, established by the Veterans Administration Office of Construction. The conclusions of the evaluation are that the hospital complex does not conform to the new seismic criteria and that the structure will require major corrective action. Several strengthening modification schemes were explored for their structural feasibility, dynamic characteristics, and functional application. The decision process for selecting the proposed scheme is outlined here, and some of the structural details are presented.

● **7.3-42** Lee, D. L. N., Wight, J. K. and Hanson, R. D., **Repair of damaged reinforced concrete frame structures,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2486-2491.

An investigation of the effectiveness of repair of reinforced concrete exterior beam-column subassemblages is presented. The epoxy injection technique and the removal and replacement technique, using high early strength materials, were used to repair the beams. Based on the results of this investigation, it was concluded that epoxy injection and removal and replacement techniques of repair can restore structural integrity to the members.

● **7.3-43** Wyllie, Jr., L. A. and Degenkolb, H. J., **Improving the seismic response of braced frames,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1911-1916.

The energy-absorbing capacity and ductility of steel braced frames can be increased by the introduction of bending stresses through eccentric connections. The benefits of stiff elements to minimize drift with its consequent damage and increased stability can be combined with the ductility of the moment frame in a very economical construction. Design procedures are suggested.

● **7.3-44** Popov, E. P. *et al.*, **On seismic design of R/C interior joints of frames,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1933-1938.

Two alternative designs for beam reinforcement at the interior joints of moment-resisting reinforced concrete frames are discussed. The proposed details, verified by experiments, avoid the danger of bond degradation of the main beam bars within the column.

● **7.3-45** Kato, B. and Akiyama, H., **Earthquake resistant design for steel buildings,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1945-1950.

An ultimate strength design method for steel-frame buildings subjected to severe earthquakes is presented. The proposed method is based on current knowledge of the inelastic deformability of steel skeletons and the elementary response characteristics of inelastic vibrational systems observed through numerical analyses. An underlying concept is the energy concept similar to that proposed by Housner. The structure is capable of resisting an earthquake when the energy absorption capacity is greater than the energy input by the earthquake. The concentration of

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the energy input into a story is the most important matter to be considered in the design of multistory structures.

● 7.3-46 Eisenberg, J. M., Neiman, A. I. and Abakarov, A. D., **Limit states and mathematical models of structures determined from seismic risk optimization problem,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1356–1362.

Some results of the analysis of the seismic risk optimization problem are described. It is shown that the safety of structures designed for different seismic zones is unequal at present. A scheme for optimum seismic zoning is suggested which takes into account the probability of recurrence of earthquakes of different intensity. The problem of choosing mathematical models for the optimum design of structures under earthquake loads is considered.

● 7.3-47 Paramzin, A. M. and Gorovits, I. G., **Analysis of effect of lightweight concrete use in seismic-resistant multistorey buildings,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 2124–2125.

A computer analysis has been made of design intensity 9 seismic load and force distribution in frame elements of 5- to 18-story framed buildings, with ordinary heavy concrete fully replaced by ceramsite concrete of equal strength, and of 1200 to 1800 kg/m^3 volume weight in all structural and and nonstructural elements, such as columns, beams, floors, walls, and partitions. The analysis has been carried out on the basis of earthquake-resistant dynamic theory in conformity with the U.S.S.R. seismic code, using a design of a cantilever system with concentrated masses, the number of which is equal to the number of stories.

● 7.3-48 Stepanian, V. A., **Ultimate state parameter and seismoresistance reserve studies on the basis R. C. column and frame tests under dynamic and static cyclical loading,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 2126–2129.

● 7.3-49 Cheng, F. Y. and Srifuengfung, D., **Earthquake structural design based on optimality criterion,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 2096–2097.

The results of an optimum design for framed structures subject to earthquake motions are presented. The objective of the design procedure is to obtain a minimum weight for a structure without exceeding the strength limits that are determined by the stiffness requirement and optimality criterion. The numerical examples provided are based on the lateral forces recommended by the U.S. Uniform Building Code and Housner's response spectrum of the 1940 El Centro record.

● 7.3-50 Glina, Y. V., **Structural design of earthquake-proof frameless in-situ concrete residential buildings in the USSR,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 2069–2078.

This report is a short survey of the in-situ housing construction development in seismic areas of the U.S.S.R. The paper deals with the principal solutions and designs of structural members in aseismic frameless buildings, such as walls, floor-slabs, and reinforcement systems.

● 7.3-51 Zagajeski, S. W. and Bertero, V. V., **Computer-aided optimum seismic design of ductile reinforced concrete moment-resisting frames,** *UCB/EERC-77/16,* Earthquake Engineering Research Center, Univ. of California, Berkeley, Dec. 1977, 146. (NTIS Accession No. PB 280 137)

A computer-aided design procedure based on limit state design concepts is proposed for multistory reinforced concrete frames of buildings which are expected to experience severe earthquake ground shaking during their service life. In this procedure, a structure is designed to meet (1) various serviceability criteria under service loading conditions, (2) damage limitations for abnormal environmental conditions, and (3) safety requirements for extreme earthquake excitations. The design procedure, which makes use of computer optimization methods as well as static and dynamic elastic and nonlinear analysis procedures, consists of five basic steps which are grouped into a preliminary design phase and a final design phase.

The objective of the preliminary design phase is to obtain a preliminary design which is as close as possible to the desired final design. It entails three steps, preliminary analysis, preliminary design, and analysis of the preliminary design. In the first step, serviceability, damageability, and safety criteria, gravity and wind loading conditions, and the design earthquakes are established according to the site, type, and function of the building. Seismic design shears are obtained from an inelastic design response spectrum constructed for appropriate values of damping, and acceptable values of displacement ductility and average story drift using an iterative preliminary analysis procedure based on modal analysis.

In the second step, a preliminary design of structural members is obtained employing a simplified storywise optimization procedure. The optimization objective is to minimize the volume of flexural reinforcement. A weak girder-strong column design philosophy is followed in the formulation of the design problem. Design constraints are

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imposed to satisfy equilibrium and to arrive at a serviceable and practical design. The solution of the optimum design problem yields beam design moments which are used in conjunction with the weak girder-strong column design philosophy to obtain member sizes and reinforcement.

Once the preliminary design is completed, structural response to various loading conditions is computed (step 3). Both elastic and nonlinear behavior is considered under static and dynamic loading. The results of these analyses are evaluated with respect to the assumptions made and design criteria established in step 1 to determine whether the design is acceptable. Steps 1 through 3 are repeated until satisfactory agreement is reached and the final seismic design story shears can be obtained.

The objective of the final design phase is to obtain an optimum final design. This phase consists of two steps, final design and an analysis of the reliability of the final design. The final design is found by solving an optimization problem based on a more sophisticated and realistic design subassemblage than that used in the preliminary design and using the final seismic design shears obtained in the preliminary design. In the second step, the reliability of the final design is checked under service and ultimate loading conditions. Information needed for detailing critical regions in the structure where yielding and significant inelastic deformations may take place is obtained by determining its nonlinear time history response to severe earthquake ground motions.

The design procedure is illustrated on a ten-story, three-bay reinforced concrete frame. The influence of the different limit states considered is indicated by the example. This computer-aided design procedure is shown to be versatile. The use of computers allows alternate designs to be rapidly and economically formulated and evaluated, and it allows realistic consideration of complex environmental actions. The nonlinear dynamic response of the designed structure indicates that the procedure is capable of achieving acceptable designs if the response spectrum used in the definition of the design forces adequately represents the characteristics of the ground motion which excites the structure.

● 7.3-52 Walker, Jr., N. D., **Automated design of earthquake resistant multistory steel building frames,** *UCB/EERC-77/12,* Earthquake Engineering Research Center, Univ. of California, Berkeley, May 1977, 179. (NTIS Accession No. PB 276 526)

This report presents a methodology for automating the design process for earthquake-resistant multistory steel building frames. The design process is viewed as a complex collection of interrelated decision processes, the conduct of which requires specification of the motivation for making the decisions and identification of the decision constraints. Total cost, including both construction-related expenses, as well as the cost of expected future damage, is adopted as the basic decision motivator. Decision constraints are composed essentially of standard and projected building code restrictions.

The design process as a whole is explored first, followed by a detailed investigation of the "frame-sizing" portion of this process. Static loading, in the form of dead/live load on the beams, and earthquake-generated horizontal ground motion are considered in evaluating structural system response. Linear and approximate nonlinear analyses are employed. Expressions describing the frame-sizing process are introduced. In addition, an automating algorithm is presented. These procedures are then employed on two example problems which serve to develop insight into the design philosophy under study and into the operating characteristics of the proposed automated design procedure.

● 7.3-53 Bresler, B. *et al.*, **Developing methodologies for evaluating the earthquake safety of existing buildings,** *UCB/EERC-77/06,* Earthquake Engineering Research Center, Univ. of California, Berkeley, Feb. 1977, 157. (NTIS Accession No. PB 267 354)

This report contains four papers written during an investigation of methods for evaluating the safety of existing school buildings under National Science Foundation–Research Applied to National Needs grants. These papers are not readily available to researchers and engineers in the United States and are therefore issued as a single report.

● 7.3-54 Kelly, J. M., Eidinger, J. M. and Derham, C. J., **A practical soft story earthquake isolation system,** *UCB/EERC-77/27,* Earthquake Engineering Research Center, Univ. of California, Berkeley, Nov. 1977, 150. (NTIS Accession No. PB 276 814)

This report describes the experimental and analytical results of a practical earthquake isolation system. The experimental work was carried out using a 20 ton three-story single-bay moment-resistant steel frame structure on the 20 by 20 ft shaking table at the Earthquake Engineering Research Center at the Univ. of California, Berkeley.

The soft-story isolation system is composed of elastic natural rubber bearings and a highly nonlinear energy-absorbing device, all placed beneath the base floor of the model structure. The bearings allow for lateral movement of the base of the model and are designed so that no adverse column P-Δ effects can occur. The energy-absorbing devices act as highly efficient dampers and are based upon the two-way plastic torsion of steel bars. For smaller earthquakes, the structure behaves as it would with a rigid foundation. For large earthquakes, the structure's first mode period increases from 0.6 to 1.0 sec and equivalent first mode damping is between 30% and 35%. Thus, for destructive earthquakes, the use of the isolation system typically reduces the structure's response by over 50% of that of a conventional rigid foundation structure. An inelastic time-history analysis gives good correlation with experimental test data. A simple design procedure based upon elastic response spectra is suggested.

A full-scale structure located in a seismic zone and built with such an isolation system achieves two major cost benefits over a conventional structure: (1) lower initial construction costs due to reduced lateral load requirements; (2) lower earthquake-caused repair costs due to decreased structural and nonstructural damage.

7.3-55 West, W. H. W. *et al.*, **The resistance of brickwork to lateral loading**, *The Structural Engineer*, 55, *10*, Oct. 1977, 411–430.

Part I describes flexural tests on small masonry specimens and tests on full-sized walls, without preload, up to 5.5 m long and 3.6 m high, uniformly loaded laterally by means of air bags. Results for 61 different clay bricks and three mortars have enabled characteristic flexural strengths to be related to the water absorption of the units. A few results for concrete block walls are also reported. At constant height and thickness, the lateral load resistance is proportional to the reciprocal of the wall length; and cavity walls behave as the sum of the two leaves, providing the ties are stiff enough to transmit load. The effect of height is assessed, and the contribution made by top anchors is considered. The effect of different materials is considered.

In Part II, several methods of design are compared with the experimental results, and an interim method based on yield-line theory is favored. A semi-empirical approach is also examined. The possibility of arching action is considered, but no allowance is made for it so that it provides a hidden reserve of safety. Although there are still difficulties, notably the effect of openings, it is possible to make rational engineering calculations to give factors of safety consistent with the limit state approach to design.

● **7.3-56** Hisada, T., Bessho, S. and Okamoto, K., **Consideration of earthquake design for reinforced concrete columns of tall buildings**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. II, 1977, 1892–1897.

A new column, the KS column, has been developed for tall reinforced concrete buildings. The column system is designed for aseismicity and simplicity of construction. Described are the earthquake-resistant properties of the column, and a comparison is made with the aseismic effects of conventional lateral reinforcements of the columns. The application of the technology to practical construction and the results of experiments on the aseismic characteristics of KS columns in the lower stories of tall buildings are discussed.

● **7.3-57** Portillo, M. and Ang, A. H.-S., **Safety of reinforced concrete buildings to earthquakes**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. II, 1977, 1886–1891.

The safety of buildings to earthquakes of specified intensity is evaluated in terms of probability of failure. Probabilities of the first sign of distress, such as yielding, are assessed for reinforced concrete buildings designed in accordance with current earthquake-resistant codes.

● **7.3-58** Kost, G. *et al.*, **The interaction of building components during seismic action**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. II, 1977, 1880–1885.

A unified design approach is developed to account for the interaction of building components under seismic loading. A four-part model is presented which provides a dynamic viewpoint for design strategies. This model consists of the ground, the integrated structure, interdependent elements, and dependent elements. Enclosure, finish, and service systems may be part of the integrated structure or interdependent or dependent elements. Specific, but significantly different, design strategies apply, depending upon which of these dynamic roles they perform. Other issues discussed include the nature of in-building motions and principal design and analysis methods for building components.

● **7.3-59** Matsushita, K., Izumi, M. and Ide, H., **Some analyses of mechanisms to decrease earthquake effects to building structures (Part 4): steel dampers**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. II, 1977, 1874–1879.

There are two fundamental methods in the aseismic design of buildings: one is to make structures strong enough to resist earthquake forces, and the other is to control input earthquake forces and minimize the effects to the structures. In this paper, the installation of a steel damper is proposed and the effect of earthquakes on structures so constructed is discussed. When this damper is installed in combination with a double column system in tall buildings of 15 to 20 stories, the shearing force produced by earthquakes is reduced by about 30% in comparison with ordinary structures of a similar height.

● **7.3-60** Ruiz, P. and Depassier, H., **Ductility required in code designed structures**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. II, 1977, 1868–1871.

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The dynamic response of elastoplastic systems subjected to a set of earthquake records is analyzed using Monte Carlo techniques to evaluate the general trend of nonlinear response spectra associated with ductility requirements. These response spectra, for various earthquake intensities, are compared to a typical design spectrum, as specified by earthquake-resistant design regulations, to determine typical ductility requirements for code-designed structures.

● 7.3-61 Baratta, A., **Ultimate analysis of plastic structures in a seismic environment,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1864-1867.

The problem of progressive weakening of structures under subsequent earthquakes is discussed in order to define an "ultimate threshold" of structural damage. This threshold is defined as the maximum damage a structure can suffer with a sufficiently small probability of collapse during the next earthquake. An example is given to show the possibility of calculating admissible damage as a function of the seismic strength of the undamaged structure. It is pointed out that a lower bound of seismic strength exists such that a small amount of damage may be allowed before condemnation of the building.

● 7.3-62 Englekirk, R. E., **A comparative study of seismic design criteria for ductile structures,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1858-1863.

Design criteria for ductile structures must be carefully evaluated prior to their use. Analytical techniques and material testing have made such an evaluation meaningful. A procedure for evaluating the effectiveness of design criteria for ductile structures is presented and used in evaluating five design criteria. The evaluation is made on a 10-story ductile concrete frame and a 10-story ductile steel frame. Member ductility demand is compared with cyclic load test results.

● 7.3-63 Anagnostopoulos, S. A. and Biggs, J. M., **An evaluation of response spectra design procedures through inelastic analysis,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1841-1847.

The use of modal analysis with inelastic spectra for design and response prediction is evaluated using a computer program that integrates numerically the nonlinear equations of motion. Two buildings, a 10-story reinforced concrete structure and a two-story steel structure with moment-resisting frames, designed using inelastic spectra, are analyzed for an artificial motion which matches approximately the design spectra. Comparisons are made and some tentative conclusions are drawn.

● 7.3-64 Kobori, T., Minai, R. and Fujiwara, T., **Optimum design of the structural members due to ground motion,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1813-1818.

Optimum distribution of the dynamic characteristics of structural members with elastoplastic joints is discussed. This distribution minimizes the deviation of the root mean square responses of ductility ratios of column and girder joints caused by random excitation. The elastic and inelastic responses of structures subjected to earthquake motions are computed, with plastic flow of axial deformation taken into consideration. Aseismic design data for structures with strong and weak girders are presented.

● 7.3-65 Mujumdar, V. S., **An approach to earthquake resistant design of precast concrete bearing wall buildings,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 2093-2095.

It is inherent in the nature of precast concrete buildings that several joints exist between various components. The joint material may be different than that of the components; furthermore, the various components may not be positively connected. The behavior of the structure is difficult to predict because of the presence of joints; however, it is necessary that all components should be well tied together. Some details are presented that take into account standard available products.

● 7.3-66 Klingner, R. E. and Bertero, V. V., **Infilled frames in earthquake-resistant construction,** *EERC 76-32,* Earthquake Engineering Research Center, Univ. of California, Berkeley, Dec. 1976, 291. (NTIS Accession No. PB 265 892)

The effects of engineered masonry infill panels on the seismic hysteretic behavior of reinforced concrete frames are investigated experimentally and analytically. The experimental phase consists of quasistatic cyclic load tests on a series of one-third-scale model subassemblages of the lower three stories of an eleven-story, three-bay frame with infills in the two outer bays. Emphasis is placed on simulating the proper force and displacement boundary conditions. The engineered infilled frames are designed and constructed in accordance with the following guidelines: (1) frame members (particularly the columns) are designed for high rotational ductility and resistance to degradation under reversed cyclic shear loads, (2) gradual panel degradation is achieved by using closely spaced infill reinforcement, and (3) panel thickness is limited so that the infill cracking load is less than the available column shear resistance.

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The infilled frames are found to offer many advantages over comparable bare frames, particularly with respect to their performance under strong ground motions. The analytical phase consists of developing relatively simple, macroscopic mathematical models for predicting the experimentally observed bare and infilled frame behavior. The infilled frame model is found to give excellent predictions of the observed response. It is concluded that the procedure can be applied to the analysis of large, engineered infilled frame structures. The aseismic design implications of these results are discussed, and areas for further investigation are recommended.

7.3-67 Gupta, A. K. and Singh, M. P., **Design of column sections subjected to three components of earthquake,** *Nuclear Engineering and Design,* **41,** *1,* Mar. 1977, 129–133.

This paper presents the derivation of a procedure for the seismic design of column sections subjected to combinations of axial force and moments which probabilistically can occur simultaneously. The design process simply consists of providing a section with an interaction diagram or surface which would completely include the appropriate interaction ellipse or ellipsoid. The concept of interaction ellipse or ellipsoid can be easily coded into existing computer programs which design sections by calculating the points on the interaction diagram or surface. The proposed method will result in the most economical design of the section. However, where economics is not a major consideration, the usual methods of designing for square root of the sum of the square values will still provide a safe section.

● 7.3-68 Nemtchinov, Ju. I. *et al.,* **Design method and experimental investigations of seismic stability of buildings from box units,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 2013–2018.

Design and erection of structures in seismic areas from bearing box units are considered. The design is based on a variational formulation of equilibrium conditions in the displacement of thin-wall spatial systems from the separate finite elements of the box structures. Results of experimental studies of strength and stiffness of box-unit structures are described. Studies were conducted with natural fragments and with elastic (glass) and natural (concrete) models with similar properties.

● 7.3-69 Djabua, Sh. A. *et al.,* **Research on seismic resistance of large-panel apartment buildings,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 2041–2045.

Improvement of research and design methods for large-panel structures in seismic areas is investigated. A design for a large-panel building is presented, using a three-dimensional system of panels. Design analysis is by the finite element method, with inside and outside wall panels and floor slabs as the finite elements. A definition for the elements of panel stiffness matrices is proposed. Aseismic design is in accordance with U.S.S.R. design codes. Panel interaction forces, caused by vertical loads and seismic motion, are defined analytically.

● 7.3-70 Velkov, M. and Simeonov, B., **Earthquake resistant design of ten storied mixed steel and reinforced concrete structure with infill brick masonry,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1975–1979.

An architectural concept was adopted by the investor for several ten-story residential buildings, using a steel structure and infill walls of light prefabricated blocks. In applying this concept to the actual design, there were difficulties in finding an adequate economic solution for the main steel structure system, which along with the infill walls and the remainder of the structure, had to be earthquake resistant. After analysis and study of various architectural solutions, a system consisting of a steel frame structure and a composite steel reinforced concrete core was selected. The system was analyzed and optimized for static and dynamic loads and linear and nonlinear response of structural members. The results show that the system completely satisfies safety and economic criteria.

● 7.3-71 Velkov, M. P., **Earthquake resistant design of twenty-one story prefabricated large panel building,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1980–1985.

To determine the seismic stability of a selected, relatively tall prefabricated large-panel structural system, the Inst. of Earthquake Engineering and Engineering Seismology, Skopje, Yugoslavia, conducted experimental and theoretical investigations of several apartment structures which were built by this system. Engineering seismological investigations were carried out on the Zemun-Beograd site in order to determine intensity and time duration of earthquake motion. Forced-vibration tests of embedded piles and foundation-structure fragments were simultaneously performed. Full-scale forced-vibration tests on a similar 17-story structure were implemented in order to adopt a mathematical model for the system and to use it in the preliminary analysis of a 21-story structure. The results showed that the joints of the system should be improved. The strength and deformation properties of new joints were defined by experimental quasistatic full-scale tests.

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On the basis of the investigations, a mathematical model for the system, soil-structure interaction, and seismic input data was defined. Static and dynamic linear and nonlinear analyses of the structural members and the system in general which was considered three-dimensional, were carried out. The concept of monolithic stories as the largest energy absorbers was adopted in the construction of the lower stories providing suitable ductility capacity without strength and deformability deterioration. A full-scale forced vibration test was performed on a structure of this design, and comparison is made between the results of the test and the results of the mathematical model analysis of elastic behavior.

● 7.3-72 Igarashi, S. *et al.*, **A study on the earthquake resistance design of braced multi-story steel frames,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1939–1944.

A method is described for obtaining the vertical distribution of the ratios of story shear force shared by bracings to the total story shear force along the height of the frame when the ultimate factored design loads are given. The effect of horizontal deflection caused by column shortening and elongation of the braced bay on the ratios is taken into account in the designing process. A design example, using the method, is illustrated for a ten-story three-bay braced frame. Overall inelastic static and dynamic behaviors of the frame are examined, and the response quantities are compared with those of an allowable stress designed brace frame and a plastically designed open frame.

● 7.3-73 Glogau, O. A., **Damage control in New Zealand public buildings through separation of non-structural components,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1773–1779.

In New Zealand, rigid nonstructural components in public buildings have for over ten years been detailed separately from the structures. The justification, objectives, and design basis for this damage control strategy are discussed with respect to recorded earthquake damage. Practical details for connections of exterior precast panels, curtain walls, partitions, and stairs are given. Economics are briefly discussed along with the problem of successfully implementing the designer's aims.

● 7.3-74 Bertero, V. V. *et al.*, **Seismic design implications of hysteretic behavior of reinforced concrete structural walls,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1898–1904.

After reviewing current code provisions for aseismic design of R/C structural wall systems and present knowledge of their hysteretic behavior, research being conducted on this subject at Berkeley is summarized. The facility for testing frame-wall subassemblages, details of the 1/3-scale wall subassemblage models tested, the fabrication procedure, mechanical characteristics of the materials, and selection of the test loadings for these models are discussed. The results are evaluated, and their implications for aseismic design of frame-wall structural systems are discussed.

● 7.3-75 Pekin, O. *et al.*, **A construction method providing high earthquake resistance in reinforced concrete buildings,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 2064–2068.

A method is presented for aseismic construction which isolates a foundation from the superstructure. Instead of the attached foundation method, the "floating foundation" concept is applied. This method stiffens the superstructure by means of rigid cornered triangles and/or trusses with stable surfaces.

● 7.3-76 Izmailov, Yu. V., **Present-day solutions of antiseismic stone buildings,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 2025–2028.

Frame and stone structures, whose main walls are made of a combination of masonry and prefabricated monolithic or monolithic reinforced concrete frames, are becoming more common. During the construction of a building, the wall masonry is utilized as a form for the reinforced concrete elements of the frame. The reinforcement of these elements is minimal. Such structures can be built to heights of at least 12 stories. Experimental and theoretical examination of stone structures of different kinds of wall construction showed that, during earthquakes, they exhibit high seismic stability while meeting a number of other desirable conditions. The main results of these experiments and the requirements imposed on design and construction of aseismic stone structures is presented.

● 7.3-77 Ismailov, Yu. V. *et al.*, **Seismic stability of composed wall structures made of natural stone,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 2029–2033.

● 7.3-78 Leyendecker, E. V. and Ellingwood, B. R., **Design methods for reducing the risk of progressive collapse in buildings,** *NBS Building Science Series 98,* U.S. National Bureau of Standards, Washington, D.C., 1977, 68.

Progressive collapse is described as a chain reaction of failures following damage to a relatively small portion of a structure. The damage which results is out of proportion to

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the damage which initiated the collapse. The basic concepts associated with progressive collapse are described and the background leading to the concepts is summarized. Possible causes of progressive collapse are discussed, with concentration on abnormal events which have a low probability of occurrence but may have catastrophic consequences. A case study of the probability of structural failure as a consequence of one type of abnormal load (a gas explosion) shows that these probabilities exceed levels generally considered acceptable to engineers. Direct design strategies for reducing progressive collapse are described as (1) the alternate path method and (2) the specific local resistance method. Equations for load combinations, including appropriate load factors, are presented for each design method. The advantages and disadvantages of each approach are described, and it is concluded that the alternate path method affords the designer more flexibility. Although the design strategies are applicable to any type of structure at any time in its life cycle, this report provides detailed recommendations for completed buildings.

● 7.3-79 Merritt, R. G., **A seismic risk simulation model for Army facilities: phase one, development of deterministic model,** *CERL-SR-M-223*, U.S. Construction Engineering Research Lab., Champaign, Illinois, Aug. 1977, 103.

This report describes the first phase in the development of a method for assessing (1) the seismic hazard to Army facilities, and (2) the cost of reducing the hazard. A simulation model was developed to determine the cost of repairing damage to a facility resulting from seismic activity, as well as the cost of strengthening or replacing the facility to mitigate the effects of seismic activity. The cost to repair damage and the costs to strengthen and replace four-, seven-, and ten-story facilities are presented for three basic building configurations and a continuum of loads ranging from the 1968 Structural Engineers Assn. of California Zone 3 requirements to a 1.0 g response spectrum. An example for use of the model is given, along with two decision methodologies.

● 7.3-80 Heger, F. J. and Luft, R. W., **Structural evaluation of existing buildings in Massachusetts for seismic resistance,** *MIT-CE R77-44, Seismic Design Decision Analysis Report No. 33*, Dept. of Civil Engineering, Massachusetts Inst. of Technology, Cambridge, Nov. 1977, 128.

Results of a pilot study of technical aspects involved in determination and improvement of seismic resistance of older existing buildings in areas of low to moderate seismic risk, such as Massachusetts, are presented. The question of adequacy of seismic resistance arises particularly in the renovation, restoration or preservation of existing older buildings which often have substantial economic value and/or historical and architectural significance, and which often cannot be practically reinforced to meet existing seismic requirements given in present building codes.

Methods for evaluation of seismic resistance of existing structures are summarized and methods for determining behavior of buildings with nonductile structural systems are emphasized. Seismic requirements of the new Massachusetts State Building Code, as they affect renovations of existing buildings, are summarized, and a comparison of the implications of such requirements and the objectives originally postulated for introduction of seismic requirements into Massachusetts is discussed. A proposal for reevaluation of seismic design requirements for renovation of existing buildings is made, and tentative suggested code provisions are given. Finally seismic resistances are estimated for two existing buildings which are typical of buildings being renovated today in the Boston area. Available approximate methods of analysis are used and results are presented and compared with seismic requirements of the present Massachusetts State Building Code. One of the buildings analyzed is a six-story mill-type warehouse, with unreinforced masonry bearing walls and wood floors, originally constructed in the 1870s. The other is a six-story reinforced concrete flat-slab-frame factory building originally constructed in the 1930s. Original plans are available for the reinforced concrete structure. Lateral resistance in this building is provided by both the frame action of the reinforced concrete floors and columns and the exterior reinforced concrete frame of beams and columns which is infilled with partial height unreinforced brick masonry spandrel panels below windows. Very approximate estimates of the probable level of damage expected in earthquakes of various intensities are made for the two buildings studied, based on their existing condition and their condition if strengthened to the extent tentatively proposed in the study as a minimum code requirement.

● 7.3-81 Giles, W., **A methodology for seismic design and construction of single-family dwellings,** Applied Technology Council, San Francisco, 1977, var. pp.

This report examines design and construction practices for single-family residences that minimize the potential economic loss and the life loss risk associated with earthquakes. The report discusses the ways structures behave when subjected to seismic forces; and sets forth suggested design criteria for conventional layouts of dwellings constructed of conventional materials. It also presents construction details that do not require the designer to perform analytical calculations, suggests procedures for efficient plan-checking, and recommends details and schedules for use in the field by construction personnel and building inspectors.

● 7.3-82 Pinkham, C. W. and Hart, G. C., **A methodology for seismic evaluation of existing multistory residential buildings,** S. B. Barnes and Assoc. and J. H. Wiggins Co., Los Angeles, June 1977, 3 vols., 870.

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This manual describes a method of structural analysis, design, and analysis of cost for the determination of strengthening of existing multistory residential buildings to conform to the basic earthquake force requirements of the 1973 Uniform Building Code. The report is presented in three volumes: Volume 1 covers methodology, Volume 2 is a computer users' manual, and Volume 3 gives examples. The examples in Volume 3 illustrate both simplified and more complex evaluation of stress distribution and different types of multistory residences.

7.4 Design and Construction of Nuclear Facilities

7.4-1 Danisch, R. and Labes, M., **Aseismic design of turbine houses for nuclear power plants,** *Nuclear Engineering and Design,* 38, 3, Sept. 1976, 495-501. (For an additional source, see Abstract No. 1.2-2.)

The turbine house does not belong to the safety-related parts of equipment of a nuclear power plant. A special protection against earthquakes is not demanded by authorities as long as it is proven that safety-related parts of equipment will not be restricted in their function by a collapse of the turbine house. The degree of an aseismic design is largely up to the customer, who has to weigh the risk of costs and availability against the additional costs that are necessary for the earthquake calculation and for constructive hardening.

Low-tuned turbine foundations as a result of helical-spring-support, which are constructed by the KWU exclusively, pose special problems for aseismic design. This will be discussed in particular in this paper. The spring-supported mass constitutes about a quarter of the building mass. For mechanical reasons the spring elements are chosen in such a way that the turbine foundation has a natural frequency of about 3 Hz. Thus, it remains within the same frequency range as the turbine house and within that very range which is particularly amplified by an earthquake. It is therefore likely that resonance effects as well as oscillation annulment effects may occur.

The standardized calculation methods for conventional buildings without safety functions, such as DIN 4149 (Germany) or SIA 162 (Switzerland), do not cover the oscillation conduct of such a complicated structure. Information is obtained about possible relative displacements between the building and the turbine foundation (hammering effect) and about the stresses on the turbine and other components only by dynamic calculation methods such as the time-history or the response spectrum method. Depending on the assumed earthquake (operating basis earthquake), the following constructive hardening measures will be necessary: (1) Small earthquakes—reinforcement of the building structure; the concept of the building structure may be kept unchanged. (2) Medium earthquakes—reinforcement of the building structure; slight alterations of the building concept; installation of damping elements at the spring assemblies; and reinforcement of the turbine foundation without changing the concept. (3) Strong earthquakes—essential alteration of the building concept such as light steel construction or renunciation of the machine hall, additional shear walls; deeper foundation level of the building; installation of damping elements at the spring assemblies; reinforcement of turbine bearings; and modified design of the turbine foundation.

7.4-2 Reddy, D. V. and Kierans, T. W., **Dynamic analysis for design criteria for underground nuclear reactor containments,** *Nuclear Engineering and Design,* 38, 2, Aug. 1976, 177-205.

This paper is based on (i) the recent input of the authors for the Underground Containment Sub-section of the Seismic Task Group Report of the ASCE Committee for Nuclear Structures and Materials, and (ii) parametric studies carried out by the first author on the principal underground concepts.

The extensive work on aseismic design of above-ground reactors and recent studies on missile impact effects, aircraft impact, blast effects due to chemical explosions, reactor core melt-down and tornadoes indicate the advantages of underground siting with inherent general reduction to complexity of seismic amplification and benefits of structural and biological integrity. Other advantages are possibilities of urban siting, ecological considerations, reduced effects on the landscape, ability to design three dimensionally, separation of component facilities, support capability to equipment, reduced power transmission costs, increased number of acceptable units and power capability from a single location, and reduction of decommissioning problems.

In view of the limited actual experience in the structural design of underground containments (only four European reactors), the proposals are based on (a) the transposition of applicable design specifications, constraints and criteria from existing surface nuclear power plants to underground, and (b) the use of many years of experience in the structural design of large underground cavities and cavity complexes for other purposes such as mining, hydropower stations, etc. All concept options are assumed to be similar in design criteria for structural competence to contain radioactivity and fuel heat and meet the functional, servicing, protective and aesthetic requirements. The choice of underground siting should be based on criteria developed from the sequential consideration of load-causing phenomena, concept and site characteristics. From the criteria, loads for a particular concept and site are then calculated and the design formulated.

The state-of-the-art is presented and guidelines are outlined for (1) Load causing phenomena, (2) Underground siting concept considerations, (3) Siting factors and structural selection, (4) Structural types, (5) Analysis (including comparison of lumped parameter and finite element analysis), (6) Design procedures, and (7) Miscellaneous considerations (laboratory tests and field measurements, secondary equipment, faults, leakage of radioactivity, ground water control, environmental factors).

Parametric studies are described for structural characteristics of the four principal underground concepts: (a) Cut-and-cover in rock or soil, (b) Unlined cavity in rock, (c) Lined cavity in rock or soil, and (d) Lined cavity in rock or soil with annular filling of soft material—with respect to shape, backfill material, cavity wall reinforcement, passive and active rock bolting, lining and annular filling. The response to a step pulse, representing a blast excitation applied horizontally, is studied. As the character, intensity, duration and frequency of earthquake and blast-induced ground motions are roughly similar, the results have practical value in studying earthquake effects.

7.4-3 Howard, G. E., Ibanez, P. and Smith, C. B., **Seismic design of nuclear power plants—An assessment,** *Nuclear Engineering and Design,* 38, 3, Sept. 1976, 385-461. (Expanded version of Paper U3/1 presented at International Seminar on Extreme Load Conditions and Limit Analysis Procedures for Structural Reactor Safeguards and Containment Structures, Sept. 8-11, 1975. For a full bibliographic citation, see Abstract No. 1.2-2.)

This paper presents a review and evaluation of the design standards and the analytical and experimental methods used in the seismic design of nuclear power plants with emphasis on United States practice. Three major areas were investigated: (a) soils, siting, and seismic ground motion specification; (b) soil-structure interaction; and (c) the response of major nuclear power plant structures and components. The purpose of this review and evaluation program was to prepare an independent assessment of the state-of-the-art of the seismic design of nuclear power plants and to identify seismic analysis and design research areas meriting support by the various organizations comprising the nuclear power industry. Criteria used for evaluating the relative importance of alternative research areas included the potential research impact on nuclear power plant siting, design, construction, cost, safety, licensing, and regulation.

Three methods were used in the study herein. The first involved the review of current literature, focusing primarily on publications dated later than 1970. This review included the results of numerous studies, of which those of Japanese origin and those presented in recent international conferences were predominant. The second method entailed a review of international experience in the dynamic testing of nuclear power plant structures and components, and related experience with scaled and model tests. Included in this experience, in addition to the questions of analysis, design, and measurement of dynamic parameters, are related efforts involving a review of responses obtained during measured earthquake response and investigations into appropriate methods for backfitting or upgrading older nuclear power plants to meet new seismic criteria. The third approach was to obtain the opinions and recommendations of technically knowledgeable individuals in the U.S. nuclear industry; the survey results are shown in the appendix.

● 7.4-4 Watabe, M. and Ohsaki, Y., **Outline of basic philosophy and practices of aseismatic design for nuclear facilities in Japan,** *Wind and Seismic Effects,* VII-29-VII-35. (For a full bibliographic citation, see Abstract No. 1.2-4.)

In Japan, consistent standard specificatons or guidelines for the aseismic design of nuclear facilities have not been officially established. Only recently, in Mar. 1975, has the Regulatory Guide of Design Earthquake Ground Motions for Nuclear Power Facilities been proposed. The proposed regulatory guide has already had an effect in a practical sense in spite of its interim nature. It is the purpose of this paper to briefly summarize the philosophies and procedures of this aseismic design guide which is currently in use in Japan.

● 7.4-5 Shao, L. C., Stuart, R. J. and Hofmayer, C. H., **Wind and seismic design of U.S. nuclear power plants,** *Wind and Seismic Effects,* VII-1-VII-28. (For a full bibliographic citation, see Abstract No. 1.2-4.)

A brief discussion of wind and seismic design of U.S. nuclear power plants has been presented. All plants are currently designed to resist the effects of severe environmental loads in combination with operating and accident loads (tornado and accident loads are not combined). This affords two levels of resistance to wind and seismic loads, i.e. the plant can remain operational during and following the operating-basis earthquake or the design wind, and shall be able to safely shut down following the design-basis tornado or safe-shutdown earthquake.

● 7.4-6 Schnobrich, W. C. and Gupta, A. K., **Design of cooling towers to resist earthquakes,** *Proceedings of U.S.-Japan Seminar on Earthquake Engineering Research with Emphasis on Lifeline Systems,* Japan Society for the Promotion of Earthquake Engineering, Tokyo, Nov. 1976, 365-379.

The hyperbolic cooling tower currently used in conjunction with many power plants represents a sizeable structure in terms of both physical dimensions and cost. The structural design of such towers is normally governed

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by the specified wind loading because the distribution of that loading over the structure tends to activate a load-resisting capability from only part of the tower; that is, the leeward half of the tower does not significantly help the windward half in resisting the cantilever or overturning moment effect of the load. This is evident from bending solution calculations for the inplane or membrane forces in the shell.

If the tower is located in a region susceptible to strong-motion earthquakes, however, the earthquake loading may control. Several papers have discussed the concepts of an analysis of the tower response to an earthquake. Focus has been directed toward the response of the shell. Some of the published design values, however, centered on spectra which were transferred from a building reference frame. This resulted in design stress resultants substantially below those computed for the wind conditions. The conclusion was, therefore, frequently made that wind loading will govern the design even for towers in strong-motion areas. With a closer investigation of the earthquake problem, the dismissal of the problem appears premature. The support columns represent vulnerable members to the earthquake. Since severe cracking in the shell would not destroy the functioning of the tower, design criteria which allow deformations well into the nonlinear range seem justified. However, care must be exercised with regard to the columns.

● 7.4-7 Shibata, H., **Some comments on the seismic loading condition and the design criteria of nuclear vessels, pipings, and other equipment,** *International Seminar on Extreme Load Conditions and Limit Analysis Procedures for Structural Reactor Safeguards and Containment Structures,* U 5/1, 8. (For a full bibliographic citation, see Abstract No. 1.2-2.)

The Japan Electric Assn. published guidelines for the aseismic design of nuclear power plants five years ago. In this guideline, a new criteria of allowable stresses for aseismic design was presented. This new criteria was based on the philosophy of the ASME Code Section III and was a completely different system from the traditional criteria used in Japan. These criteria have been under review for five years.

The author has been engaged in studying the reliability problem of response analysis for the aseismic design of nuclear equipment. The main problem is, how should the potential maximum earthquake be decided? The coefficient of 1.5 has been employed to multiply the design basis earthquake to define the potential maximum earthquake in acceleration. Another question is whether Class A items include those whose failures might cause a fatal nuclear accident such as a pressure vessel and part of a primary coolant system.

A nuclear power plant designed using a design-basis earthquake may bring an emergency condition to the plant. A jet force caused by breaking of the main-coolant piping acts on the wall of a containment vessel for very short duration compared to the duration of the normal accident load. So the question arises whether or not the combination of the loads caused by a jet force and the earthquake load should be considered. How is the conclusion checked by a probabilistic assessment? A three-year project of experimental study on the behavior of pressurized pipings under excessive stress caused by earthquake loadings has been performed. The summary of this project was reported at 3rd SMiRT. But the problem of the elasto-plastic response analysis of the piping system still remains unsolved. This was also discussed in Session K at 3rd SMiRT.

● 7.4-8 Muller-Dietsche, W., Jehlicka, P. and Steinhilber, H., **Objectives of seismic tests in the HDR safeguards research program** (in German), *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(b), Paper K 8/5, 11. (For a full bibliographic citation, see Abstract No. 1.2-5.)

The studies being performed on the Heissdampfreaktor (HDR) are intended to verify and, if necessary, optimize with respect to earthquake effects the methods of computation for the dynamic analysis of reactor pressure vessel, piping systems, soil equipment, and fluids. The program is divided into phase I "low excitation level," and phase II "high excitation level." Phase I is characterized by the fact that linear vibration behavior is investigated experimentally with the stresses in the structures largely remaining in the elastic range. The results of the experimental studies of phase I are available for comparison with the results of theoretical studies carried out by means of linear models. The linear computer models used in the theoretical investigations of phase I represent the multitude of variants of the present status of structural dynamic techniques now customarily applied in the licensing procedure. The individual studies have been planned so that one substructure each will be treated by different discretizations. The assessment of the boundary conditions and the influence of the mechanical-mathematical techniques are included.

The experimental studies were carried out at a low excitation level at the HDR in 1975. The test primarily serves for comparison between the calculated and the measured results. The scope, type, and local range of experimental investigations are determined primarily by objectives pertaining to the theoretical studies. In principle, three types of information are made available: accurate indication of the positions of the locations of measurement and excitation; information about the excitation; and measured results for comparison between measurement and calculation.

Phase II relates mainly to the assessment of the nonlinear vibration behavior of the structures. The experimental studies are planned to go beyond the range of purely elastic material behavior. For this reason, it may be necessary in these theoretical studies to use computer models, or to develop new ones, which allow the treatment of nonlinear material behavior and nonlinear geometric effects. This becomes particularly important if nonlinear effects have such significant consequences that an approximate assessment of the vibration behavior by linear methods will no longer be meaningful.

The verification of methods and dynamic models by detailed comparison of calculated and experimental results is directed to the information about reliability of existing structural dynamic techniques and methods of measurement; the quantification of the influence of interactions between the soil and the containment structure and between the reactor pressure vessel or other circuit components and the pipes; the assignment of the techniques to their optimum fields of application; the quantification of the influence on the vibration behavior of the parameters of materials, soil and design structure; improved methods, especially for the assessment of nonlinear effects.

It is expected at the end of this research program that judgments can be made on the most accurate and most cost-effective methods to be used in seismic analysis and design.

● 7.4-9 Fischer, E. G., **Seismic qualification of systems, structures, equipment, and components,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(b), Paper K 8/1, 9. (For a full bibliographic citation, see Abstract No. 1.2-5.)

The purpose of this paper is to give an overview of the various qualification procedures available to the vendors of nuclear power plants and equipment to achieve NRC (Nuclear Regulatory Commission) plant licensing and overall guaranteed safe operation. These procedures usually involve computer-aided analyses for large systems and structures, but tend toward shaking table tests for small equipment and components. For example, analysis is used to investigate primary piping loops where calculated stresses are the criteria for failure. On the other hand, testing of sensitive devices becomes necessary where the avoidance of conceivable malfunctions is the criterion for safe performance.

The seismic qualification of the safety systems for an entire power plant begins with an evaluation of the earthquake environment at the plant site. This consists of test borings and vibration measurements to establish local soil conditions and to determine how stress waves can be transmitted to the site. Continuing the emphasis on dynamics, a modal analysis of the soil interaction with the subsequent foundation and buildings is made for specific earthquake occurrences. Vibration field tests are then useful to validate the computer modeling. Design response spectra are made available not only for the building foundations, but also for the building floor areas that support safety-related equipment and components. Finally, the shaking tables used to qualify devices mounted in cabinets must simulate earthquake dynamic effects in terms of reproducing seismic response spectra.

The dynamic analysis and testing required for seismic qualification can be covered in a practical manner by reference to several pertinent regulatory guides and standards. They have been issued by the NRC on specific subjects, but often represent a consensus of more general standards prepared by professional societies. These documents cover such diverse subjects as (a) reactor site criteria, (b) seismic design limits and loading combinations, (c) system damping values, and (d) recommended vibration test practices. Engineers have had to learn that their final designs are subject to review by the NRC, university advisors and environmentalists. Eventually, they must account for the legitimate concerns for safety raised by such groups, and this goal can be approached by learning what the guides and standards have to say.

The available documents on seismic qualification procedures, including the regulatory guides, are subject to change. In particular, some of the engineering society standards amount to discussions of recommended practices that represent the state of the art. From the equipment vendors' point of view, the early seismic specifications were both tentative and exploratory. However, they have proliferated in the form of specific regulations and standards that have become the basis for contractual obligations. There has been a continuing escalation of safety-related seismic requirements. This present overview indicates a need for R & D work and reexamination of published documents to counterbalance unwarranted conservatism.

● 7.4-10 Katayama, I., **Some considerations on the design earthquakes and response spectra for the aseismic design for Ikata nuclear power station,** Japan NUS Co., Tokyo, Aug. 1977, 29.

The procedure for determining design earthquakes for the aseismic design of the Ikata nuclear power station is described. The design response spectrum method was used. A dynamic model of the nuclear reactor containment facility is shown. The model consists of three mutually independent basic systems, one each for the concrete outer shielding wall, the inner concrete structure, and the steel containment vessel, which are completely fixed at their thick foundation mat.

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The results of an analysis of the eigenvalues of each of the basic models and of a typical soil-structure interaction model (combined model) are shown. In the analysis of the combined models, evaluations have been made for the cases of uniform stress and rigid plate stress distributions at the base plane of the foundation contact with foundation rock in regard to the evaluation of the sway and rocking spring, with 5% and 10% damping ratio for the swaying and rocking mode of vibration. For a series of these combined systems, the maximum values of the time-dependent responses, due to the typical design earthquake accelerograms and the reference earthquake accelerograms, were compared with the response values obtained through the design model with fixed ends to which design response spectra had been applied. The response values obtained by connecting the design models and the design response spectra envelop the response values obtained from the more detailed dynamic analysis with proper margin. For the containment vessel, a thin-walled shell model was used in the design instead of a lumped-mass model, but the lumped-mass model used in the study for other parts of the structure has been determined after confirming that the eigenvalues and response values (cross-sectional forces) are virtually the same for these models.

● **7.4-11** Jolivet, F. and Richli, M., **Aseismic foundation system for nuclear power stations,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(b), Paper K 9/2, 12. (For a full bibliographic citation, see Abstract No. 1.2-5.)

The aseismic foundation system, as described in this paper, is a new development, which makes it possible to build standard nuclear power stations in areas exposed to strong earthquakes. By adopting proven engineering concepts in design and construction of components, great advantages are achieved in safety and reliability, efficiency, design schedule, and cost. The principle consists of interposing a device between the buildings of the nuclear island and the foundation soil which limits the value of the horizontal acceleration in the nuclear island buildings. The nuclear island foundation consists of a double raft with aseismic bearings interposed between the upper and lower rafts. These bearings are placed on top of short concrete pedestals constructed on the lower raft. The space provided between the two rafts allows access to the bearings. The bearing consists of two parts: a block of reinforced elastomer, and a set of friction plates such as stainless steel and bronze with lead.

● **7.4-12** Hsieh, T.-M. and Okrent, D., **On design errors and system degradation in seismic safety,** *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology,* Vol. K(b), Paper K 9/4, 12. (For a full bibliographic citation, see Abstract No. 1.2-5.)

Recent probabilistic studies on seismic aspects of reactor safety have not specifically included the possible influence of design errors or of some forms of system degradation during plant lifetime. In this paper, a rough approach is outlined for the inclusion of possible design errors in such probabilistic estimates. Also, the possible influence of existing flaws on piping system failure under severe seismic shaking is examined. In the U.S. Nuclear Regulatory Commission's (NRC) 1975 report, *An Assessment of Accident Risks in U.S. Commercial Nuclear Power Plants,* it was concluded that the earthquake risk was negligibly small compared to other reactor accident risks. One important factor was the assumption that the simultaneous failure of two systems is required to produce a core melt accident. Based on Newmark's work, the probability of damage to a safety system with a builtin safety factor of 20 is of the order of 1.5×10^{-3}. The joint failure probability of two safety systems, using log normal mean, would be of the order of 10^{-5}. When this damage probability is multiplied by the estimated earthquake probability, the probability of core melt per reactor year, due to a design basis earthquake, would be of the order of 10^{-7} to 10^{-8}, and thus the above conclusion was reached. In estimating the conservatism in the nuclear reactor facility seismic design, Newmark suggested a safety factor of 20 for the safe shutdown earthquake (SSE) which resulted from the conservatism in the predicted free field ground motion and in the predicted structural responses.

In this paper, an estimate of the possible number and influence of seismic-related design errors is obtained by examining the historic record of such errors for a specific reactor and assuming that, with inclusion of a factor to represent a learning curve, this record would be representative of other reactors. The historic errors were divided into three categories: major, considerable and moderate. About twenty of each of the first two categories were found. The range of undiscovered seismic design errors was estimated, and Newmark's safety factor was reduced proportionately depending on the seriousness of the design error. It was assumed that at least ten failure paths exist, rather than the single path assumed in the NRC report; also, it was estimated that for some failure paths (e.g., liquefaction) the original safety margin would be much less than 20. Also, a closely coupled failure in redundant systems from seismic events was assumed. Using the same relationship between recurrence frequency and acceleration as used in the NRC report, the probability of severe core damage from seismic events was then estimated to be between 8×10^{-5} and 8×10^{-4} per reactor year. While the absolute values obtained are subject to large error and uncertainty, they are substantially larger than the values obtained in the NRC report. Further study is suggested.

The number of cycles to failure of a pipe undergoing about 100 plastic cycles due to a severe earthquake was related to original flow size, using a crude method based on measurements by Cherepanov. As an example of the results, it was found that for flaws initially about 1 in. in size, fifty cycles having about 1% plastic strain might cause failure.

● 7.4-13 Pal, N., Dalcher, A. W. and Gluck, R., **Preliminary seismic design of dynamically coupled structural systems**, *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology*, Vol. K(b), Paper K 6/17, 16. (For a full bibliographic citation, see Abstract No. 1.2-5.)

Recognizing that most structural systems, especially those used in nuclear design applications, are usually dynamically coupled, a technique for preliminary seismic design of such large systems using the design earthquake response spectrum method has been suggested in this paper. The technique consists of representing a dynamically coupled structural system by an assemblage of two or more dynamically viable subsystems: (a) primary subsystem which is of primary interest and (b) secondary subsystem or subsystems which are of interest only in their effects on the primary subsystem. The primary subsystem is modeled in detail for seismic analysis. To include the dynamic interaction effects between the primary and secondary subsystem or subsystems, the primary seismic model is modified by adding discrete generalized spring mass parameters representing the secondary subsystem along juncture degrees-of-freedom in three orthogonal directions. Both translation and rotation parameters are included. The parameters representing the secondary subsystems are derived from separate dynamic analyses in which the generalized coordinates are selected at the juncture degrees-of-freedom and initially no interconnection is assumed between the primary and secondary subsystems. Only a few dominant natural modes of each secondary subsystem are used to represent the dynamic interaction effects. Any difference between damping properties of the primary and secondary subsystems is recognized by pre-assigning mass or stiffness proportional modal damping ratios to the primary and secondary constituents of the primary subsystem modified seismic model. This model is then analyzed using design earthquake response spectra which are appropriately scaled to include effects of different modal participation factors of dominant modes of the primary and secondary subsystems.

Example analyses are presented to demonstrate application of the suggested technique in nuclear design applications. The extent of error indicated by these analyses appears compatible with accuracy usually desired in preliminary seismic design of components, piping, and support structural systems. Advantages of the proposed technique are simplicity in design applications and substantial reduction in computational effort in the preliminary analysis or design of especially large practical size systems. Furthermore, the technique is flexible enough to permit, as is customary, the conducting of seismic analysis of a nuclear power plant by dividing systems according to design classifications and different design group responsibilities.

● 7.4-14 Newmark, N. M., **Inelastic design of nuclear reactor structures and its implications on design of critical equipment**, *Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology*, Vol. K(a), Paper K 4/1, 10. (For a full bibliographic citation, see Abstract No. 1.2-5.)

Based on research and experience in earthquake engineering, criteria applicable to the seismic design of nuclear power plant facilities are described, including selection of the earthquake hazard for design, earthquake ground motions, soil-structure interaction, damping and energy absorption in structural and equipment response, methods of dynamic analysis for design, and design procedures for structures and equipment. These criteria differ somewhat from the design criteria of the U.S. Atomic Energy Commission and its successor, the U.S. Nuclear Regulatory Commission. The major differences include, among other things, the possibility of designing into the nonlinear or inelastic range. In considering the response of a nuclear reactor structure to seismic motions, one must take account of the implications of various levels of damage, short of impairment of safety, and definitely short of collapse, of the structure. Some structural elements of nuclear power plants must perforce remain elastic or nearly elastic in order to perform their allocated safety function. However, in many instances, a purely linear elastic analysis may be unreasonably conservative when one considers that, even up to the near yield point range, there are nonlinearities of sufficient amount to reduce required design levels considerably. Moreover, limited yielding of a structure may reduce the response of equipment located in the structure below those levels of response that would be excited were the structure to remain elastic. Energy absorption in the inelastic range is most conveniently treated by use of the so-called ductility factor, introduced by the author for design of structures and equipment to resist explosion and blast forces. In general, for small excursions into the inelastic range, especially when the latter can be approximated by an elastoplastic resistance curve, the design response spectrum is decreased by a simply determined factor that is related to the ductility factor.

Many important parts of equipment of a nuclear power plant facility are attached to the principal parts of the structure and respond in a manner determined by the structural response as well as by the general ground motion to which the structure is subjected. This matter involves some difficulty in analysis, but appropriate calculational techniques and design methods are available. A suitable design simplification is one in which the response of the attachment is related to the modal responses of the structure. This equipment response is affected by the relative mass of the attachment and the structure. Where this relative mass is infinitesimal, the response is affected primarily by the damping of the structure and the equipment, but as the relative mass becomes finite, even though small, an additional effective relative damping is involved which is related to the effective generalized mass ratio of the equipment to the structure. Under some circumstances particular elements of the equipment may be permitted to go into the nonlinear range also. However, it is always essential to provide ample margins of safety in design to provide for possible uncertainties in input motions and response characteristics. Such uncertainties may include, among other things, nonuniform torsional and tilting motions of the ground as well as the usually considered horizontal and vertical ground motions. Nevertheless, it is not reasonable in design to take every input and resistance parameter at an enveloping limit; the actual benefits of doing so may in fact be negative.

● **7.4-15** ACI Committee 349, **Proposed revisions to Section 9.5 of code requirements for nuclear safety related concrete structures (ACI 349-76),** *Journal of the American Concrete Institute,* 74, 9, Title No. 74-43, Sept. 1977, 452–456.

The report presents a number of revisions to the ACI Code Requirements for Nuclear Safety Related Concrete Structures (ACI 349-76). These revisions concern the anticipation and determination of deflections when nonstructural elements (such as equipment) are attached to the structure and for loading computations accounting for normal, severe, extreme, or abnormal environments.

● **7.4-16** Kanga, B. K., **SNUPPS: an experience in standardization of nuclear power plants,** *Civil Engineering, ASCE,* 47, 7, July 1977, 47–50.

The standardized Nuclear Unit Power Plant System (SNUPPS) is the first successful attempt by a group of utilities at standardization of nuclear power plants in the United States. It is a project for building five identical units, in four different states, by a group of five utilities. Design engineering and procurement of equipment and materials are 60% complete, and construction work has started at two of the sites. This article outlines the approaches and methods used in standardization.

● **7.4-17** ACI Committee 349, **Code requirements for nuclear safety related concrete structures (ACI 349-76),** *Journal of the American Concrete Institute,* 74, 2, Title No. 74-4, Feb. 1977, 53–58.

The proposed standard covers the proper design and construction of concrete structures which form part of a nuclear plant and which have nuclear safety-related functions, but does not cover concrete reactor vessels and concrete containment structures (as defined by ACI Committee 359). The structures covered by the proposed code include concrete structures inside and outside the containment system.

Appendix A, Thermal Considerations, proposed as an addition to the standard, presents minimum requirements for members subjected to time-dependent and position-dependent temperature variations occurring during normal operation and accident conditions. However, it does not address temperature and shrinkage reinforcement or temperature requirements during curing. The proposed Appendix C presents special provisions for design of structural elements subjected to time-dependent loads due to collision of masses (impact) and time-dependent loads not associated with the collision of solid masses (impulse).

● **7.4-18** Dong, R. G. and Tokarz, F. J., **Seismic effects on modularized spent fuel storage facilities,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2652–2658.

Large pools are being considered to provide additional storage capacity for spent fuel from nuclear power plants. Because of the large size, modularization of the pool into cells would enhance operational safety and convenience in terms of isolating trouble spots and performing localized cleanup. However, the effects of modularization on earthquake resistance was not clear. An investigation of these effects was made and the results are presented. The findings indicate that modularization may or may not be advantageous in terms of structural loads, depending on the pool configuration and installation.

● **7.4-19** Kamil, H. and Bertero, V. V., **A nonlinear seismic design procedure for nuclear facilities,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2627–2632.

A procedure is presented for an efficient, economic, and reliable earthquake-resistant design for nuclear facilities. The main emphasis is on frame-type structures, such as auxiliary or turbine buildings of a nuclear power plant complex. Reactor containment structures are also discussed. The design procedure for frame-type structures consists of a step-by-step nonlinear, inelastic, optimum design approach, including proposed preliminary and final design techniques. Probabilistic methods for determining the reliability of designs so obtained are also reviewed.

● **7.4-20** Patwardhan, A. S., Cluff, L. S. and Tocher, D., **Approach to probability evaluation of design earthquakes for nuclear plants,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2587–2593.

Two theoretical earthquakes provide a basis for seismic design of nuclear power plants: a safe shutdown earthquake and an operating basis earthquake. The two

● *See* ***Preface****, page v, for availability of publications marked with dot.*

approaches commonly utilized to define the characteristics of these earthquakes—the capable fault approach and the tectonic province approach—are based on predominantly deterministic procedures involving uncertainties in the selection of parameters and relationships. These uncertainties can result in significant variations in the probabilities of occurrence of each characteristic. This paper identifies the areas of uncertainty and suggests a procedure for assessing the probability of two primary characteristics of design earthquakes: magnitude or intensity, and peak accelerations.

7.4-21 Buttemer, D. R. and Torri, A., **Safety features of the gas-cooled fast breeder reactor,** *Nuclear Engineering and Design,* **40,** *1,* Jan. 1977, 43–54.

The safety features of the gas-cooled fast breeder reactor (GCFR) are described in the context of the 300-MW(e) demonstration plant design. They are of two general types, inherent and design-related. The inherent features are principally associated with the helium coolant and the nuclear coefficients. Design-related features influencing safety include shutdown systems, residual heat removal systems, method of core support, and the prestressed concrete reactor vessel (PCRV). This paper discusses the safety-related aspects of each of these. Recently completed residual heat removal system reliability studies are also discussed. The probability of residual heat removal system failure in the GCFR is found to be lower than that described for light water reactors. The safety characteristics of larger plants are examined, and increases in size are found to improve GCFR safety margins.

● 7.4-22 **Seismic design verification of LMFBR structures,** *SAN/1011-117,* Agbabian Assoc., El Segundo, California, July 1, 1977, 111.

Code of Federal Regulations 10 CFR 50, presented in Appendix B, provides quality assurance requirements for nuclear power plants and fuel reprocessing plants. It stipulates that design verification can be provided by performance of design reviews and use of alternate or simplified calculation methods, or by the performance of a qualification test that demonstrates adequacy of performance under the most adverse design conditions. This report provides an assessment of the seismic design verification procedures currently used for nuclear power plant structures, a comparison of dynamic test methods available, and conclusions and recommendations for future LMFBR structures. The results of this investigation indicate that it is possible to provide a seismic environment adequate for a qualification test of nuclear power plant structures. However, the expense of the test and the damage liability resulting to other nearby conventional structures make such tests impractical and unfeasible. As a result, seismic design verification of nuclear power plant structures is currently being provided only by design reviews and alternate calculations. The study further indicates that these procedures can provide results the reliability of which is comparable to that of qualification testing only when it is certain that the design assumptions are all correct and have been previously verified.

A comparison of experimental determinations of mode frequencies and damping of structures using ambient transient, steady state mechanical oscillator, underground explosive, and earthquake excitation concludes that ambient transient tests will not provide useful information on the response of massive LMFBR structures. Also, only in special cases will meaningful information result from steady state mechanical oscillator tests. Additional test information on the response of nuclear power plant structures in strong earthquake ground motion environments is needed to verify design assumptions. Two approaches are recommended. First, tests using underground chemical explosives are recommended if appropriate decommissioned plants located where the test environment will not cause damage to other nearby structures can be found. Second, a study is recommended of experimental, demonstration, and commercial reactors in areas having high seismic activity in order to select appropriate facilities for additional strong-motion instrumentation, since instrumentation currently provided for nuclear power plants does not permit a complete interpretation of the dynamic response of the structure.

7.5 Design and Construction of Miscellaneous Structures

● 7.5-1 ACI Committee 350, **Revisions to concrete sanitary engineering structures,** *Journal of the American Concrete Institute,* 74, *6,* Title No. 74-26, June 1977, 235–237.

Revisions to the 1971 report of the committee are presented. One revision brings the section on recommended stresses into line with ACI 318-71; the other adds information on design of tanks to resist seismic loads.

● 7.5-2 Grant, A., **Pasco-Kennewick Bridge–the longest cable-stayed bridge in North America,** *Civil Engineering, ASCE,* 47, *8,* Aug. 1977, 62–66.

This 2503 ft long prestressed concrete, cable-stayed structure, built in an earthquake zone, is the second of its kind to be constructed in the U.S. It has a continuous concrete girder, supported from steel cables over 1794 ft of its length, and from piers for the remainder of the length at both approach ends. The cables are anchored at the tops of two pairs of 250 ft concrete towers, and along the edges of the bridge at 27 ft centers. When completed in mid-1978, it will cross the Columbia River, Washington. The article describes the structure, design planning, and erection in detail and discusses the bridge's wind and earthquake stability.

● *See* ***Preface****, page v, for availability of publications marked with dot.*

● 7.5-3 Foss, J. W., **Protecting communications equipment against earthquakes,** *Proceedings of U.S.-Japan Seminar on Earthquake Engineering Research with Emphasis on Lifeline Systems,* Japan Society for the Promotion of Earthquake Engineering, Tokyo, Nov. 1976, 237–255.

Building-housed equipment, if not properly braced during an earthquake, forms one of the most vulnerable segments of a communications system. The building floors that support equipment will undergo seismic motions with accelerations up to 1 g in major earthquake zones. Responding to these motions, equipment whose vibration frequencies lie below 10 Hz will sustain accelerations up to about 5 g. Thus, strength of the equipment and that of its fastening or bracing to the building must be equal to the forces induced by such accelerations.

Resistance of equipment to seismic shaking is enhanced by designing for high stiffness, frequency, damping and ductility, and low center of gravity in a symmetric, torsion-resistant structural framework. Subcomponents must be securely attached or latched into place to prevent being dislodged from their supports. Damage to cabling, ducts, or piping between adjacent pieces of equipment because of unequal or out-of-phase motions is prevented by minimizing or eliminating the relative response, as for example, through the use of tie-struts.

Earthquake protection of communications equipment is generally cost effective in major seismic risk areas. The feasibility of similar protection in low-risk or marginal earthquake areas may be determined from cost/loss analyses which consider the importance of the facility as well as plant expenditures. Applique bracing that is graded in strength to match the zonal and/or importance classifications may provide low-cost options for such areas.

● 7.5-4 Peck, R. B., **Pitfalls of overconservatism in geotechnical engineering,** *Civil Engineering, ASCE,* 47, *2,* Feb. 1977, 62–66.

The author presents three main reasons why geotechnical engineers may be overly conservative when designing structures. The discussion is illustrated through case histories of nuclear power plants, tracking radar systems, underpinning for subways, and the seismic stability of arctic pipelines.

● 7.5-5 Rigoni, Jr., D. L., **Seattle's freeway park,** *Civil Engineering, ASCE,* 47, *5,* May 1977, 47–49.

The Seattle Freeway Park connects two downtown areas severed by a major highway. One design problem centered on potential earthquakes. Existing retaining walls could not withstand superimposed earthquake loads. Design criteria provided for new piers in the freeway median to carry the load, and girder ends at the walls were set on steel rollers to allow the girders to move back and forth during an earthquake. A drawing of the construction plan is presented.

● 7.5-6 ACI-ASCE Committee 334, **Reinforced concrete cooling tower shells—practice and commentary,** *Journal of the American Concrete Institute,* 74, *1,* Title No. 74-2, Jan. 1977, 22–31.

This document is a report on the design and construction of reinforced concrete cooling tower shells. In general, this report is addressed to towers 300 ft or greater in height having the geometric form of a hyperboloid of revolution. The specific intent is to provide the design profession with guidelines reflecting the best state of the art, as interpreted by the committee. In order to facilitate continued reinterpretation and updating of this report, ample flexibility is provided to modify or replace specific provisions and formulas, as stated, with substantiated new information.

● 7.5-7 Kubo, K. and Katayama, T., **A simple method for evaluating seismic safety of existing bridge structures,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1951–1956.

A statistical analysis of the effects of seismic damage to various characteristics of bridges was made. Based on the results obtained, a simple criteria was proposed for the preliminary evaluation of seismic safety (or vulnerability) of existing bridge structures with particular emphasis on the collapse of superstructures. The adequacy of the criteria was examined by using past damage data of actual bridges.

● 7.5-8 Nojiri, Y. *et al.,* **Earthquake resistant design of prestressed concrete cable stayed bridge,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1957–1962.

The earthquake response of a prestressed concrete cable-stayed bridge may be characterized by the effect of coupling between the vibration of a tower and a girder. In this paper, the response characteristics are discussed using the response analysis of an actual highway bridge. It is noted that a peculiar interaction phenomenon between a tower and a girder probably occurs in the transverse response and rarely occurs in the longitudinal response. To verify this phenomenon, it is proposed that such a bridge be designed using the procedure described.

● 7.5-9 Pant, B. and Saxena, P. C., **Basis of design decisions in strengthening a dam subjected to earthquake damage—a case history,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2541–2548.

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The Koyna Dam which was subjected to a severe earthquake in Dec. 1967 had to be strengthened in two stages: a temporary stage in which prestressing and epoxy grouting of the cracked portion of the nonoverflow monolith was resorted to, and a permanent stage in which the blocks were backed with concrete masses. The basic considerations for deciding what restorative measures to take are briefly reviewed. It is shown that a purely analytical approach had several limitations and that the decisions had to be based on observations of the existing dam, constructional convenience, and codes of practice in different countries.

● **7.5-10** Fujiwara, T., **New automatic train stopping system during earthquake,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2943–2948.

A model of a new automatic train stopping system has been developed. This system is based on the following two ideas. One is that the seismic detection locations are set away from the railway line toward the expected foci. The other is prediction of the amplitude of the subsequent maximum motion from waveforms of a preliminary tremor. The tripartite network observing system with an online minicomputer will make it possible to estimate approximately the focus and the magnitude in an early stage.

● **7.5-11** Muto, K., Nagata, M. and Fukuzawa, E., **Earthquake resistant installation device of computers,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3317–3322.

A new earthquake-resistant installation device for computers has been developed in order to prevent sliding or overturning which can lead to the loss of their functions. This device fastens computers to supports which are anchored tightly to a concrete slab passing through a free-access floor. A design calculation method based on floor response spectra also has been proposed. These methods were used for the installation of computers in the International Telecommunications Center building (32 stories high) in Tokyo. The safety of the computers against strong earthquakes was verified by vibration tests on a shaking table using floor-response time-histories obtained from the dynamic design analyses of the building.

● **7.5-12** Liu, S. C., **Earthquake protection of communications facilities,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3304–3309.

The continued reliable operation of telephone equipment in earthquake-prone areas depends on the equipment's ability to survive earthquakes of a realistic magnitude. This paper describes procedures for developing regional earthquake protection practices for electronic, electromechanical, and power systems based on cost and reliability. The following elements are included in this procedure: characterization of the earthquake environment, equipment vulnerability, and the cost of protection as opposed to potential losses.

● **7.5-13** Benuska, K. L., Aroni, S. and Schroll, W., **Elevator earthquake safety control,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3334–3340.

The Public Health Building elevators at the Univ. of California, Los Angeles, suffered severe damage from counterweight collision with the cars during the San Fernando earthquake of Feb. 9, 1971. The elevators are moderate speed, 500 fpm, gearless machines. They have been returned to service with the addition of an earthquake safety control. This is an active system which utilizes a vertical seismic trigger to initiate a controlled shutdown sequence. The system was tested by a minor earthquake on Nov. 16, 1973.

● **7.5-14** Clough, D. P., **Experimental evaluation of seismic design methods for broad cylindrical tanks,** *UCB/EERC-77/10,* Earthquake Engineering Research Center, Univ. of California, Berkeley, May 1977, 281. (NTIS Accession No. PB 272 280)

Earthquake damage to ground-supported cylindrical liquid storage tanks during recent years demonstrates the need for better understanding of the seismic behavior of these structures and improvement in seismic design procedures. Analytical procedures epitomizing the current seismic design approach for cylindrical tanks are presented in detail, and their application in a typical design situation is illustrated. Records of tanks damaged in four earthquakes are examined, and consistent with the predictions of current seismic design methods, a relatively high earthquake vulnerability is found in tanks with height greater than radius.

Methods, models, and facilities utilized in an experimental investigation of the seismic behavior of tanks are described. A practical distinction is made between tall tanks, with height greater than radius, and broad tanks, with height less than, or equal to, radius. Principal experimental results from the first phase of this research, covering a broad tank model, are presented in the form of a critical evaluation of current seismic design practice. While typical design procedures assume that the critical seismic response mechanism in ground-supported tanks is a quasi-static rocking motion, without distortion of the

● *See **Preface,** page v, for availability of publications marked with dot.*

circular cross-section, the principal response of the broad model was high-frequency section-distortion vibration with dominant radial displacements in cos(3Θ) and cos(4Θ) circumferential distributions. It is concluded that present seismic design methods neglect the most important aspects of seismic response in broad tanks. Results from the second phase of the research, covering a tall model, will be presented in a future report.

● **7.5-15** Wang, L. R.-L. and O'Rourke, M. J., **State of the art of buried lifeline earthquake engineering,** *SVBDUPS Project Report No. 1,* Dept. of Civil Engineering, Rensselaer Polytechnic Inst., Troy, New York, 1977, 53.

This report provides state-of-the-art information on buried water/sewer lifeline earthquake engineering as a basis for future research. It contains a survey of records of pipeline failure as a result of earthquakes, current design practices, and current research activities. Observations from past earthquakes of the failure mechanism of buried pipelines yield more qualitative than quantitative information. This is because many parameters affect failure behavior, and, as yet, no earthquake damage investigation has covered all the possible parameters. No formal provision has been set by any code organization for the design of buried pipelines for earthquake loads. However, certain structural details and/or "fail safe" structural systems have been used to reduce the effects of earthquakes.

A survey of recently published literature in the areas of earthquake engineering and structural dynamics indicates that there is no single analytical model capable of completely evaluating an underground lifeline system during an earthquake. To facilitate the study of seismic effects, hazards, analysis, and the design of buried pipelines, the National Science Foundation has awarded three grants, including one to the authors of the present report, to research this topic independently but cooperatively. A description of each project is found in Appendix 1. This report also includes the methodology the authors intend to use in the study of seismic vulnerability, behavior, and design of underground piping systems.

● **7.5-16** Baron, F. and Hamati, R. E., **Effects of non-uniform seismic disturbances on the Dumbarton Bridge replacement structure,** *EERC 76-19,* Earthquake Engineering Research Center, Univ. of California, Berkeley, July 1976, 359. (NTIS Accession No. PB 282 981)

This report is the third in a series that deals with the design of the Dumbarton Bridge replacement structure for resisting seismic disturbances. The replacement structure is a long multispan bridge which will be located approximately equidistant (ten miles) from the San Andreas and Hayward faults. In the report, particular reference is made to a final selection of the girder types and the articulation of the replacement structure. The final design is analyzed for various ground motions, including ground surface motions which propagate along, oblique, and transverse to the longitudinal axis of the bridge.

This third and final phase of the investigation deals with the influences of spatial distributions of ground motion of the maximum values of response of the replacement structure. Three categories of ground motion are considered: (1) uniform distributions of the S18 + design earthquake motions developed for thick-mud and deep-water sections, (2) various distributions of propagating and stationary ground surface waves, and (3) various possible patterns of ground displacements which may be caused by residual deformations of the soils along the structure. The characteristics of each category of ground motion are defined, and bounds on the maximum values of the different parameters of motion are obtained. The characteristics and bounds are both related to the properties of the two soil conditions at the site and their use in design for the articulation and horizontal dimensions of the replacement structure.

Finally, criteria are developed for the seismic design of the replacement structure. The criteria are for requirements of seismic strength to resist inertia effects and for provisions for sufficient ductility to absorb the displacements and deformations caused by uniform and propagating distributions of ground motions.

Design spectra for propagating waves are developed for both soil conditions at the site. For each soil condition, the spectrum relates maximum values of possible amplitudes with the corresponding values of wave lengths. By means of the spectra and consistent relations for propagating waves, values of maximum forces and moments in the elements of the replacement structure are obtained for various cases of propagating waves. In addition, values of maximum displacements caused by propagating waves are obtained for various nodes of the structure.

● **7.5-17** Gordon, L. A. and Zabolotnaya, V. A., **Earthquake design of reinforced concrete cooling towers,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 2019–2024.

Reinforced concrete cooling towers of thermal power stations are space structures that can be represented as shells of revolutions with heights of 150 m. The principal loads for these important structures under ordinary conditions are the dead load and the wind load. In areas of high seismicity, seismic loads should be taken into account. Numerous publications recognize the above factors within the bounds of different seismic stability theories. A method for calculating reinforced concrete cooling towers according to standards currently used in the U.S.S.R. is given.

● **7.5-18 Dams within jurisdiction of the State of California,** *17-76,* California Dept. of Water Resources, Sacramento, July 1976, 76.

This bulletin tabulates statistical data on all dams in California. There is a cross-referenced, alphabetical tabulation for reservoir names which do not correspond to dam names. Data listed include name of dam, dam number, name of owner, county in which dam is located, stream on which dam is located, type of dam, storage capacity, and drainage area. Also included are reservoir area, elevation of crest, crest length, height, distance from dam crest to permanent spillway crest, maximum operating level, crest width, dam volume, and year completed. Federal projects are listed in an appendix.

● **7.5-19** Isenberg, J. and Wright, J. P., **Survey of existing underground water pipelines with emphasis on their seismic resistance,** Weidlinger Assoc., New York, July 1, 1977, 25.

A survey of the characteristics of underground water pipeline systems which may affect their seismic performance was conducted. The main objectives of the survey were (1) to establish a data base on the types of water pipelines in current use, including materials, sizes, types of joints, depths of burial, and backfill conditions; (2) to determine perceptions of seismic risk among water utilities; and (3) to collect data on seismic performance of water pipelines. The survey was conducted by questionnaires mailed to 516 water utilities in the U.S. The group selected reflects relative state populations, but is biased somewhat toward cities with larger populations and toward western states which are seismically more active.

● **7.5-20 API recommended practice for planning, designing, and constructing fixed offshore platforms, 7th ed.,** *API RP 2A,* American Petroleum Inst., Washington, D.C., Jan. 1976, 47.

Engineering design principles and practices that have evolved during the development of offshore oil resources are presented. Consideration is given to the safety of personnel, compliance with existing regulations, and antipollution of water bodies.

● **7.5-21** Stockdale, W. K., **Seismic design methods for military facilities–preliminary recommendations,** U.S. Construction Engineering Research Lab., Champaign, Illinois, June 1976, 79. (NTIS Accession No. AD-A027 384)

This report presents preliminary recommendations for the methods of structural analysis to be used in the design of critical facilities on military installations. The recommended dynamic analysis methods are described and discussed, and examples are presented which illustrate the elastic and inelastic response spectra methods. It is recommended that dynamic methods be used in all areas where the expected ground acceleration exceeds 0.10 g.

7.5-22 Pavlov, O. V., **Earthquake engineering aspects of the construction of the Baikal-Amur main railroad line** (Inzhenerno-seismologicheskie usloviya stroitelstva Baikalo-Amurskoi zhelezodorozhnoi magistrali, in Russian), *Sovremennye issledovaniya zemnoi kory,* Irkutsk, 1975, 128–134.

Information concerning recent major earthquakes occurring near the Baikal-Amur railroad line is presented. The area considered has many unfavorable features, e.g., high seismicity, several sharply delineated climatic zones complicated by permafrost and high altitudes.

● **7.5-23** Krakovskii, M. B. and Pyatikrestovskii, K. P., **Optimal design of earthquake-resistant reinforced concrete beam space structures** (Optimalnoe proektirovanie seismostoikikh zhelezobetonnykh sterzhnevykh prostrantstvennykh konstruktsii, in Russian), *Seismostoikoe stroitelstvo, 5,* 1977, 18–22.

The possibility of applying methods of optimal design based on the theory of planned experiments to the optimization of the design of earthquake-resistant reinforced concrete space structures is investigated. An example of a water tower design is discussed. The effects of seismic and wind loads are considered and the finite element method is used in the calculations.

● **7.5-24** Yamada, Y., **Earthquake resistant design of long-span bridges,** *Proceedings of U.S.-Japan Seminar on Earthquake Engineering Research with Emphasis on Lifeline Systems,* Japan Society for the Promotion of Earthquake Engineering, Tokyo, Nov. 1976, 287–304.

This paper presents new developments in the earthquake response analysis and design of long-span suspension bridges. The earthquake-resistant design of a long-span cantilever truss bridge is also discussed.

● **7.5-25 Exxon's offshore oil platform nearly doubles water-depth record,** *Civil Engineering, ASCE,* **47,** *3,* Mar. 1977, 51–54.

In 1976, Exxon Company erected a 90-story high drilling and production platform (Hondo) in 850 ft of water near Santa Barbara, California. Design criteria for earthquakes, wave forces, storm, and fatigue are met with a frame of steel tubes up to 900 ft long and 54 in. in diameter. The Hondo platform will make possible development of oil and gas resources that were initially out of reach due to water depth. It represents the tallest offshore structure yet installed. Safety against collapse is provided for in the event of the strongest potential ground shaking, or ground shaking having a small probability of occurrence.

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● 7.5-26 Chopra, A. K. and Liaw, C. Y., **Earthquake-resistant design of intake-outlet towers,** *Proceedings of U.S.-Japan Seminar on Earthquake Engineering Research with Emphasis on Lifeline Systems,* Japan Society for the Promotion of Earthquake Engineering, Tokyo, Nov. 1976, 381–397. (Reprinted with minor changes from *Journal of the Structural Division, ASCE,* **101**, *ST7,* July 1975, 1349–1366.)

The problem of designing reinforced concrete intake-outlet towers, partially submerged in water, to withstand earthquake ground motion is discussed in general terms. It is shown that water, surrounding as well as inside, significantly affects the response to ground motion and these hydrodynamic effects should be considered in the design of towers. It is recommended that towers be designed to resist without significant yielding the ground motions which they may experience several times during their useful life. A rational method for design on this basis including the hydrodynamic effects is proposed. The ductility requirements that would be imposed on code-designed towers by the intense ground motions expected near the causative fault during high-magnitude earthquakes are evaluated; they appear to be rather large. It is recommended that forces for elastic design be increased so that the lateral displacement ductility requirements imposed by the most intense ground motion that can occur at the site would be no larger than a ductility factor of 2.

● 7.5-27 Egawa, K., **Some examples of earthquake resistant design in electric power supply system,** *Proceedings of U.S.-Japan Seminar on Earthquake Engineering Research with Emphasis on Lifeline Systems,* Japan Society for the Promotion of Earthquake Engineering, Tokyo, Nov. 1976, 223–235.

Earthquake-resistant considerations of two of the typical structures in an electric power system are described. One is the buried pipe for a steam condenser and for an underground transmission line and another is the air circuit breaker composed of porcelain pipes. Two types of flexible rubber joints are installed for the buried pipes and new criteria for the earthquake-resistant design of an air circuit breaker are determined.

● 7.5-28 Shibata, H., **Fundamental concept of aseismic design code of lifeline systems and industrial facilities,** *Proceedings of U.S.-Japan Seminar on Earthquake Engineering Research with Emphasis on Lifeline Systems,* Japan Society for the Promotion of Earthquake Engineering, Tokyo, Nov. 1976, 487–494.

This paper investigates the aseismic design code of lifeline systems and industrial facilities. The idea of a safety importance factor has been used in the design of nuclear power plants. The author introduces the idea of a functional importance factor and compares it with the safety importance factor.

● 7.5-29 Parmelee, R. A., **Organization and activities of the ASCE Technical Council on Lifeline Earthquake Engineering,** *Proceedings of U.S.-Japan Seminar on Earthquake Engineering Research with Emphasis on Lifeline Systems,* Japan Society for the Promotion of Earthquake Engineering, Tokyo, Nov. 1976, 481–485.

In July 1974, the American Society of Civil Engineers (ASCE) established the Technical Council on Lifeline Earthquake Engineering (TCLEE). The purpose of the TCLEE is to establish the means by which the civil engineering profession can undertake a comprehensive role in elevating the state of the art of lifeline earthquake engineering. The present technology is dangerously underdeveloped, no major organization is committed to the problem area, and ASCE's goals and structure correspond uniquely with the dimensions of the problem.

The objective of TCLEE is to elevate the level of engineering practice in design of lifelines to survive earthquakes. The council will develop earthquake safety provisions for lifelines similar to those developed for buildings by the Structural Engineers Association of California (SEAOC), taking into consideration the different risk potential in various geographic areas. The council will also coordinate field investigations of lifeline damage due to earthquakes. Recommendations for design criteria for lifelines will be made available to applicable code authorities, government agencies, and public utilities. The council acts as the coordinating body of the lifeline interests of the various divisions of ASCE and provides the membership with information on lifeline practice to survive earthquakes.

● 7.5-30 Hakuno, M., **Damage probability of line structures due to earthquake,** *Proceedings of U.S.-Japan Seminar on Earthquake Engineering Research with Emphasis on Lifeline Systems,* Japan Society for the Promotion of Earthquake Engineering, Tokyo, Nov. 1976, 435–439.

The author discusses the probability of damage to such long and narrow structures as railways, road and river embankments, water supply systems, and electric power supply systems.

● 7.5-31 Tamura, C., **Design of underground structures by considering ground displacement during earthquakes,** *Proceedings of U.S.-Japan Seminar on Earthquake Engineering Research with Emphasis on Lifeline Systems,* Japan Society for the Promotion of Earthquake Engineering, Tokyo, Nov. 1976, 417–433.

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In this paper, it is pointed out that in ordinary cases, the distribution of displacement of the earthquake motion of the ground surface along the axis of a structure is a fundamental factor for the design of a long underground structure. An earthquake-resistant design method for structures of this type based on response displacements of surface ground layers is presented. Since the cross section of a submerged tunnel is designed to have sufficient strength against earthquake load, the design method for the cross section is not touched upon here. Using a mathematical model described previously, it is possible to study the stability during earthquakes of a structure in the direction lateral to its axis. For long underground structures in general, seismic-resistant capacities should be investigated taking structural characteristics into consideration.

● 7.5-32 Miyajima, N. *et al.*, **An example of seismic design and earthquake response measurement of buried pipeline,** *Proceedings of U.S.-Japan Seminar on Earthquake Engineering Research with Emphasis on Lifeline Systems,* Japan Society for the Promotion of Earthquake Engineering, Tokyo, Nov. 1976, 177–196.

An underground water-conveying steel pipeline of 1,200 ml diameter has been designed against earthquakes based on the design concept recently established for petroleum pipelines in Japan. New formulas for calculation of seismic wave stresses on pipelines are proposed. These formulas are introduced by relating seismic stress to spring force and to the cohesive force of the soil acting on the pipeline. These forces are treated as a nonlinear function of relative displacement between the pipeline and the surrounding soil. Earthquake response measurement of the pipeline has been conducted since Apr. 1975. Some records obtained so far are presented, and a study investigates the rationality of the seismic design concept.

● 7.5-33 Podolny, Jr., W., Chmn., ASCE Structural Div. Task Committee on Cable-Suspended Structures, **Tentative recommendations for cable-stayed bridge structures,** *Journal of the Structural Division, ASCE,* **103**, *ST5*, Proc. Paper 12933, May 1977, 929–939.

Engineers in the United States are planning and designing cable-stayed bridges despite the paucity of design and construction data in the American technical literature. There are no American standard codes governing the design of cable-stayed bridges at this time. This report is presented as a guide to the design and construction of cable-stayed bridges and, as such, it is a recommended practice not a code or specification. This report has been developed in the hope that it may be useful until such time as a more complete specification may be presented to the profession by American specification and code writing bodies. The report presents information relative to loads and forces, design assumptions, analysis, deflections, cables, saddles and end fittings, corrosion protection, fabrication, erection, inspection, temperature, fatigue, and aerodynamics.

● 7.5-34 Podolny, Jr., W., Chmn., ASCE Structural Div. Task Committee on Cable-Suspended Structures, **Commentary on the tentative recommendations for cable-stayed bridge structures,** *Journal of the Structural Division, ASCE,* **103**, *ST5*, Proc. Paper 12934, May 1977, 941–959.

The material presented is intended to explain and substantiate the information presented in the tentative recommendations by presenting more detailed information, data, and references for the salient topics, thus presenting the rationale for the recommendations presented. Topics discussed are plans and drawings, loads and forces, design assumptions, analysis, deflections, cables, saddles and end fittings, fabrication, fatigue, and aerodynamics.

7.6 Design and Construction of Foundations, Piles and Retaining Walls

● 7.6-1 Hayashi, S., Noda, S. and Uwabe, T., **Relation between seismic coefficient and ground acceleration for gravity quaywall,** *Wind and Seismic Effects,* VI-192–VI-198. (For a full bibliographic citation, see Abstract No. 1.2-4.)

The present design standard for port and harbor structures utilizes seismic coefficients which were obtained from records of 129 gravity quaywalls in 49 ports damaged by 12 earthquakes. The maximum ground accelerations in the ports were estimated by calculating the ground response during the earthquakes with reference to the attenuation curves of the base rock acceleration based on the accelerograms in the port areas. The seismic coefficients in past earthquakes had upper values of 0.25, and this upper limit of the relation between the seismic coefficient and the maximum ground acceleration is expressed by an equation.

● 7.6-2 Tamura, K., **Fundamental study on the earthquake resistant design for caisson foundations of bridges,** *Quarterly Reports, Railway Technical Research Institute,* **18**, 3, 1977, 98–104.

In order to investigate the behavior of caisson foundations subjected to static and dynamic forces, tests were conducted on seven kinds of small-sized reinforced concrete caissons at Sakura, Japan (Sobu Line, Chiba prefecture). From these test results, a linear or bilinear relation between the static horizontal force and the displacement of a caisson and its ultimate bearing capacity are introduced. Dynamic analysis of a caisson, considering the interaction with soils around it, is developed and verified using earthquake observation data recorded at Sakura and the Chikumagawa railway bridge.

7.6-3 Martemyanov, A. I., **Effects of foundation design features on the earthquake resistance of precast panel buildings [based on data from the 1976 Kizil-Kum earthquake]** (Vliyanie konstruktivnykh reshenii fundamentov na seismostoikost krupnopanelnykh zdanii [po materialam Kyzilkumskogo zemletryasemiya 1976 g.], in Russian), *Osnovaniya, fundamenty i mekhanika gruntov*, *6*, 1977, 5-7.

Data on the consequences of the 1976 Kizil-Kum earthquake are discussed. Precast panel buildings constructed from elements manufactured in Bukhara were studied. The various soil bases were subjected to identical seismic excitation providing the ability to evaluate the effects of three types of foundations (pile, pier, and continuous footing) on the earthquake resistance of buildings. At the same time, the effects of deformations occurring as a result of earthquakes on the changes in the dynamic properties of those structures were also analyzed.

● **7.6-4** Noda, S. and Uwabe, T., **Relation between seismic coefficient and ground acceleration for gravity quaywall,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1963-1968.

Using the present design standard for port and harbor structures, the seismic coefficients corresponding to the severity of ground motions were obtained for 129 gravity quaywalls in 49 ports damaged by 12 earthquakes. The maximum ground accelerations in the ports were estimated by calculating the ground response during the earthquakes with reference to the attenuation curves of the base rock acceleration based on the accelerograms in port areas. The seismic coefficients in the earthquakes were as high as 0.25, and the upper limit of the relation between the coefficient and the maximum ground acceleration was expressed by a mathematical formula which is presented in this paper.

7.6-5 Snowdon, J. C., **Resilient mounting of machinery on platelike and modified platelike substructures,** *The Journal of the Acoustical Society of America,* **61**, *4*, Apr. 1977, 986-994.

The problem of isolating machinery vibration from platelike substructures is analyzed. Simply supported, internally damped, square, rectangular, and circular plates are considered, as are rectangular plates with rigid cross members that divide the plates into separate quadrants. The machinery is supported either by eight or by four antivibration mounts located symmetrically about the plate centers. The attachment of dynamic vibration absorbers or lumped masses to the plates at each mount location is shown to be effective in reducing the force transmitted to the plate boundaries. The dynamic absorbers are tuned to suppress transmissibility at the fundamental resonance of the plate of concern, whereas the lumped masses become effective at frequencies above this resonance where their impedance predominates that of the plate. Utilization of the circular and quadrant plates greatly reduces the number of plate resonances that contribute to the force transmitted to the plate boundaries. Further, when machinery is supported by four antivibration mounts on square and rectangular plates, the number of resonances that are excited can also be reduced significantly if the mount locations are chosen judiciously. However, the use of other mount locations can yield a large number of pronounced resonances in excess of those excited when the machinery is supported by various arrangements of eight antivibration mounts for which transmissibility does not appear to change greatly with the choice of mount location.

7.6-6 PCI Committee on Prestressed Concrete Piling, **Recommended practice for design, manufacture and installation of prestressed concrete piling,** *Journal of the Prestressed Concrete Institute,* **22**, *2*, Mar.-Apr. 1977, 20-49.

The report covers the design of prestressed concrete piling, materials and manufacture, handling and transportation, and an extensive discussion of installation including best driving practices. Under design, allowable stresses are given, typical interaction charts show their application to piles subject to bending, and sample design problems suggest design approaches. Load testing is also covered. Selected references and reference standards are included.

● **7.6-7** **Study tour: England, France, Russia, and Japan, centrifuge installations and modeling techniques in structure foundation design, 13 June-13 July 1977, Trip report,** U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, July 1977, var. pp.

It was the primary objective of this study to become familiar with centrifuge installations and modeling techniques employed in the design of foundations for structures under static and dynamic loading. In addition, attention was to be directed toward construction methods in seismic areas and the evaluation of various techniques that have been employed in this regard. To be added to these objectives is the information to be gained from the exposure to new (if not different) engineering methods, needs, and procedures.

7.7 Design and Construction of Soil and Rock Structures

● **7.7-1** Lee, K. L. and Roth, W., **Seismic stability analysis of Hawkins hydraulic fill dam,** *Journal of the Geotechnical Engineering Division, ASCE,* **103**, *GT6,* Proc. Paper 12990, June 1977, 627-644.

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This 70 ft high hydraulic fill dam was constructed as a home-made do-it-yourself project on a ranch near Hollister, California, from 1912–1931. It is in one of the most seismically active zones in the state. Following a policy set for all hydraulic fill dams, as a result of the 1971 San Fernando earthquake, a recent request from the State Div. of Dam Safety required it to be analyzed according to the latest techniques for seismic safety. The paper describes the analysis which was carried out and the basis for the conclusions that the dam should be adequately stable against the maximum credible earthquake. The major reason for this unexpected result for an old hydraulic fill dam is that it was constructed slowly in lifts at about 0.5 m to 1 m per year for 19 years, predominantly of clayey soil, and with ample time for desiccation drying and consequent strengthening of the soil during construction.

● 7.7-2 Seed, H. B. and Booker, J. R., **Stabilization of potentially liquefiable sand deposits using gravel drains,** *Journal of the Geotechnical Engineering Division, ASCE,* **103,** *GT7,* Proc. Paper 13088, July 1977, 757–768.

The installation of a drainage system offers an attractive and economical procedure for stabilizing an otherwise potentially liquefiable sand deposit. Such a procedure has already been used in one case involving the construction of stone columns in a relatively loose sand deposit, and it is being proposed for stabilization of a medium dense sand layer. The paper presents a simplified theory that provides a convenient basis for evaluating the possible effectiveness of a grain drain system in such cases. Where appropriate, additional analyses may readily be made using a computer program (LARF) based on the theory presented, but for most practical cases, it is believed that the charts presented in the paper will provide for effective stabilization of potentially liquefiable sand deposits.

● 7.7-3 Srivastava, L. S., **A method of design of rock slopes subjected to strong ground motion,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2375–2376.

The stability of a rock slope is controlled by the strength characteristics of the rock material and the degree and extent of rock defects present in the rock mass. Since discontinuity surfaces and weak zones, in general, occur as sets of inclined planes, intersecting with each other as well as with the slope surface, the stability and equilibrium of the rock volume bounded by these planes has to be evaluated for the design of the slope. This requires a three-dimensional analysis of the various forces acting on the rock volume (block) and its bounding surfaces. Data and the results of analysis of a rock slope as an example are presented. The analysis assumes that the rock material is competent and that movements occur by translation along the bounding surfaces. The procedure gives a conservative estimate of the factor of safety, and the assumptions made can be accepted, provided that the geological data are carefully examined and a judicious selection of the set of discontinuity surfaces is made to determine the shape of the rock volume likely to move. The analysis also provides data on forces to be sustained by restraining devices.

● 7.7-4 Makdisi, F. I. and Seed, H. B., **A simplified procedure for estimating earthquake-induced deformations in dams and embankments,** *UCB/EERC-77/19,* Earthquake Engineering Research Center, Univ. of California, Berkeley, Aug. 1977, 65. (NTIS Accession No. PB 276 820)

The inadequacy of the pseudo-static approach recommended by the International Committee on Large Dams to predict the behavior of low embankments during earthquakes has been clearly recognized and demonstrated. The committee pointed out the need for early revision of the method, which, in several instances, predicted a safe condition for dams which were known to have had major slides.

This report presents a simplified procedure for the seismic design of small embankments. The method is an approximate one which uses the concept, originally proposed by Newmark, for calculating permanent deformations; but it is based on an evaluation of the dynamic response of the embankment rather than on rigid body behavior. The method is applied to dams with heights in the range of 100 to 200 ft and constructed of compacted cohesive soils or very dense cohesionless soils. Design curves on the basis of analyzed cases are presented with an example problem. The method is an approximate one and involves simplifying assumptions. The design curves are averages based on a limited number of cases and should be updated as more data become available.

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8. Earthquake Effects

8.1 General

● **8.1-1** Kuribayashi, E., Tazaku, T. and Hadate, T., **Relationship between earthquake damage of existing wooden houses and seismic intensities,** *Wind and Seismic Effects,* IV-1–IV-17. (For a full bibliographic citation, see Abstract No. 1.2-4.)

This report discusses a quantitative relationship between a ratio of earthquake damage to existing wooden houses and seismic intensities. The ratio of earthquake damage of the houses is useful not only for building design criteria but for predicting the damage ratio of other structures, such as bridges, roads, and public utilities. Using disaster statistics for the Fukui earthquake (M = 7.3, focal depth = 20 km, 1948), Izuhanto-oki earthquake (M = 6.8, focal depth = 20 km, 1974), and the Ebino earthquake (M = 6.1, focal depth = 0 km, 1968), the authors analyzed the relationship between the ratio of razed houses and epicentral distances or magnitudes.

● **8.1-2** Bolt, B. A. and Hansen, R. A., **The upthrow of objects in earthquakes,** *Bulletin of the Seismological Society of America,* 67, *5,* Oct. 1977, 1415–1427.

Occurrences of vertical seismic ground accelerations greater than gravity have been inferred over the years, based mainly on reports of objects separating vertically from each other during earthquakes. Only one such instrumentally recorded acceleration has been measured. A survey is made of these reports, and the observations are divided into three dynamic categories: rigid body, rocking, and internal accelerations of systems of particles with and without elasticity. Mathematical solutions obtained for two dynamic systems resting on the ground demonstrate that upthrow does not necessarily entail vertical ground acceleration greater than gravity. The results are in agreement with an argument of N. M. Newmark but the authors extend the argument to systems of purely rigid bodies. The implications in assessing intensity are discussed.

● **8.1-3** Bresler, B., Graham, L. and Sharpe, R., **Emergency post-earthquake inspection and evaluation of damage in buildings,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2549–2553.

Recommended procedures are presented for emergency inspection and evaluation of post-earthquake damage to assess the extent of damage and to evaluate the relative safety for occupancy of buildings. Guidelines for recruiting and training competent field personnel and procedures for carrying out field operations safely and effectively are developed.

● **8.1-4** Selna, L. G., **Collapse analysis of multistory buildings,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2480–2485.

After a building is noticeably damaged in an earthquake, the earthquake engineer needs to know if the building should be repaired. A method of static incremental analysis suitable for predicting the likelihood of collapse is needed. The objectives of this paper are to: (1) develop a method for large deflection quasistatic incremental analysis of ductile reinforced concrete frames, and (2) interpret the results of static collapse studies. The frames studied are loaded incrementally with static loads simulating earthquake-like load distributions.

● **8.1-5** Anagnostopoulos, S. A. and Whitman, R. V., **On human loss prediction in buildings during earthquakes,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 671–677.

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The problem of human loss prediction in buildings during earthquakes is discussed, and a methodology for such predictions is proposed. Also a building classification scheme that accounts for performance and hazard potential necessary for this methodology is presented. Due to lack of sufficient data, several assumptions are made in an illustrative example showing the application of the method. This methodology can form the framework for future data collection to relate human losses to damage and damage to intensity.

● **8.1-6** Schumacker, B. and Whitman, R. V., **Single earthquake loss probabilities,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 665–670.

A computer program gives the probability that *n* buildings in a region will fail during a single earthquake. As input, it is necessary to specify the usual seismic risk parameters (source zones, recurrence rates, attenuation laws) plus failure probability as a function of intensity of failure for each type of building and soil condition. Z-transforms are used to combine the several binomial probability distribution functions. An example illustrates the application of the program and the effect of varying the parameters.

● **8.1-7** **A study of earthquake losses in the Salt Lake City, Utah, area,** *Open-File Report 76-89,* U.S. Geological Survey, Washington, D.C., 1976, 357.

● **8.1-8** Benedetti, D. and Vitiello, E., **Statistical modeling of seismic damages,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 888–894.

When performing a cost-benefit analysis in the field of earthquake engineering, seismic damage can be expressed as the product of the amounts of direct and indirect damages times the probability of their occurrence. By examining historical records, this paper points out the importance of indirect damage, such as disruption of production or transportation systems. A model of the connections between different functions of an urban system is given. The fault tree technique is used to derive probability of indirect damage to urban functions based on a given probability of collapse of a system or building.

● **8.1-9** Schumacker, B. and Whitman, R. V., **Models of threshold exceedance and loss computations of nonhomogeneous, spatially distributed facilities,** *Seismic Design Decision Analysis Report No. 30,* Dept. of Civil Engineering, Massachusetts Inst. of Technology, Cambridge, Mar. 1977, 71.

Loss thresholds during an earthquake are examined. A model is developed for situations in which a two-state analysis is reasonable. An approach for scenario development and methods for multistate analyses are described with applications for some of the methods for the areas of greater Boston and eastern Massachusetts.

8.2 Studies of Specific Earthquakes

● **8.2-1** Adams, R. D., **The Haicheng, China, earthquake of 4 February, 1975: The first successfully predicted major earthquake,** *Bulletin of the New Zealand National Society for Earthquake Engineering,* 9, *1,* Mar. 1976, 32–42.

The earthquake of magnitude 7.3 that occurred near the town of Haicheng in northeast China on Feb. 4, 1975, was the first major earthquake anywhere in the world known to have been predicted with enough certainty for people to have been warned, and measures taken for civil protection. These steps were successful in keeping the number of casualties low. This paper describes a visit to the affected area seven and a half months after the earthquake, and discussions with Chinese scientists about their successful prediction methods. The Chinese prediction methods employed a synthesis of many types of investigations. Long-term predictions are based upon seismicity studies; mid-term predictions, upon deformation studies; and short-term predictions, upon foreshock studies.

● **8.2-2** Zhunusov, T. Zh., Itskov, I. E. and Rotgaus, B. A., **Analysis of damage to residential precast panel and brick buildings in Bukhara following the Gazlii earthquake** (Analiz povrezhdeniya zhilykh krupnopanelnykh i kirpichnykh zdanii v Bukhare vo vremya gazliiskikh zemletryasenii, in Russian), *Seismostoikoe stroitelstvo, 3,* 1977, 58–62.

Consequences of the May 17, 1976, Gazlii earthquake were investigated. Approximately 200 residential buildings of various heights and supporting structures were inspected. A detailed description is given of the types of damage and crack formation encountered in precast panel and brick buildings.

● **8.2-3** Kilimnik, L. Sh. and Zharov, A. M., **Preliminary results of an analysis of the consequences of the January 31, 1977 earthquake in northern Tadzhikistan and the southern Kirgiz S.S.R.** (Predvaritelnye rezultaty analiza posledstvii zemletryaseniya v severnom Tadzhikistane i yuzhnoi Kirgizii 31 yanvarya 1977 g., in Russian), *Seismostoikoe stroitelstvo, 4,* 1977, 55–60.

A major earthquake occurred in the southern part of the Kirgiz S.S.R., and northern Tadzhikistan on Jan. 31, 1977. According to preliminary data, the magnitude of the

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earthquake was 6.4 and its focal depth was estimated at 25–30 km. The length of the epicentral zone was approximately 20–30 km. Residual deformations caused severed roads and trenches and flooding of ground water. In this paper, the damage is surveyed and the performance of recently constructed aseismic structures is evaluated on the basis of preliminary results.

● 8.2-4 Mitchell, W. A., **The Lice earthquake in southeastern Turkey: a geography of the disaster,** *USAFA-TR-76-24,* Dept. of Economics, Geography and Management, USAF Academy, Colorado, Dec. 1976, 102.

On Sept. 6, 1975, an earthquake of Richter magnitude 6.9 occurred near the town of Lice in southeastern Turkey. Two thousand eighty-five people were killed and 3339 were injured in the disaster. The earthquake had a disastrous effect in 193 villages and towns, demolishing 7713 homes and damaging 8453 others. Soon after the earthquake, the Turkish government implemented a vast restoration program which included rebuilding the town of Lice and building 5805 homes throughout the area. This report is a comprehensive systematic analysis of the nature and extent of the Lice disaster. Recommendations for future reconstruction practices are offered.

● 8.2-5 Husid, R., Espinosa, A. F. and de las Casas, J., **The Lima earthquake of October 3, 1974: damage distribution,** *Bulletin of the Seismological Society of America,* **67**, *5*, Oct. 1977, 1441–1472.

The Oct. 3, 1974, earthquake caused severe damage to buildings of adobe and quincha construction, and also to masonry, reinforced masonry, and reinforced concrete structures in Lima and vicinity. Most of the damage to well-built structures was due, in part, to the lack of lateral resistance in the original design and to the fact that this earthquake had more energy around the 0.4 sec period than prior destructive earthquakes. Water tanks on the roofs of structures with four or five stories were damaged. Well-engineered single-story buildings were less affected than taller structures. Considerable structural damage to reinforced concrete structures occurred in the districts of Barranco, La Campina, La Molina, and Callao. In La Campina, a three-story building partly collapsed and other buildings sustained considerable damage. In La Molina, the buildings of the Agrarian Univ. sustained severe damage and some collapsed. In Surco, the district adjacent to La Molina, there was no appreciable damage. In Callao, a four-story building collapsed, and the upper half of a concrete silo collapsed. In reinforced concrete structures, column ties were frequently small in diameter, widely spaced, and not well connected. Usually, the reinforcement of resisting elements had no relation to their stiffnesses. Front columns in school buildings were restrained by high brick walls and had short effective lengths to allow building displacement in that direction. The windows in the rear walls gave the rear columns a much greater effective length; therefore, longitudinal displacement induced large shear forces in the front columns where most of the severe damage occurred. This problem was not considered in the design of these structures.

● 8.2-6 Rojahn, C. and Morrill, B. J., **The island of Hawaii earthquakes of November 29, 1975: strong-motion data and damage reconnaissance report,** *Bulletin of the Seismological Society of America,* **67**, *2*, Apr. 1977, 493–515.

Two earthquakes occurred on the island of Hawaii on Nov. 29, 1975, with magnitudes of (M_S) 5.7 at 0335 (local time) and (M_S) 7.2 at 0447. During the larger event, a maximum acceleration of 0.22 g was recorded in the southern part of Hilo, 43 km north of the epicenter. A 0.05 g threshold duration of 13.7 sec was measured for the same component. Smaller amplitude accelerograph records and four seismoscope records were obtained at two other locations on the island.

During or subsequent to the larger event, a large sector of the southeastern coastline subsided by as much as 3.5 m. A tsunami generated by the larger event caused at least one death (one person also missing), injury to 28 persons, and significant structural and nonstructural damage. Only scattered evidence of strong ground shaking was observed in the epicentral area, and most of the several dozen nearby structures sustained little or no structural damage from ground shaking. In Hilo, 45 km north of the M_S = 7.2 epicenter, structural and nonstructural damage was slight to moderate but more extensive than elsewhere on the island.

● 8.2-7 Mitchell, W. A., **Partial recovery and reconstruction after disaster: the Lice case,** *Mass Emergencies,* **2**, *4*, Dec. 1977, 233–247.

Natural hazard research includes the examination of human adjustment to earthquake disasters. The importance and need for empirical case studies of earthquake disasters have been recognized for developed and developing countries. This report is a systematic case study appraising the restoration after an earthquake disaster in Turkey. It is designed to assist in meeting the need for research on post-disaster recovery.

● 8.2-8 Saleem, A. S., **The Khulm (Tashqurghan) earthquake of March 19, 1976, Samangan Province, Afghanistan,** *Earthquake Notes,* **48**, *1-2*, Jan.-June 1977, 25–35.

The Khulm area, Samangan Province, northern Afghanistan, was shaken by an earthquake of 5.5 M_L on March 19, 1976. Twenty-nine persons died and 1139 residences were destroyed. The Khulm Gorge highway was closed by an estimated 12,000 m^3 of rockslides. An aftershock was felt about ten hr later. Damage in the areas close

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to the gorge was extensive. Many of the mud-brick dome roofs collapsed. The felt area was between 200-300 km^2, which suggests that the earthquake more likely occurred at a very shallow depth in the crust.

● **8.2-9** Arya, A. S. *et al.*, **A macroseismic study of November 6, 1975 Roorkee earthquake, Roorkee, India,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 255-261.

A moderate earthquake of magnitude 4.7 occurred on Nov. 6, 1975, around Roorkee, India. The earthquake was felt at Delhi, Dehradun, Bijnor, and other adjoining districts in the state of Uttar Pradesh. Different degrees of damage occurred mostly to old construction around Roorkee (29°51′N, 77°53′E). Cracks along mortar joints and falling of plaster were observed in a number of residential buildings. On the basis of macroseismic studies, the epicenter was estimated to be 6 km south of Roorkee. The maximum observed modified Mercalli intensity was VI. The earthquake was recorded on a multiple structural response recorder. An isoseismal map and the structural response results obtained are presented.

● **8.2-10** Poliakov, S. V. *et al.*, **Effects of the Kyzyl-Kum earthquakes on April 8 and May 17, 1976,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 262–271.

Two earthquakes which occurred in western Uzbekistan are described. Their magnitudes are estimated at 7.0 and 7.3, respectively. Photographs of damaged structures and tentative analyses of the consequences of both ruptures are presented.

● **8.2-11** Stratta, J. L. and Wyllie, Jr., L. A., **Preliminary reconnaissance report on Italian earthquake of May 6, 1976,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 272–278.

This report presents the results of a technical investigation of the effects of the Italian earthquake on structures of all types. The following are the preliminary conclusions reached by the reconnaissance team of the Earthquake Engineering Research Inst. in northern Italy. (1) The buildings in the damaged area had not been designed to resist earthquake effects. This lack of seismic resistance was the primary cause of damage to structures. (2) Many lowrise residential buildings of recent construction with substantial concrete or infilled masonry walls performed reasonably well in the earthquake. (3) Reinforced concrete members were not detailed for ductility considerations, and they exhibited damage consistent with past observations. (4) Bridge spans tend to slide on their supports while abutments tend to move independently of each other. Support details and restrainers must be designed for these considerations. (5) Steep mountain slopes are susceptible to earthquake-induced landslides. The potentially disastrous effects of these landslides on villages at the base of the mountains should be recognized.

● **8.2-12** Cano, J. H., **Guatemala earthquake of February 4th, 1976: description and analysis of damages caused on buildings,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 299–303.

The effects of damage to buildings in central Guatemala are analyzed. Special consideration is given to the materials and types of construction used in the affected areas. It is concluded that most of the damage resulted from the lack of seismic-resistant provisions.

● **8.2-13** Singh, S. *et al.*, **Damage during Kinnaur earthquake of January 19, 1975 in Himachal Pradesh, India,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 184–190.

On January 19, 1975, a strong earthquake of magnitude 6.8 occurred in the border districts of Himachal Pradesh which lies in the Himalayan belt. It is reported that 60 persons were killed and 2000 homes were damaged. Stone masonry construction underwent extensive damage, while corrugated iron sheets nailed on timber arches and frames forming cylindrical shell and barrack-type structures sustained little damage. This paper briefly describes the damage to various structures and highways in the area. An isoseismal map of the earthquake is presented.

● **8.2-14** Glauser, E. C., **The May 6, 1976, Friuli - earthquake assessment and interpretation of building damage,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 279–288.

This report presents the results of an investigation of the ultimate carrying capacity of diverse types of structures which were damaged or destroyed during the Friuli earthquake. Structural elements and systems, joints and supports, and earthquake design specifications are considered. Photographs of damaged structures are included.

● **8.2-15** Braga, F., Briseghella, L. and Pinto, P. E., **The Friuli (Italy) May and Sept. 1976 earthquake: a brief survey of the damages,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 289–294.

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An overview is presented of the damage caused to the principal types of construction in the Friuli region by the sequence of shocks that lasted from May to September 1976.

● **8.2-16** Ohtani, K. and Kubota, T., **Damage of structures due to the 1974 Izuhanto-oki earthquake,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 191–196.

The southern part of the Izu peninsula in Japan was shaken at 8:33 a.m. on May 9, 1974, by a strong earthquake. Landslides caused considerable damage to houses, especially in the Nakagi district. Destruction of houses due to foundation and ground movement occurred mainly in the Irozaki and Iruma districts. This report classifies the states and causes of damaged structures in these areas and investigates problems connected with disaster prevention and earthquake engineering.

8.2-17 Stevens, A. E., **Some twentieth-century Canadian earthquakes,** *Geoscience Canada,* **4,** *1,* Mar. 1977, 41–45.

Six contemporary earthquakes have been chosen to illustrate typical effects of past earthquakes in southern Canada: 1925, 1929, 1944, 1973 in eastern Canada; 1909 in central Canada; 1946 in western Canada. In the epicentral region, brick chimneys of frame houses have broken at the roof line or have lost bricks from the top. Damage has been greater in structures not located on rocky ground or not well designed, constructed, or maintained. Twenty-eight persons drowned following two earthquakes. The effects of future earthquakes may differ in nature and extent from those experienced in the past, particularly if such events are centered near urban areas.

● **8.2-18** Deza, E., Jaen, H. and Kuroiwa, J., **Investigation of the Peruvian earthquake of October 3, 1974 and seismic protection studies of the Lima metropolitan area,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 866–871.

Many developing countries located in seismically active areas have a large percentage of their population concentrated in their capital cities, where a great percentage of their economic and industrial activities take place. These cities are vulnerable to earthquake damage inasmuch as earthquake-resistant designs have only recently been adopted. This paper deals with the studies being made in Lima, Peru, since early 1973, of the probable effects of a serious earthquake on the buildings and lifeline networks of that city. The study of the damage caused by the Oct. 3, 1974, earthquake added valuable new information. Short-, medium-, and long-range plans for improving the earthquake safety of Lima are being proposed to the Peruvian government through the National Committee of Civil Defense, based on the preliminary results of the studies.

● **8.2-19** Fattal, G., Simiu, E. and Culver, C., **Observations on the behavior of buildings in the Romania earthquake of March 4, 1977,** *NBS SP 490,* U.S. National Bureau of Standards, Washington, D.C., Sept. 1977, 172. (NTIS Accession No. PB 272 356)

Observations of the damage to buildings resulting from the earthquake of March 4, 1977, in Romania are presented. The report was prepared by engineers from the National Bureau of Standards who participated as members of the U.S. government team dispatched to Romania under the auspices of the Office of Foreign Disaster Assistance Agency for International Development. A summary of the team's activities is included. Background data on the seismic history of Romania, the characteristics of the earthquake, and descriptions of damage to specific buildings are also included. The types of building construction and the history of the development of seismic design requirements for buildings in Romania are discussed. Recommendations, based on the observations, are presented for structural research.

● **8.2-20** Nielsen, N. N. *et al.,* **The Honomu, Hawaii earthquake,** Committee on Natural Disasters, National Research Council, Washington, D.C., 1977, 79.

The island of Hawaii is the most seismically active of the Hawaiian Islands, with 12 earthquakes of magnitude 6 or larger since 1929. It has two active volcanos but they seem to be unconnected with larger earthquakes. At the time of the Honomu earthquake on Apr. 26, 1973, the Kilauea volcano was in active eruption, but the epicenter was far removed from the volcano. The earthquake occurred on a known fracture zone. The depth of focus was approximately 41 km; Hawaiian earthquakes usually occur at much shallower depths. The magnitude of the earthquake was 6.2. The epicenter was located at Honomu, which is 10 km north of Hilo and about 2 km inland.

About 2 months before the earthquake occurred, two strong-motion accelerographs and four seismoscopes were installed in the islands. One of the strong-motion accelerographs was installed at Kilauea on the island of Hawaii; its distance from the epicenter was 50 km. Another accelerograph was installed in Honolulu, about 300 km from the epicenter. Both instruments were triggered by the earthquake. The maximum acceleration at Kilauea was 0.17 g and strong ground motion lasted about 7 seconds. The Honolulu record showed a maximum acceleration of 0.03 g. Three days after the earthquake, additional instrumentation was flown in to permit monitoring of aftershocks. Approximately 10 shocks of magnitudes between 2 and 3 were recorded.

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The Wailuku River is the dividing line between the Mauna Kea and the Mauna Loa lava flows. Mauna Loa is still active. At the last stages of volcanic activity of Mauna Kea, ash deposits covered the northeast portion of the island, north of the Wailuku River. The result is that soil conditions are different on the two sides of the river. North of the river the ash cover is about 20 to 30 ft thick. The ground south of the river is lava rock. It was evident from the earthquake damage that behavior of the two soils was different; damage was much heavier on the volcanic ash north of the river. Aftershock equipment was installed on both sides of the river to explore the differences in response. For most frequencies, the velocity response of the volcanic ash was five to ten times as large as the response measured on the lava rock. There were no signs of liquefaction, even though volcanic ash tends to liquefy under traffic of heavy construction equipment.

Damage was rather minor considering the magnitude of the earthquake. This can be attributed to several facts: (1) the earthquake was very deep seated; (2) the duration of the strong motion was short; and (3) the island of Hawaii is sparsely populated. There are very few tall buildings in Hilo. The total amount of damage has been estimated at $6,000,000, a significant portion of which was damage to public roads and bridges. Numerous land and rock slides occurred in the northeastern portion of the island. Most of the damage to residential units occurred to buildings located on volcanic ash deposits within about a 20-km radius of the epicenter.

There was significant nonstructural damage. Many students in school buildings were injured from falling light fixtures and false ceilings. Many residential units shifted on their foundations. It was evident that an earthquake of longer duration would have caused considerably more damage. Damage to power lines, water supply, and telephone lines was severe. The northeastern portion was in a state of emergency for several days. In Honolulu, about 300 km from the epicenter, damage was minor.

● 8.2-21 Pichard, P., Ambraseys, N. N. and Ziogas, G. N., **The Gemona di Friuli earthquake of 6 May 1976,** *Technical Report RP/1975-76/2.222.3,* United Nations Educational, Scientific and Cultural Organization (UNESCO), Paris, Dec. 1976, 184.

On May 6, 1976, an earthquake of magnitude 6.5 occurred in the region of Friuli in northern Italy. The earthquake killed 965 and injured 2,286 people and caused damage estimated at $2.5 million. Damage to historical monuments and the loss in terms of art and architecture is inestimable. A UNESCO mission was sent to the affected area to study the effects of the earthquake and, in consultation with the local authorities, to develop plans for the protection and restoration of the affected historical monuments and housing facilities in the region. The individual reports of the three members of this mission constitute the three parts of the report. Part I considers Friuli's historical monuments after the earthquake; Part II discusses the earthquake itself; and Part III is concerned with the destruction of educational facilities in the area.

● 8.2-22 Ambraseys, N. N., **The Romanian earthquake of 4th March 1977. Preliminary report of UNESCO Earthquake Reconnaissance Mission,** Imperial College of Science and Technology, London, Apr. 1977, 48.

The Mar. 4, 1977, Romania earthquake, magnitude 7.2, is discussed. Its characteristics are compared with those of a similar event which occurred on Nov. 10, 1940. Damage to structures is assessed and recommendations for revising Romania's building codes are given. Whether to repair or demolish damaged structures is also discussed.

● 8.2-23 **The Guatemala earthquake of February 4, 1976: a preliminary report by the EERI reconnaissance team,** *Earthquake Engineering Research Institute Newsletter,* **10,** *2B,* Mar. 1976, 100.

Summaries of investigations, news reports, and correspondence regarding the Feb. 4, 1976, Guatemala earthquake are presented. Regional and local maps are included.

● 8.2-24 **Contribution to the study of Friuli earthquake of May 1976,** CNEN-ENEL Commission on Seismic Problems Associated with the Installation of Nuclear Plants [Rome], Nov. 1976, 135.

The contents follow: research on the relationships between surface geology and seismogenetic tectonic structures; macroseismic aspects and seismologic and statistical considerations; processing and interpretation of the accelerometer recordings; soil behavior; and damage to the structures and structural analyses.

8.3 Effects on Buildings

● 8.3-1 Foutch, D. A., Housner, G. W. and Jennings, P. C., **Dynamic responses of six multistory buildings during the San Fernando earthquake,** *EERL 75-02,* Earthquake Engineering Research Lab., California Inst. of Technology, Pasadena, Oct. 1975, 108.

The main purpose of this report is to provide the practicing engineer with a collection of brief descriptions of a variety of multistory buildings and their responses to the San Fernando earthquake, and also to provide a list of references where more detailed information about these and other buildings can be found. Of the six buildings included in this report, three are steel frame buildings, two are reinforced concrete frame buildings and one is a reinforced concrete shear wall building. The buildings range from 7 to 42 stories in height and are located at

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distances of approximately 8 to 21 miles from the center of the San Fernando earthquake.

● 8.3-2 Zhunusov, T. Zh. *et al.*, **Damage to school buildings in Bukhara during the 1976 Gazli (Kazylkum) earthquake** (Povrezhdeniya shkolnykh zdanii v Bukhare pri Gazliiskikh [Kazylkumskikh] zemletryaseniyakh 1976 g., in Russian), *Seismostoikoe stroitelstvo, 5,* 1977, 52–56.

The intensity of the May 17, 1976, earthquake in Bukhara was investigated by a damage analysis performed on approximately 500 buildings of various types, including 113 school buildings. The schools selected are uniformly distributed in the city and they include buildings of diverse types, from frame buildings with wall panels to single-story brick structures. A detailed analysis of the performance of 20 school buildings is given.

● 8.3-3 Arioglu, E. and Anadol, K., **Response of rural dwellings to recent destructive earthquakes in Turkey (1973-1975),** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 249–254.

The seismic activity of Anatolia has been increasing. Three destructive earthquakes are described: Izmir, Feb. 1, 1974, M = 5.2; Gelibolu, Mar. 27, 1975, M = 5.7; and Lice, Sept. 6, 1975, M = 6.9. The parameters and characteristics of the Izmir and Gelibolu earthquakes are outlined, and the damage to rural dwellings in Izmir, Gelibolu, and Lice is described. The design criteria of prototypical low-cost rural dwellings in the region are discussed.

● 8.3-4 Wyllie, Jr., L. A. *et al.*, **Performance of Banco Central related to faulting,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2417–2422.

The Banco Central de Nicaragua is located on an active fault trace which displaced about 17 cm left lateral in the earthquake of Dec. 23, 1972. The deep basement of the structure had sufficient strength and rigidity to resist the faulting and displace the surface trace to the west. The extensive structural and nonstructural damage to the Banco Central was related solely to ground shaking. In this paper, structural observations are related to the documented faulting and backfill settlement.

● 8.3-5 Niccum, M. R. *et al.*, **Banco Central de Nicaragua: a case history of a high-rise building that survived surface fault rupture,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2423–2428.

The Banco Central de Nicaragua was inadvertently built across an active trace of the Banco fault. During the 1972 Managua earthquake, surface faulting occurred on one trace of that fault, and the strong security vault in the basement of the bank resisted the faulting. Near the building, the rupture deviated from the active trace and broke a series of new fault cracks through the tensional quadrant in the old fault zone. Northeast of the building, surface displacements again coincided with the northward projection of the fault trace. Model studies suggest that structures can be built in a fault zone if the characteristics of the fault and the soils are known so that fault displacements can be considered in the design.

● 8.3-6 Tomii, M. and Yoshimura, K., **Damage to a reinforced concrete hotel building due to the Ohita earthquake of April 21, 1975,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 209–214.

Information about the K. L. Hotel, damaged beyond repair during the 1975 Ohita earthquake, is given, and the principal cause of the damage is presented.

● 8.3-7 Scholl, R. E. and Blume, J. A., **Damaging response of low-rise buildings,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 979–984.

Because lowrise buildings constitute a large percentage of buildings in earthquake-prone areas of the world, a large percentage of economic and life losses sustained during past earthquakes have resulted from damage to lowrise buildings. This paper on such structures discusses the factors contributing to earthquake-caused damage, summarizes the available motion-damage relationship data, and presents some of the limitations of damage prediction and of earthquake-resistant design and construction.

● 8.3-8 Karaesmen, E., **A survey of building damages in September 6, 1975 Lice (Turkey) earthquake,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 221–226.

On Sept. 6, 1975, the town of Lice, Turkey, experienced an earthquake with an estimated intensity of VIII on the modified Mercalli scale. In this report, structures in the Lice region are classified by types, and the damage to each type is discussed. Emphasis is given to stone masonry and reinforced concrete structures. One of the reinforced concrete structures is analyzed in detail.

● 8.3-9 Tsuchiya, H. *et al.*, **Damage to reinforced concrete buildings due to the Oita earthquake of April 21, 1975,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 203–208.

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The dynamic response of the main building of the K. L. Hotel, a four-story reinforced concrete structure damaged beyond repair during the Oita earthquake, is analyzed and compared with the results of an investigation after the earthquake. The analysis presented in this work demonstrates that the damage was caused primarily by the severity of ground motion and the large change in stiffness and strength across the second floor level, thus imposing extremely large ductility requirements to the elements of the first story, which apparently had not been anticipated. It is concluded that, in order to protect buildings from damage, it is important to consider the average shear stress of columns and shear walls and the variance of axial stress in the column during earthquakes, especially for buildings near a fault.

● 8.3-10 Shiga, T., **Earthquake damage and the amount of walls in reinforced concrete buildings,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2467–2472.

An analysis was made of the observed damage to lowrise reinforced concrete buildings around Hachinohe City resulting from the 1968 Tokachi-oki earthquake. By evaluating the two parameters, wall area ratio and average shear stress in walls and columns, it was concluded that damaged and undamaged buildings could be significantly distinguished. Based on the relation of earthquake damage to wall ratio and nominal shear stress in columns and walls, the probability distribution of the earthquake-resistant capacity of existing buildings was estimated, and the prediction of the extent of earthquake damage was made.

● 8.3-11 Yamada, M. and Kawamura, H., **Damaged and non damaged reinforced concrete modern buildings at the Ooita-earthquake, Apr. 21, 1975, Japan,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 197–202.

The most noteworthy result of the Ooita earthquake was the simultaneous existence of damaged and undamaged reinforced concrete buildings located close to the epicenter. Clarification of the causes of such differences may disclose useful knowledge for the aseismic design of reinforced concrete buildings. The authors formerly proposed a new fundamental aseismic design method for lowrise and rigid-type reinforced concrete buildings, but there was a lack of experimental verification on an actual full-scale building. This paper describes the application of the authors' aseismic design approach to damaged and undamaged reinforced concrete buildings.

8.3-12 Fintel, M., **Performance of precast concrete structures during Rumanian earthquake of March 4, 1977,** *Journal of the Prestressed Concrete Institute,* 22, 2, Mar.-Apr. 1977, 10–15.

This report is based on the author's observations during a visit to Bucharest between Mar. 19-22, 1977. The highrise precast concrete buildings performed excellently during the earthquake.

8.4 Effects on Miscellaneous Structures and Systems

● 8.4-1 Cooper, J. D., **Structural damage to bridges resulting from the Guatemala earthquake,** *Wind and Seismic Effects,* VIII-1–VIII-27. (For a full bibliographic citation, see Abstract No. 1.2–4.)

The Guatemala earthquakes of Feb. 4 and 6, 1976, caused severe economic hardships because of highway bridge failures and damage. The damage to three major bridges, Agua Caliente, La Asuncion, and Incienso, is described. A general discussion of damage to bridges and the roadway along a major highway, the Atlantic Highway (Route CA9), is also presented.

● 8.4-2 Sawada, K., **Earthquake damages to earth structures,** *Wind and Seismic Effects,* VI-44–VI-49. (For a full bibliographic citation, see Abstract No. 1.2–4.)

This paper presents the problems associated with the damage to earth structures during earthquakes and how some of these problems have been solved. The principle of the design of earth structures not discussed to date are also presented. Initially, data have been collected on the amount of damage to earth structures during earthquakes, how they have failed, and relationships to such failures. These data are then used in formulating the mechanics of the failure. Finally, details of the repairs required, as listed by field engineers performing the repairs, were examined.

● 8.4-3 Narita, K., **Study on pipeline failure due to earthquake,** *Proceedings of U.S.-Japan Seminar on Earthquake Engineering Research with Emphasis on Lifeline Systems,* Japan Society for the Promotion of Earthquake Engineering, Tokyo, Nov. 1976, 157–176.

Reports of damage to buried pipelines during the Niigata earthquake have been studied. From the result of this study, it is concluded that stratification of pipeline components by their ductility and flexibility, and investigation of ground movement caused by earthquakes are important for improving the seismic resistance of pipelines.

● 8.4-4 Hoshiya, M., **Seismic damage of embankment by quantification analysis,** *Proceedings of U.S.-Japan Seminar on Earthquake Engineering Research with Emphasis on Lifeline Systems,* Japan Society for the Promotion of Earthquake Engineering, Tokyo, Nov. 1976, 141–156.

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Damages to railroad and river embankments were analyzed based on verbal reports taken following major earthquakes in Japan. Utilizing a theory of quantification, the main causes of the damages were qualitatively evaluated. It is recommended that the results be integrated into the established damage appraisal criteria for embankments.

● 8.4-5 California Dept. of Water Resources, **Performance of the Oroville Dam and related facilities during the August 1, 1975 earthquake,** *203,* California Dept. of Water Resources, Sacramento, Apr. 1977, 102.

This bulletin documents the performance of Oroville Dam and related facilities during the Aug. 1, 1975, earthquake, the sequence of aftershocks that followed during the remainder of August, and the scattered shocks that continued during the fall months. The bulletin is presented in two parts. In Part 1, the performance of the Oroville complex during the main event and succeeding aftershocks is described. Also included is a discussion of the detailed inspections and investigations of the Oroville facilities that followed the Aug. 1 event. Part 1 also contains discussions of the seismology and geologic history of the affected area, along with seismological and geological data from investigations conducted before and after the earthquake. Part 2 describes the actions of the Dept. of Water Resources in carrying out its responsibilities for the safety of all dams in the area affected by the earthquake. Part 2 discusses the actions of the Div. of Safety of Dams following the Aug. 1 event, including its evaluation of the safety and performance of all jurisdictional dams in the Oroville area. Also discussed are studies of the seismicity of the Oroville area. Finally, Part 2 presents a discussion of the effect of the Oroville earthquake on seismic criteria that should be used as a basis for (1) the design of new dams, and (2) evaluations of the adequacy of existing dams and related facilities.

A subsequent report will be prepared that will include structural reanalysis of the Oroville Project facilities for a larger earthquake, and results of the seismological and geological investigations.

● 8.4-6 Cooper, J. D., **Bridge and highway damage resulting from the 1976 Guatemala earthquake,** *FHWA-RD-76-148,* Office of Research and Development, Federal Highway Admin., Washington, D.C., May 1976, 50.

The Guatemala earthquakes of Feb. 4 and 6, 1976, caused severe economic hardships because of highway bridge failures and damage. The damage to three major bridges, Agua Caliente, La Asuncion, and Incienso, is described. A general discussion of damage to bridges and the roadway along a major highway, the Atlantic Highway (Route CA9), is also presented.

● 8.4-7 Smith, D. W., **Why do bridges fail?,** *Civil Engineering, ASCE,* 47, *11,* Nov. 1977, 58-62.

The author has studied 143 bridge failures that occurred throughout the world between 1847 and 1975 and has grouped the causes of failure into nine categories. From the most to the least frequent, the following causes are listed: (1) flood and foundation movement, (2) unsuitable or defective permanent material, (3) overload or accident, (4) inadequate or unsuitable temporary works or erection procedure, (5) earthquake, (6) inadequate design in permanent material, (7) wind, (8) fatigue, and (9) corrosion. Examples and discussions are presented.

● 8.4-8 Clough, G. W. and Fragaszy, R. F., **A study of earth loadings on floodway retaining structures in the 1971 San Fernando Valley earthquake,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2455-2460.

The floodway system in the greater Los Angeles, California, area includes over 160 km of channel and serves to transport the runoff from the winter rains to the Pacific Ocean. The floodway structures consist of open U-shaped channels with the wall tops set flush to the ground surface and completely buried culverts; both types of sections were significantly damaged in localized regions during the 1971 San Fernando Valley earthquake. For the investigation described in this paper, the behavior of the open channel structures was studied. The behavior of the underground structures has been previously described by Hradilek. The location of the study region, the floodways and the earthquake energy center, and contours of estimated maximum accelerations are shown. Estimated maximum accelerations in the region range from 0.65 g to 0.2 g; the closest floodway is located only 10 km from the energy center. The well-known San Fernando Dam, which failed during the earthquake, is located near several of the floodways.

The major cause of damage to the open channel floodways was excessive earth loads exerted on the channel walls by the wall backfills. Because the behavior of the walls was well documented and field conditions were well defined, the seismically induced earth loads can be accurately calculated. The principal effort of this paper is to use this information to: (1) evaluate the conventional approach towards determining seismically induced earth loadings; and, (2) investigate the reserve of strength built into earth retaining structures by conventional structural design procedures. Previous work of the type undertaken herein has had to rely on purely analytical or model studies. The San Fernando Valley floodways offer a case history which, for the first time, allows study of the earth loading problem under actual earthquake conditions.

● 8.4-9 Yen, B. C. and Wang, W. L., **Seismically induced shallow hillside slope failures,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2461-2466.

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A one-dimensional pseudostatic model is used to study shallow slides caused by the 1971 San Fernando seismic activity. Slide length and displacement can be estimated theoretically from the various bond strengths between the soil and base rock. Field investigation has shown a reasonable range for the pseudostatic acceleration and a characteristic range for the dimensionless sliding length for three geological formations in the Lopez Canyon area north of San Fernando Valley, California.

● 8.4-10 Katayama, T., Kubo, K. and Sato, N., **Quantitative analysis of seismic damage to buried utility pipelines,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3369–3375.

Seismic damage to buried utility pipelines caused by past earthquakes is summarized quantitatively. By using the San Fernando data, the relation between damage and severity of ground shaking is discussed. Results of analysis of the relation between seismic damage to buried water pipes and ground conditions are reported.

● 8.4-11 Sato, Y. and Miura, S., **Deformation of railway track and running stability of train in earthquake,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 3341–3346.

This paper reviews a survey of train damage in ten recent strong earthquakes in Japan, actual conditions for the damage, the calculated tolerance of the deformation of railway structures with use of the vehicle model on the Shinkansen, the behavior and amplification factor of structural displacement in the longitudinal direction, and the stability of ballasted track and the deformation of it during an earthquake. Finally, equipment for simulating track movement in an earthquake for the further analysis of the running stability of railway vehicles is demonstrated.

● 8.4-12 Seed, H. B., Makdisi, F. I. and De Alba, P., **The performance of earth dams during earthquakes,** *UCB/EERC-77/20,* Earthquake Engineering Research Center, Univ. of California, Berkeley, Aug. 1977, 55. (NTIS Accession No. PB 276 821)

In the design of embankments against earthquakes, in addition to the proper use and understanding of the material properties and behavior during seismic loading, and in addition to the proper use of analytical procedures to estimate the dynamic response, considerable insight and judgment are required. This can only be gained through an intimate knowledge of the strengths and limitations of the analysis and testing procedures themselves, and by studying the behavior of similar embankments during actual earthquake loading conditions.

The purpose of this report is to summarize available information concerning the behavior of dams subjected to strong earthquake shaking. The report is intended to complement an initial review by Ambraseys by presenting further details concerning embankment construction materials, procedures, and performance records in earthquakes during the past 17 years. The salient features of observed embankment performance are summarized, mainly for six major earthquakes, and conclusions are drawn which may aid the design engineer in making more meaningful evaluations of the data provided by analytical design procedures.

8.5 Effects and Near Surface Geology

● 8.5-1 Katayama, T., **Effect of ground conditions on seismic damage to buried pipelines,** *Proceedings of U.S.-Japan Seminar on Earthquake Engineering Research with Emphasis on Lifeline Systems,* Japan Society for the Promotion of Earthquake Engineering, Tokyo, Nov. 1976, 197–210.

The effects of ground conditions on the seismic damage to buried pipelines were statistically analyzed by using the damage data obtained for the buried water pipes in the metropolitan area of Tokyo during the 1923 Kanto earthquake. It is concluded that, although the coefficient of multiple correlation was low, the existence of correlation between damage and ground conditions was apparent from the results of a multivariate regression analysis. Also discussed is how to apply this type knowledge to the macroscopic assessment of possible future earthquake damage to buried pipelines.

● 8.5-2 Tezcan, S. S. *et al.*, **Resonant period effects in the Gediz, Turkey earthquake of 1970,** *Earthquake Engineering and Structural Dynamics,* 5, 2, Apr.-June 1977, 157–179.

One of the most interesting features of the Gediz, Turkey, earthquake of Mar. 28, 1970, was the damage and partial collapse of some buildings at the Tofas automobile factory located at a distance of about 135 km from the epicenter. Studies of the damage at the factory site illustrate the potential effects of soil amplification and resonant period interaction in inducing damaging motions in engineering structures. The probable effects of the earthquake on the garage and paint shop buildings are illustrated schematically. It seems likely that a base rock acceleration of the order of 0.02 g was amplified about 330% by the overlying soil deposit, and this in turn was amplified a further 150–360% by the garage and paint shop buildings, to the point where major damage occurred. The primary cause of these large amplifications was probably the similarity in values of the predominant period of the base rock motions, the fundamental period of the overlying soil deposit and the fundamental periods of the buildings, all of which were of the order of 1.0–1.25s.

For other structures in the same area, with different natural periods of vibration, no significant damage occurred because of their much smaller response to the earthquake motions. The studies also lead to the following conclusions: (1) The damage which developed at the Tofas factory site would not have been anticipated on the basis of the standard spectral shapes often used for design purposes or standard seismic design parameters. (2) The damage patterns could have been anticipated in this case on the basis of soil engineering analyses of site response. Although other potential patterns would also be indicated by these procedures, the predictive capability of this method of approach can be useful in determining the potential range of ground response characteristics in many cases. (3) The analysis of soil response and damage at the Tofas factory site provides an example of the usefulness of aftershock motion records in predicting the probable characteristics of ground response during stronger earthquakes. It should be noted in this regard that the rock/soil amplification factor was about 6 during the aftershock and only about 3 during the main shock. Thus, some analytical treatment is necessary in translating aftershock records to other excitation levels. In spite of this, aftershock records can do much to clarify uncertainties in soil and rock response characteristics and facilitate interpretation of ground response investigations during earthquakes.

● **8.5-3** Espinosa, A. F. *et al.*, **The Lima earthquake of October 3, 1974: intensity distribution,** *Bulletin of the Seismological Society of America,* **67,** *5,* Oct. 1977, 1429–1439.

The epicenter of the Oct. 3, 1974, earthquake was 80 km west of Lima, Peru, at 12.2°S and 77.67°W. This earthquake caused severe damage in Lima and vicinity, producing a maximum modified Mercalli intensity of IX in a few small scattered areas in Lima. The modified Mercalli intensity in Lima varied from V to IX; in towns south of Lima the intensity exceeded VIII. The areas of high intensity, both in Lima and along the coast, appear to be related to unfavorable soil conditions or to a high water table, as is evident in the areas where large damage and/or differential settlement took place. Slumping was also observed along the coastal road south of Lima. In Callao, differential earth settlement was associated with liquefaction of the soil. Pockets of high intensities, such as in the districts of La Molina and Chorrillos, are correlated with possible local ground-amplification effects. Subsidence of up to 35 cm took place in some areas along the wave-cut terrace in Miraflores, and 15 cm of subsidence was observed in Chorrillos. The isoseismal map constructed for Lima can be used in a preliminary zonation of Lima for potential earthquake effects.

● **8.5-4** Lance, R. J., Fogle, G. H. and Long, L. T., **Report on the earthquake of December 27, 1976 in southern Georgia,** *Earthquake Notes,* **48,** *1-2,* Jan.-June 1977, 51–56.

The earthquake in southern Georgia on Dec. 27, 1976, occurred about 1:57 a.m., EST. The hypocenter was located near 32.05° north latitude and 82.3° west longitude. Instrumental data indicate a magnitude of 3.5 to 3.7 with a focal depth of about 5 km. The maximum epicentral intensity was MM V and the earthquake was felt over about 5620 sq km.

● **8.5-5** Fox, F. L. and Spiker, C. T., **Intensity rating of the Attica (N.Y.) earthquake of August 12, 1929: a proposed reclassification,** *Earthquake Notes,* **48,** *1-2,* Jan.-June 1977, 37–46.

The Attica, New York, earthquake of Aug. 12, 1929, has been classified MM VIII. Recent detailed investigations indicate that the shock was of intensity VII. These investigations include a review of scientific accounts, original reports, and original newspaper accounts, studies of instrumental records, comparison with other shocks of intensity VII and VIII, and interviews with local residents and witnesses to the disturbance. A detailed review of the Modified Mercalli Scale of intensity and its application was part of the investigation.

None of the investigations support a classification of MM VIII. Original accounts of the earthquake indicate an intensity no higher than VII. Magnitude estimates, developed from diverse instrumental records, correlate with an intensity of VII or less. Comparison with the effects of intensity VIII shocks clearly indicates that the 1929 Attica event was less intense, and comparison with other intensity VII shocks supports this contention. Interviews with local residents indicate that the Attica shock was MM VII. It is concluded that the Attica earthquake of Aug. 12, 1929, was an intensity VII event and should be so classified in the record.

● **8.5-6** Johnson, W. J. and Kissenpfennig, J. F., **Vibratory ground motion from a distant large magnitude earthquake: a discussion of the 1755 Lisbon earthquake,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 330–331.

Large-magnitude earthquakes (Richter magnitude > 8.5) are characteristically different from medium- or low-magnitude earthquakes. Even at large epicentral distances, large-magnitude earthquakes cause significant damage; but this damage, unlike low-magnitude nearby earthquakes, is associated with low frequency, long duration, and low-amplitude peak ground motions. This article discusses the example of the large-magnitude 1755 Lisbon earthquake which occurred offshore along the Azores-Gibraltar fault.

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● 8.5-7 Yarar, R., Tezcan, S. S. and Durgunoglu, H. T., **Soil amplification effects in the Adapazari, Turkey, earthquake of 1967,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. III, 1977, 2435–2440.

A soil amplification study was performed to determine the reasons for the collapse of two reinforced concrete apartment buildings during the July 22, 1967, earthquake in Adapazari, Turkey. The soil data are based on a series of borings, in-situ measurements, and laboratory testing. Results of the nonlinear shear wave propagation analysis illustrate that the predominant period of the soil is very close to the natural periods of the collapsed structures. The base rock accelerations were amplified at the surface by about four to five times and developed such an intensity of severe shaking that the moment-resistance capacities of the columns were much exceeded as a result of the quasi-resonance condition.

● 8.5-8 Blume, J. A., **Engineering intensity scale data for the 1971 San Fernando earthquake,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 729–734.

The Engineering Intensity Scale (EIS), based on 5%-damped spectral response velocities, is used to provide mapped iso-intensity lines of the 1971 San Fernando earthquake in three period bands. Variations of EIS intensity with distance from the energy source are shown, as are comparisons with modified Mercalli (MM) intensity. EIS intensity ratings for a high-frequency band are related statistically to damage of lowrise buildings, and ratings of an intermediate-period band are related to damage of high-rise buildings.

● 8.5-9 Srivastava, L. S. and Singh, S., **Ground failure during Kinnaur earthquake of January 19, 1975 Himachal Pradesh, India,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 295–296.

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9. Earthquakes as Natural Disasters

9.1 Disaster Preparedness and Relief

● **9.1-1** Tamura, K., Ashida, Y. and Okamoto, S., **Control of train operation on the new trunk lines on the occasion of earthquake,** *Proceedings of U.S.-Japan Seminar on Earthquake Engineering Research with Emphasis on Lifeline Systems,* Japan Society for the Promotion of Earthquake Engineering, Tokyo, Nov. 1976, 399–415.

When an earthquake occurs, trains running on the new Tokaido and the Sanyo trunk lines are stopped automatically by cutting the electric power supply and by applying the emergency brake. For this purpose, two types of alarm seismographs, which are triggered when the horizontal acceleration of an earthquake exceeds 40 gal or 80 gal, have been set at substations located along the trunk lines at intervals of about 22 km. To improve the existing system of the control of train operation, simultaneous observations of earthquakes at three locations on the Pacific Coast have been conducted.

● **9.1-2** Kuribayashi, E. and Tazaki, T., **A trend on earthquake engineering researches for lifeline systems,** *Proceedings of U.S.-Japan Seminar on Earthquake Engineering Research with Emphasis on Lifeline Systems,* Japan Society for the Promotion of Earthquake Engineering, Tokyo, Nov. 1976, 459–479.

This article presents: (1) the present status and scope of investigations of the earthquake-resistant design of lifeline systems and a summary of earthquake-resistant design criteria in Japan, (2) the results of investigations of the methods of retrofitting existing structures, and (3) the results of investigations related to earthquake disaster prevention which are now being conducted by various organizations in Japan.

● **9.1-3** Jennings, P. C., **Earthquake problems of networks and systems,** *Proceedings of U.S.-Japan Seminar on Earthquake Engineering Research with Emphasis on Lifeline Systems,* Japan Society for the Promotion of Earthquake Engineering, Tokyo, Nov. 1976, 5–14.

The author presents a general view of the earthquake problems of lifeline systems, identifies and discusses some characteristics of the systems that are important for improving their earthquake performance, and points out some of the areas of needed research.

● **9.1-4** Schiff, A. J., Feil, P. J. and Newsom, D. E., **Evaluating power system response to earthquakes with simulation,** *Proceedings of U.S.-Japan Seminar on Earthquake Engineering Research with Emphasis on Lifeline Systems,* Japan Society for the Promotion of Earthquake Engineering, Tokyo, Nov. 1976, 305–315.

A method is described for using computer simulation to evaluate the ability of a lifeline to provide services in a postearthquake environment. The simulation provides statistical data on the expected types, geographical distribution, and effects of equipment damage, as well as the extent and duration of customer disruptions. This is done by realistically taking into account the redundancy and flexibility of lifeline systems. These factors are particularly important for power and communications systems. The recovery process is simulated and thus amenable to analysis and evaluation. While the methodology which is developed could be readily applied to other ground-based lifelines, parts of the present project are tailored to modeling electric power systems.

● **9.1-5** Cullen, J. M., **A comparison of lifeline system vulnerability in two large regional disasters: the Wyoming Valley flood and the projected Puget Sound earthquake,** Federal Disaster Assistance Admin., Seattle [1976], 58.

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In June 1972, Hurricane Agnes struck the eastern part of the United States with strong winds and high water, creating such widespread flooding that Agnes has come to be known as the worst natural disaster ever to strike the United States. One of the areas hardest hit was the Wyoming Valley in eastern Pennsylvania. The fact which makes the Wyoming Valley flood an important object of study for the planner concerned with natural hazard mitigation and disaster lifeline systems is not the event itself—however devastating it may have been. It is rather the human reactions before, during, and immediately after the disaster which make it a vital object of study for urban planners and those involved in civil preparedness.

In 1973, the Federal Disaster Assistance Admin. (FDAA) contracted with the U.S. Geological Survey to do a study of the vulnerability of essential public services (lifeline systems) in a six-county area surrounding Puget Sound to the largest earthquake that could reasonably be expected to occur in the region. The study was released late in 1975 and has been the focus of a major educational effort by the staff of the Region 10 office of FDAA. This paper is a part of that effort to assist city, county and state officials, civil preparedness agencies, hospitals, public utilities, and other providers of emergency services in planning, not only for the largest earthquake expected to occur but for other earthquakes of the magnitude that have already shaken the area in the past.

The root causes of the Wyoming Valley problems were two: the "normalcy bias" ("It can't happen here.") and the lack of both long- and short-range planning appropriate to the nature of the disaster. This paper is an effort to assist disaster planners in asking themselves if the same things could happen in Puget Sound. On the positive side, planners should ask themselves what steps are being taken to find and train the potential community and neighborhood leaders before a disaster strikes.

● **9.1-6** Quarantelli, E. L. and Taylor, V. A., **An annotated bibliography on disasters and disaster planning, 3rd edition,** *Misc. Report 16,* Disaster Research Center, Ohio State Univ., Columbus, Apr. 1977, 17.

This bibliography is divided into two parts. The first set of references listed is from work done at the Disaster Research Center at the Ohio State Univ. The subdivisions in this listing are (1) Books, Monographs and Reports, and (2) Articles. The second set of references, selectively chosen from the literature, is from work done at other than the Disaster Research Center. The subdivisions in this listing are (1) Books, Monographs, and Reports, and (2) Articles. Items have been listed on the basis of their general availability to the public and their direct relevance to disaster research and disaster planning; almost all are of relatively recent date except for some older works that are still of value. This bibliography concludes with a listing of other bibliographies, journals in the disaster area, and newsletters on disaster topics.

● **9.1-7** Blake, J., **Emergency planning guide for federal departments in B.C.,** *Emergency Planning Digest,* **4,** *3,* May-June 1977, 2–6.

This article is based on a paper prepared for Emergency Planning Canada regional directors at a national seminar. The original document was accepted by other research directors in British Columbia as a guide to regional provincial emergency planning. The purpose of the guidelines is twofold: first, to assist in the development of plans by departments to provide for rapid federal response to emergencies in British Columbia and, second, to advise on federal departmental responses to specific emergencies by establishing a total response structure. This includes the detailing, marshalling, and deployment of resources (including manpower) to ensure effective action by the federal government should assistance be required by other levels of government.

● **9.1-8** de Ville de Goyet, C. and Jeannee, E., **Earthquake in Guatemala: epidemiological evaluation of the relief effort,** *Emergency Planning Digest,* **4,** *1,* Jan.-Feb. 1977, 2–8.

On Feb. 4, 1976, the most populated area of Guatemala experienced the first of a series of earth tremors numbering over 1500. The main shock measured 7.5 on the Richter scale. Destroyed villages and small cities were isolated by numerous landslides delaying relief teams and preventing full knowledge of the extent of the disaster. In Santa Apolinia (population 4000), mortality rates reached 21.5%; in the region of Chimaltenango (population 194,-735), the death rate was 7.1%. The National Emergency Committee estimated that 76,504 persons were injured. This article discusses the problems associated with disaster relief operations and proposes plans for improvement.

● **9.1-9** Taleb-Agha, G., **Seismic risk analysis of lifeline networks,** *Bulletin of the Seismological Society of America,* **67,** *6,* Dec. 1977, 1625–1645.

The aim of this study is to show how seismic risk analysis of lifeline networks can be performed efficiently without loss of accuracy. Two efficient schemes have been developed to find the probabilities of partial network failure due to threats coming from a given set of seismically active source areas and/or faults with known seismic history. Networks with deterministic link resistances and those with random link resistances can be analyzed. Both schemes start with the transformation of the given network and its objective to an equivalent series systems in parallel network which is simpler to handle and has a certain number of tie-sets. Then, the probability that any desired number of these tie-sets fail simultaneously in any given

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year is evaluated. An efficient computer program for the deterministic resistance approach has been developed and tested on several pilot problems. The network of major highways surrounding the Boston, Massachusetts area is analyzed, and the results are presented.

● **9.1-10** Movassate, M. and Shah, H. C., **Network seismic risk,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 750–755.

Studies of recent seismic events have shown that earthquake engineering has to be strengthened in the area of public utility systems, highway systems, and lifeline systems in general. Structural earthquake engineering, having started after the 1933 Long Beach earthquake, has reached a certain level of sophistication. However, it has not been until recently, after the San Fernando earthquake in 1971, that attention was brought to the vulnerability of lifeline systems to strong earthquakes. In computing the reliability of a lifeline system, it is impossible to treat the lifeline in its entirety. There are two reasons: first, the effect of an earthquake is not the same over the entire system, and second, the resistance of the lifeline usually varies from point to point along the network. Consequently, it is necessary to divide the system into links and nodes and then analyze the reliability of the network in terms of these elements.

● **9.1-11** Yano, K., Teramoto, T. and Umemura, H., **Aseismatic design of buildings in the Refuge Center,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. II, 1977, 1807–1812.

Tokyo's Downtown Refuge Center Project is being developed as part of an extensive disaster prevention program operated by the metropolitan government. East Shirahige Refuge Center (ESRC), the construction of which is now under way, marks the first stage of the project, which upon completion will comprise six such refuge centers. ESRC's 13-story buildings are planned to serve as barriers to prevent the spread of fire and to create a safe evacuation area. This paper describes the earthquake-resistant design of these buildings.

● **9.1-12** Oliver-Smith, A., **Disaster rehabilitation and social change in Yungaỳ, Peru,** *Human Organization,* **36,** *1,* 1977, 5–13.

Social adjustment following a major disaster is discussed in light of the May 31, 1970, Peruvian earthquake. The earthquake, with a Richter magnitude of 7.7, resulted in the loss of approximately 70,000 lives, injuries to about 50,000 people, and destruction of or damage to approximately 80% of all structures in the area. An avalanche caused by the earthquake totally destroyed the town of Yungay, and of its 4,000 inhabitants, only about 250 survived. Relief efforts and their effect on this town are analyzed.

● **9.1-13** Manning, D. H., **Disaster technology: an annotated bibliography,** Pergamon Press, Oxford, England, 1976, 282.

The purpose of this bibliography is to supply relief agencies with information on the published and unpublished literature available concerning technical aspects of disaster relief and prevention, with special emphasis on developing countries. It is also intended to provide those involved in research with information on disaster topics from other disciplines. The volume is limited almost entirely to the English language literature. The references are divided into the following sections: (1) relief organization, (2) medical aspects, planning, (3) medical aspects, general, (4) medical aspects, nutrition, (5) sociological aspects, (6) physical aspects, general, and (7) physical aspects, earthquakes.

● **9.1-14** Haas, J. E., Kates, R. W. and Bowden, M. J., eds., **Reconstruction following disaster,** MIT Press, Cambridge, Massachusetts, 1977, 331.

The contents are as follows: Chapter 1—from rubble to monument: the pace of reconstruction. Chapter 2—reconstruction issues in perspective. Chapter 3—reestablishing homes and jobs: cities. Chapter 4—reestablishing homes and jobs: families. Chapter 5—alternative pasts and futures. Chapter 6—major insights: a summary and recommendations.

9.2 Legal and Governmental Aspects

● **9.2-1** Stewart, R. M., Hart, E. W. and Amimoto, P. Y., **The review process and the adequacy of geologic reports,** *California Geology,* **30,** *10,* Oct. 1977, 224–229. (Reprinted from *Bulletin of the International Association of Engineering Geology, 14,* 1976.)

Several legislative statutes in California require either the preparation of a geologic report or an environmental impact report prior to approval of specified types of development. Compliance with the intent of statutes governing construction of hospitals, other structures in hazardous fault zones, or projects that will have a significant environmental impact is discussed. Analysis of reviews of these reports indicates that initial reports tend to be inadequate for their intended purposes and improvement is noted only after a review and feedback process. Report inadequacies led the California Div. of Mines and Geology to develop and distribute guidelines to assist in report preparation and checksheets to further aid in the review and feedback process.

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● 9.2-2 Campbell, I., **The influence of geologic hazards on legislation in California**, *California Geology*, 30, *10*, Oct. 1977, 219–223. (Reprinted from *Bulletin of the International Association of Engineering Geology*, *14*, 1976.)

Legislative response to California's geologic hazards, particularly earthquakes, dam failures, and landslides, is discussed in terms of specific motivating events. Failure of the St. Francis dam in 1928 demonstrated the need for geologic evaluation of foundation conditions. The 1933 Long Beach earthquake, which damaged numerous school buildings, resulted in passage of the Field Act, requiring that all new school buildings embody aseismic design. Not until after the San Fernando earthquake of 1971 were new hospital buildings included in similar legislation. The 1972 establishment of a Joint Committee on Seismic Safety has led to advanced planning and more sophisticated legislation, requiring, for example, that California's State Geologist issue maps delineating active faults, and requiring geologic reports before any development in designated sections. California has become the first state to require licensing of geologists under a State Registration Board of Geologists. Thus California has reached a point where it is anticipating geologic hazards rather than waiting for disaster to bring response.

● 9.2-3 Slosson, J. E. and Krohn, J. P., **Effective building codes**, *California Geology*, 30, *6*, June 1977, 136–139.

Most building sites are subject to at least one type of natural hazard. Prevention of building damage or loss and concomitant injury or loss of life can be effected by adequate building codes. The 1906 San Francisco earthquake and the 1925 Santa Barbara earthquake were instrumental in bringing about California's first written building code. After the 1933 Long Beach earthquake, the Field Act and the Riley Act mandated earthquake-resistant schools and commercial buildings, respectively. The 1971 San Fernando earthquake demonstrated the effectiveness of building codes in reducing loss of life. Only 58 people died as a result of this earthquake whereas earthquakes of similar magnitude in other parts of the world have resulted in much greater losses of life.

Damage to a few schools during the 1971 San Fernando earthquake brought about passage of California Senate Bill No. 479, requiring geological and engineering studies to ensure that a school will not be built on the trace of an active or potentially active fault or on ground subject to slippage or liquefaction. California law now requires a seismic safety element to be incorporated in the general plan, or master plan, for a city or county. Other California legislation dealing with seismic safety includes 1972 Senate Bills 519 and 520. The former requires that detailed geologic-seismic and foundation engineering studies be completed prior to site selection and design of hospitals. Senate Bill 520 required the California Div. of Mines and Geology to delineate special studies zones to encompass all active and potentially active traces of the San Andreas, Calaveras, Hayward, and San Jacinto faults and any other faults which may be hazardous. Besides earthquakes, other geologic hazards in California include floods and landslides. Both have influenced the state's building codes.

● 9.2-4 Pate, E., Linville, W. and Shah, H. C., **Public policy in earthquake effects mitigation: earthquake prediction and earthquake engineering**, *Proceedings, Sixth World Conference on Earthquake Engineering*, Sarita Prakashan, Meerut, India, Vol. I, 1977, 695–701.

A model is developed to determine how earthquake prediction and enforcement of building code regulations can reduce seismic risk and at what cost. The measure of risk is the expected loss per year for a given region. This measure has two components: expected life loss and expected property damage. A flow chart and a detailed discussion of probabilistically assessed measures are presented.

● 9.2-5 Amimoto, P. Y., **Environmental impact reports**, *California Geology*, 30, *1*, Jan. 1977, 12–15.

Since the passage of the California Environmental Quality Act of 1970, the California Div. of Mines and Geology (CDMG) has been responsible for review of geologic analyses in environmental impact reports (EIRs). Through use of case histories, the article assesses the adequacy of the geologic analyses of two EIRs involving an electric power generating facility and a proposed residential development, both in seismically active areas. Included is CDMG Note 46: "Guidelines for Geologic/Seismic Considerations in Environmental Impact Reports."

● 9.2-6 Traer, J. M., comp., **Summary of 1975–1976 legislation**, *California Geology*, 30, *2*, Feb. 1977, 37–39.

California legislation summarized falls into three categories: (1) geologic hazards and seismic safety; (2) mining, minerals, and energy resources; and (3) California Div. of Mines and Geology administration. Bills introduced to comply with the Field Act (regarding earthquake-resistant schools); the Alquist-Priolo Special Studies Zones Act; and the Subdivision Map Act are included. Also summarized are bills regarding the strong-motion instrumentation program and the seismic safety of historic buildings.

● 9.2-7 Spangle, W. and Assoc. *et al.*, **Earth-science information in land-use planning-guidelines for earth scientists and planners**, *Circular 721*, U.S. Geological Survey, Arlington, Virginia, 1976, 33.

The results from a nationwide sampling of applications of earth-science information to urban land-use planning are reported. To identify and evaluate the needs and

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problems of planners and earth scientists, the authors examined earth-science applications in several urban regions in different parts of the United States. Although the problems, methods, and political structure differed in each region, many needs and problems that were found frequently appeared to be general in nature and of fundamental importance to communication between planners and scientists.

The report is a summary and is structured as a series of recommendations that are based on current methods and practices in those parts of the United States where earth-science information has been used successfully in planning. These recommendations are offered not as a set of directives, but as guidelines that provide a starting point for planners and earth scientists who wish to work together more effectively.

9.3 Socio-Economic Aspects

● 9.3-1 **Guatemala '76, earthquakes of the Caribbean plate,** Munchener Ruckversicherungs-Gesellschaft, Munich, 1976, 48.

The contents are as follows: Seismic activity and volcanism of the Caribbean plate; Exposure of the Caribbean plate to seismic and volcanic activity; Managua-1972 and today; Earthquakes in Guatemala; Constructional aspects; The insurance of the earthquake risk.

● 9.3-2 Haas, J. E. and Mileti, D. S., **Socioeconomic impact of earthquake prediction on government, business, and community,** *California Geology,* **30,** *7,* July 1977, 147–157.

The Univ. of Colorado's Inst. of Behavioral Science has researched the probable consequences of scientifically based earthquake prediction with the help of an advisory committee made up of personnel from California local and state agencies, business firms, and the general community. Solution of some of the complex issues raised by the capability of predicting earthquakes well in advance will take at least several years. A surface uplift along the San Andreas fault near Palmdale, California, has led to the designation of the area as a possible site for a major earthquake in the near future. A large number of monitoring instruments have been placed in the area. Several types of anomalies in the earth's crust apparently precede earthquakes, with greater lead time for larger earthquakes. Seismologists and government agencies have considered how best to review tentatively formed predictions. Longer lead times can bring about not only greater preparedness but also greater economic and social disruption. The study identifies likely responses of government agencies, business firms, and individuals to two possible earthquake prediction scenarios.

● 9.3-3 Sullivan, R., Mustart, D. A. and Galehouse, J. S., **Living in earthquake country: a survey of residents living along the San Andreas fault,** *California Geology,* **30,** *1,* Jan. 1977, 3–8.

In 1970, a survey of residents of the San Francisco peninsula who lived in the area of the San Andreas fault zone was initiated. The survey was conducted by students enrolled in a class at San Francisco State Univ. The objective was to evaluate the awareness of, and attitudes toward, earthquake hazards in the densely populated area. As government agencies and the news media focused increasing attention on the potential destructiveness of a major earthquake, a second study objective became the evaluation of any effect that this publicity had in changing public awareness and attitudes. Therefore, additional surveys were made in 1972, 1974, and 1976 in the same residential area.

● 9.3-4 Steinbrugge, K. V., Lagorio, H. J. and Algermissen, S. T., **Earthquake losses as a function of construction types,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 678–685.

Government agencies and insurance companies must realistically estimate life hazards and potential aggregate property damage along with the geographic distributions for destructive earthquakes in any relevant region. Often it is necessary to accomplish this by a rapid and economic methodology in order to quickly resolve specific planning problems. Recent studies have produced several practical procedures which have suited a few of these needs. This paper discusses some of these developments regarding aggregate property damage. Government-sponsored studies have improved methods for establishing aggregate total losses for maximum credible events and percentage losses as a function of construction types. Methodology and typical results are given.

● 9.3-5 Ohta, Y. and Omote, S., **An investigation into human psychology and behaviour during an earthquake,** *Proceedings, Sixth World Conference on Earthquake Engineering,* Sarita Prakashan, Meerut, India, Vol. I, 1977, 702–708.

Human psychology and behavior patterns in relation to seismic intensities were investigated by means of questionnaire surveys. Attention was given to general behavior and mental attitudes, as well as to more specific behavior, such as driving a car and dealing with a fire source. The intensity range covered by these surveys was II-V (Intensity Scale, Japan Meteorological Agency). Samples of survey questions, graphs, and a discussion of the results are presented. Although the survey is incomplete because of a

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shortage of data, the authors believe that the results will eventually be valuable in the formulation of adequate disaster prevention programs and plan to continue their investigations.

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