

Proceedings of the Seventh World Conference on Earthquake Engineering

VOLUME 5

Structural Aspects

Part 2



September 8-13, 1980

10171

86.2/8
E115

PROCEEDINGS OF THE
SEVENTH WORLD
CONFERENCE ON
EARTHQUAKE ENGINEERING

September 8-13, 1980
Istanbul, Turkey



VOLUME

5

STRUCTURAL ASPECTS PART II

METHODS OF DYNAMIC ANALYSIS

5/8/86
1498/104

TABLE OF CONTENTS

EARTHQUAKE ANALYSIS OF COUPLED SHEAR WALL BUILDINGS	1
T. Srichatrapimuk-Anil K. Chopra	
BEHAVIOR OF FLAT PLATE FLOOR SYSTEMS UNDER IN-PLANE SEISMIC LOADING	9
H.Faruk Karadogan, Ti Huang, Le-Wu Lu, Masayoshi Nakashima	
DYNAMIC BEHAVIOR OF FLEXIBLE STRUCTURES WITH VIBRATION ABSORBER	17
T. Takahashi-S.Kuranishi	
OPTIMUM RIGIDITY DISTRIBUTION OF ASEISMIC DESIGN FOR MULTI- STORY SHEAR FRAMES	25
Wang Guang-yuan, Dong Ming-yao, Guo-Hua	
EFFECT OF AXIAL DEFORMATION OF COLUMNS ON DYNAMIC BEHAVIOR OF REINFORCED CONCRETE BUILDING FRAMES	33
Koji Yoshimura-Kenji Kikuchi	
A SIMULATION OF EARTHQUAKE RESPONSE OF REINFORCED CONCRETE BUILDING FRAMES TO BIDIRECTIONAL GROUND MOTION BY IIS COMPUTER ACTUATOR ON LINE SYSTEM	41
Tsuneo Okada, Matsutaro Seki-Yung J. Park	
DYNAMIC RESPONSE OF REINFORCED CONCRETE WALL SYSTEMS	49
M. Saatcioglu, T. Takayanagi, - A.T. Derecho	
DYNAMIC PROPERTIES AND SEISMIC RESPONSE OF FRAMED WALLS AND PANEL BUILDINGS	57
V. T. Rasskazovsky, U.Sh. Shamsiyev, R.N. Musheyev, L.J. Babich	
DYNAMICS OF REINFORCED CONCRETE FRAMES SUBJECT TO HORIZONTAL AND VERTICAL GROUND MOTION	65
Sukenobu Tani - Satsuya Soda	
EARTHQUAKE RESPONSE ANALYSIS OF 19 STOREY PREFABRICATED REINFORCED CONCRETE FRAME STRUCTURE	73
Wang Kai-shun, Wang Wen-zhen	
EARTHQUAKE FAILURE CRITERIA OF DETERIORATING HYSTERETIC STRUCTURES	81
Hirokazu Iemura	
RESEARCH ON ASEISMIC DESIGN OF STEEL TOWER CONSTRUCTED ON THE TOP OF BUILDING	89
H. Yoshida, H. Takane, N. Adachi, H. Koshida-M. Iizuka	
DAMPING OF A SOIL-STRUCTURE SYSTEM	97
P. Sotirov	

A CONTRIBUTION TO THEORETICAL PREDICTION OF DYNAMIC STIFFNESS OF SURFACE FOUNDATIONS	105
H. Tajimi	
THE EFFECTS OF LARGE DISPLACEMENTS ON THE EARTHQUAKE RESPONSE OF TALL STRUCTURES	113
Athol J. Carr - Peter J. Moss	
EARTHQUAKE RESPONSE OF REINFORCED CONCRETE FRAME-CRACKED SHEAR WALL SYSTEMS	117
Dr. Celâl N. Köstem - Gilberto Areiza	
ON THE DAMAGE AND CAUTION TO WOODEN HOUSES BY RECENT EARTHQUAKES IN JAPAN	121
Gorozo Iizuka	
STUDY ON EVALUATION OF PARAMETERS IN THE SOIL-BUILDING SYSTEM	125
Y. Kitagawa, Y. Irie, - S. Tamori	
A SIMPLIFIED ANALYSIS METHOD ON INTERACTION SYSTEM ABOUT STRUCTURE-FOUNDATION-SOIL SURFACE LAYER	133
Yuzuru Yasui	
COMPARISON OF ANALYSIS METHOD FOR SEISMIC SOIL-STRUCTURE INTERACTION	141
Wayne W. H. Chen , M. Chatterjee, A. Unemori	
EFFECTS OF STRUCTURE-SOIL-STRUCTURE INTERACTION DURING VARIOUS EXCITATIONS	149
Hatsukazu Mizuno	
RESEARCH ON THE DYNAMIC CHARACTERISTICS OF THE FOUNDATION BY THE USE OF NUMERICAL INTEGRAL EXPRESSIONS FOR THE WAVE EQUATIONS	157
Isao Toriumi, Akira Kowada, Toshiharu Hisatoku Teruo Segawa	
LAYERED SOIL-PILE-STRUCTURE DYNAMIC INTERACTION	165
Yoshikazu Yamada, Hirokazu Takemiya-Shigeru Noda	
SEISMIC STABILITY OF EMBEDDED TANK	173
Takahiro Iwatate, Takeji Kokusho-Satoshi Ooaku	
STUDY ON FILTERING EFFECT OF FOUNDATION SLAB BASED ON OBSERVATIONAL RECORDS	181
Kiyoto Shioya - Hiroshi Yamahara	
DYNAMIC SOIL-STRUCTURE INTERACTION UNDER A TOPOGRAPHICAL SITE CONDITION	189
Takuji Kobori-Yuzo Shinozaki	

EVALUATION OF DYNAMIC STIFFNESS OF EMBEDDED FOUNDATION USING DYNAMIC REACTION OF SURFACE LAYER	197
Takanori Harada-Keizaburo Kubo	
SEISMIC ANALYSIS OF STRUCTURES WITH RECTANGULAR BASES ON POROUS ELASTIC SOIL MEDIUM	205
Shoon-rong Kao, Theodore G. Toridis, - K. Khozeimeh	
SEPARATION AND SLIDING BETWEEN SOIL AND STRUCTURE TO STRONG EARTHQUAKE MOTION	213
Kenzo Toki, Tadanobu Sato-Fusanori Miura	
SEISMIC RESPONSE OF SUBSTRUCTURES NONLINEARLY COUPLED WITH SOIL	221
Takashi Akiyoshi	
EFFECTS OF FOUNDATION EMBEDMENT ON SOIL-STRUCTURE INTERACTION	225
V. W. Lee	
STUDY ON SOIL-PILE INTERACTION OF PILE GROUPS IN VERTICAL VIBRATION	229
Yoshihisa Gyoten, Tadahiro Fukusumi, Takahito Inoue, Toshiki Mizuno	
SOIL STRUCTURE INTERACTION RESPONSE OF COMPLEX INDUSTRIAL STRUCTURES	233
Satyendra P. Gupta-Mani Kant Gupta	
ON THE ACCURACY OF NUMERICAL MODELS IN 3-D SOIL-STRUCTURE INTERACTION	237
G. Solari, D. Stura, C. Vardanega	
EARTHQUAKE RESPONSE ANALYSIS OF SHELL-TYPE FIXED OFFSHORE TOWERS BY MATRIX PROGRESSION METHOD	245
Y. Tanaka, T. Hamamoto, - K. Konno	
THE DYNAMIC BEHAVIOUR OF A PLANT TOWER CONSIDERING SOIL EFFECT IN LARGE STRAIN LEVEL	253
Kenji Hagio, Akio Ikeda-Souichi Kawamura	
EARTHQUAKE ANALYSIS OF 3D STRUCTURES WITH FLEXIBLE FLOORS	261
H. Faruk Karadoğan	
ELASTO-PLASTIC EARTHQUAKE RESPONSE ANALYSIS FOR MULTI- STOREYED BUILDINGS TAKING ACCOUNT OF TORSION	269
Wei Lian Zhu Jingxin Jiang Zhili	
ANALYSIS OF SEISMIC RESPONSE OF CYLINDRICAL SHELLS WITH IMPERFECT CIRCULAR SECTION CONTAINING LIQUID	277
Sukenobu Tani, Naohito Hori	
SEISMIC LOADS ON SUSPENDED SYSTEM	285
Wang Qianxin, Lu Shuhui	

EQUIVALENT INTERACTION MODAL DAMPING	292
T. Selcuk Atalik	
A STUDY REGARDING DYNAMIC ANALYSIS OF STRUCTURES SUPPORTED BY WALL FOUNDATION SYSTEM	298
Wes Study Group	
TWO-DIMENSIONAL INELASTIC SEISMIC RESPONSE OF STEEL FRAMES INCLUDING AXIAL DEFORMATION OF COLUMNS	307
Motoo Saisho	
IMPULSIVE PRESSURE ACTING ON THE TANK ROOFS CAUSED BY SLOSHING LIQUID	315
Nobuyuki Kobayashi	
SEISMIC DESIGN OF STRUCTURAL WALL BUILDINGS AND INELASTIC RESPONSE ANALYSIS	323
Boris Simeonov	
THEORETICAL INVESTIGATIONS UPON ANALYTICAL MODELS REGARDING DYNAMIC INTERACTION BETWEEN SOIL AND METRO-TYPE STRUCTURES	327
Gh. Palamaru, N. Ungureanu, I. Ciongradi	
STUDIES ON EARTHQUAKE RESPONSES OF GROUPED UNDERGROUND TANKS IN SOFT GROUND	331
Yozo Goto - Takeshi Shirasuna	
A METHOD FOR THE DYNAMIC ANALYSIS OF EMBEDDED STRUCTURES	335
A. Aydin Dumanoglu	
THE USE OF INTERPOLATION FUNCTIONS IN RESPONSE CALCULATIONS	339
Muzaffer Ipek	
COMPUTATION OF SEISMIC RESPONSE OF LIGHT ATTACHMENTS TO BUILDINGS	343
Nathan M. Newmark - Roberto Villaverde	
ULTIMATE STRESSES IN LARGE-PANEL BUILDINGS EXPOSED TO SEISMIC LOADS	351
G. A. Shapiro, G. N. Ashkinadze	
EFFECTS OF INITIAL OUT-OF-ROUNDNESS ON SEISMIC RESPONSE OF CYLINDRICAL LIQUID STORAGE TANKS	359
A. S. Veletsos - J. W. Turner.	
THE RESPONSE OF STRUCTURES TO TRAVELING BODY AND SURFACE WAVES	367
S. D. Werner - L. C. Lee	
DYNAMIC CHARACTERISTICS OF NONSYMMETRIC BUILDING STRUCTURES	375
Riko Rosman	

VIBRATIONAL CHARACTERISTICS AND EARTHQUAKE RESPONSES OF THE ASYMMETRIC MULTISTORY BUILDINGS	383
Joji Sakurai	
THREE DIMENSIONAL INELASTIC BUILDING RESPONSE TO SEISMIC LOADING	391
A. G. Gillies - R. Shepherd	
GENERALIZED MUTO METHOD FOR STRUCTURES COMPOSED OF NON- ORTHOGONAL FRAMES	399
Adnan Çakiroğlu - Günay Özmen	
SYNTHESIS OF RESPONSES OF TWO SUBSYSTEMS FOR COUPLED SEISMIC ANALYSIS OF EQUIPMENT MOUNTED ON A STRUCTURE	407
Erol Varoğlu - W. D. Liam Finn	
SOME ANALYTICAL MODELS IN THE EARTHQUAKE ENGINEERING AND SOME WAYS OF REDUCING THE SEISMIC LOAD	415
A. T. Stohl, Yu. P. Nazarov.	
INFLUENCE OF HYSTERESIS MODELS ON CALCULATED SEISMIC RESPONSE OF R/C FRAMES	423
Mehdi Saïdi	
PEAK SEISMIC RESPONSE CHARACTERISTICS OF SECONDARY SYSTEMS	431
M. P. Singh	
INELASTIC RESPONSE HISTORY ANALYSES OF EARTHQUAKE RESISTANT BUILDING STRUCTURES WITH YIELDING WALLS AND ELASTIC FRAMES	435
Mark Fintel - S. K. Ghosh	
DYNAMIC RESPONSE OF STEEL SPACE FRAMES UNDER EARTHQUAKE EXCITATION IN HORIZONTAL ARBITRARY DIRECTION	439
Shosuke Morino, Yasuhiro Uchida	
EARTHQUAKE RESPONSE ANALYSIS OF TORSIONALLY COUPLED MULTISTORY BUILDINGS	443
Mario di Paola	
DYNAMIC ANALYSIS FOR RECTANGULAR WATER TANKS	447
C. Minowa	
MODAL ANALYSIS OF NONLINEAR SYSTEMS	451
Taugir M. Sheikh, Jose M. Roesset, C.P. Johnson	
SUBSTRUCTURE METHOD FOR EARTHQUAKE RESPONSE OF HIGH- ELEVATED MULTI-SPAN CONTINUOUS BRIDGE	459
Hirokazu Takemiya - Kenji Kawano	
EARTHQUAKE RESPONSE OF SYSTEMS WITH NONPROPORTIONAL DAMPING BY THE CONVENTIONAL RESPONSE SPECTRUM METHOD	467
Roberto Villaverde	

DAMPING CHARACTERISTICS OF R.C. HIGHRISE CHIMNEY BASED ON EARTHQUAKE MOTION MEASUREMENT	475
Koji Kitazawa, Akio Ikeda - Kenji Hagio	
A SEMI-ANALYTIC F.E. MODEL FOR INERTIAL INTERACTION ANALYSIS	483
F. Venancio Filho - S. Hampshire	
INELASTIC ANALYSIS OF EARTHQUAKE RESPONSES OF SINGLE- STORY FACTORY BUILDINGS TAKING ACCOUNT OF TORSIONAL VIBRATION	490
Zhu Bolong - Lu Weimin	
3- DIMENSIONAL STUDY OF NONLINEAR BEHAVIOR OF REINFORCED CONCRETE COLUMN UNDER REPEATED LATERAL FORCES	497
Tadashi Sugano, Takashi Miyashita, Norio Inoue	
EFFECT OF STIFFNESS DEGRADATION ON SEISMIC RESPONSE OF CONCRETE FRAMES	505
Jagmohan Humar	
VIBRATIONAL CHARACTERISTICS OF SHALLOW CYLINDRICAL TANKS	513
Celâl N. Köstem - Joseph W. Tedesco	
ELASTO-PLASTIC RESPONSE ANALYSIS OF SEISMIC-RESISTANT FRAMES VIA MATHEMATICAL PROGRAMMING	521
F. Casciati, L. Faravelli - A. Gobetti	
COMPUTED BEHAVIOR OF A TEN-STORY REINFORCED CONCRETE FRAME- WALL STRUCTURE FOR STATIC LATERAL LOADS	529
Katshuhiko Emori, - W. C. Schnobrich	
APPLICATION OF DIFFERENT SPECTRAL CHARACTERISTICS AT THE SEISMIC RESPONSE ANALYSIS OF STRUCTURES	537
Emilia Juhasova	
EARTHQUAKE RESPONSE ANALYSIS OF SEVERAL KINDS OF STRUCTURES WITH REALISTIC RESTORING FORCE CHARACTERISTICS	545
Koji Mizuhata, Naoya Harada Tamotsu Tsuda, Masahiko Yamaoka	
INELASTIC RESPONSE OF BUILDING STRUCTURES BY TRAVELLING EARTHQUAKE WAVE MOTIONS	553
Yutaka Inoue - Masahiro Kawano	
SIMPLIFIED METHOD FOR COMPUTATION OF EARTHQUAKE INDUCED SHEARS AND OVERTURNING MOMENTS IN REGUIAR MULTISTOREY STRUCTURES	561
Peter Fajfar - Stefan Strojnik	
USE OF DYNAMIC ANALYSIS IN SEISMIC DESIGN	565
Jagmahan Humar	

THEORETICAL CONSIDERATION ON OVERTURNING OF BODIES BY EARTHQUAKE MOTIONS AND FIELD SURVEY OF 1978 OFF MIYAGI PREFECTURE EARTHQUAKE	569
Yuji Ishiyama	
PROBLEMS IN THE USE OF ROOT-SUM-SQUARE SOLUTIONS FOR THREE-DIMENSIONAL DYNAMIC ANALYSIS OF BUILDINGS.....	573
J. C. Jeing	
FREE VIBRATION OF ARCHES WITH AXIAL LOADING	577
Ali Aydin.	
ELASTO-PLASTIC DYNAMIC RESPONSE ANALYSIS OF FLEXURAL- SHEAR MODEL (AN APPRAISEMENT OF AN EXAMPLE OF THE APPLICATION OF PLANE-FRAME)	581
Mitsugu Asano - Takayuki Teramoto	
THE STOCHASTIC FINITE - ELEMENT METHOD	585
Humberto Contreras	
EARTHQUAKE RESPONSE ANALYSIS OF MULTI-STORY R.C. FRAMED BUILDINGS	593
Yin Zhigian, Li Shuzhen	
THREE DIMENSIONAL ANALYSIS OF 110 MW BOILER SUPPORTING STRUCTURE SUBJECTED TO SEISMIC EXCITATION	601
C. Kameswara Rao, K. Ramadasa, A. K. Singh-K.G. Bhatia	
QUANTITATIVE SEISMIC HAZARD ASSESSMENT FOR SPATIALLY DISTRIBUTED SYSTEMS	609
Hector Monzon - Despang	
NONLINEAR DYNAMIC ANALYSIS OF STRUCTURES USING VISCOPLASTIC MATERIAL MODEL	617
S. K. Thakkar	
SEISMIC RESPONSE OF MULTI-SUPPORT STRUCTURES	625
L. Esteva, S. E. Ruiz, A. Reyes	
SEISMIC ANALYSIS OF STRUCTURES WITH MASONRY WALLS	633
Enrique Bazan - Roberto Meli	
SEISMIC BEHAVIOUR OF PRECAST PANEL SHEAR WALLS	641
O. A. Pekau - H.P. Huttelmaier	
NONLINEAR EARTHQUAKE RESPONSE OF REINFORCED CONCRETE BUILDING FRAMES BY THE EQUIVALENT LINEAR METHOD	649
Matsutaro Seki - Tsuneo Okada	
INELASTIC RESPONSE BEHAVIOR OF TORSION IN BUILDINGS SUBJECTED TO STRONG EARTHQUAKE	657
Takao Nishikawa	
SPATIAL AND MODAL COMBINATIONS OF DYNAMIC RESPONSE FOR DESIGN OF FIXED OFFSHORE PLATFORMS UNDER THREE COMPONENTS OF EARTHQUAKE MOTIONS	665
Stavros A. Anagnostopoulos	

EARTHQUAKE RESISTANT ANALYSIS OF SINGLE STORY INDUSTRIAL BUILDING WITH THE CONSIDERATION OF THE SPACE BEHAVIOR	673
Gu Guo-ji, Fu Ping-jun	
EFFECTS OF DURATION AND AFTERSHOCKS ON INELASTIC DESIGN EARTHQUAKES	677
Stephen A. Mahin	
ESTIMATION OF FRAMEWORK-INFILL WALL INTERACTION FOR MULTISTOREY BUILDINGS SUBJECTED TO CODE SEISMIC LOADINGS	681
Camelia Ciongradi	
SEISMIC ANALYSIS OF A DOMED CYLINDRICAL LIQUID STORAGE TANK	685
William A. Nash, T. Balendra, S. H. Shaaban	
SHEARWALL FRAME INTERACTION UNDER EARTHQUAKE MOTION	689
A. S. Arya, S. K. Thakkar - D.B.S. Sarawat.	
DYNAMIC ANALYSIS OF ORTHOGONAL PLANAR FRAMES BY STORY TRANSFER MATRIX	693
E. Çitipitioglu, R.B. Renda	

EARTHQUAKE ANALYSIS OF COUPLED SHEAR WALL BUILDINGS

by

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SUMMARY

An efficient technique especially suited for computer analysis of coupled shear wall buildings is outlined. Application of the technique to analysis of earthquake-damaged buildings is demonstrated.

INTRODUCTION

Coupled shear wall buildings have usually been analyzed by computer programs based on standard methods of building frame analysis. These analysis methods, in determining responses to horizontal ground motions, neglect inertia forces in vertical and rotational degrees of freedom. By static condensation of these degrees of freedom, the dynamic equations are formulated in terms of lateral displacement and the problem size is greatly reduced. However, such a formulation is generally inappropriate for coupled shear wall buildings because vertical inertia effects of walls can be significant in the dynamics of such structures.

Because the stiffness, strength, and stability of coupled shear wall buildings is largely due to the walls, they should be designed to remain essentially undamaged in the event of an earthquake. The benefits of energy dissipation through inelastic action can be provided by yielding of the coupling beams. Therefore, this investigation assumes that the walls are linearly elastic, thus confining yielding to the coupling beams.

The objectives of this paper, which summarizes some results from the complete report¹ on this study, are: (1) to present an efficient technique especially suited for computer analysis of coupled shear walls; and (2) to demonstrate application of the technique to analysis of earthquake-damaged buildings.

OUTLINE OF ANALYTICAL PROCEDURE

The end shear wall of the building shown in Fig. 1 may be idealized as the structural assemblage shown in Fig. 2, with three wide-column lines (located at the respective neutral axes of the walls), and beams at every floor level coupling adjacent walls. The corner spandrel beams may be neglected; the wall-beam panel zone is idealized by rigid links.

Coupling beams in coupled shear wall buildings are usually deep and subject to high shear, resulting in shear cracking, normally accompanied by yielding of stirrups and flexural reinforcement.* Shear cracks and subsequent yielding are not localized at end sections of beams, but are

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spread over a substantial portion of the span. Therefore, instead of using the end sectional moment capacities, M_{y1} or M_{y2} , to signify a change in beam stiffness - as is done with flexural beams - the shear $P_v = (M_{y1} + M_{y2})/s$, which directly reflects nonlinearities due to diagonal shear cracking or stirrup yielding, is used to decide whether or not a change in stiffness has occurred. A bilinear hysteretic force-deformation relation is assumed for coupling beams, controlled by a bilinear shear-average end rotation relation (Fig. 3a) which can be obtained from laboratory experiments.² This model implies simultaneous changes in stiffness in the moment-rotation relation at both ends of the beam, reflecting nonlinear effects distributed throughout the beam span, not just at the ends (Fig. 3b).

By considering bending and axial deformations in walls; bending, axial, and shear deformations in coupling beams; including inertia forces associated with lateral as well as vertical motions; and using variational principles, the equations of motion were first formulated in the nodal point degrees of freedom (DOF): vertical, horizontal, and rotational displacements at each beam-wall (wide column) joint in Fig. 2. The number of these equations increases rapidly as the number of walls and stories increase and their solution requires large computational effort.

In Fig. 4, the first 10 natural vibration mode shapes computed for the McKinley Building³ are compared to the vibration mode shapes of the individual walls of the building. The general similarity of the two sets of mode shapes suggests that displacements of the structure may be effectively expressed as a linear combination of the natural mode shapes of vibration of individual walls. Thus, the displacements at the nodal points on the j th wall are expressed as a linear combination of the first few natural mode shapes of the j th wall, considered as an individual cantilever. Local plastic rotation at the base of a wall may be considered by including the associated rigid body displacement of the structure as an additional shape function. The equations of motion are transformed to the associated generalized coordinates. If a small number of generalized coordinates suffice to predict response accurately, the number of equations and the computational effort would be reduced considerably, as discussed in the next section.

Not only does the numerical step-by-step integration of the reduced system of equations require considerably less computational effort than does the original system, but a larger time step may be used in the integration, because the higher vibration modes, having very short vibration periods and contributing negligibly to structural response, are eliminated by the transformation to generalized coordinates.

EVALUATION OF REDUCTION TECHNIQUE

The simple idealization presented in Ref. 3 for the McKinley Building (Fig. 1) was employed to evaluate the effectiveness of the above-described technique for reducing the number of DOF. Coupling beams were assumed to span the two end walls and the middle pier was ignored. A reduced system of equations is designated by $H_m V_n$, where m and n denote the number of modes of lateral (horizontal) and longitudinal (vertical) vibration, respectively, of each wall included in the analysis. The natural frequencies and mode shapes of the coupled shear wall system, modal stress resultants, and the nonlinear response of the system are computed from the original system

("exact" analysis) of equations in nodal point coordinates, and from the reduced system of equations in generalized coordinates.

The first six natural frequencies and mode shapes of the structure were satisfactorily reproduced by the H_1V_4 system, whereas the first nine modes were more accurately reproduced by the H_6V_3 system (Fig. 5). Because the more significant displacements in the lower modes of vibration of the structure are in the lateral direction, it is effective to include a larger proportion of lateral vibration modes of the walls.

Although the deflected shape of the first antisymmetrical mode was very accurately reproduced by solving the eigenvalue problem for the H_6V_8 reduced system, to within 2%, predictions of the associated shear and bending moments in the walls were extremely inaccurate (Fig. 7). Stress resultants were inaccurate because the moments in the walls associated with the deformations in vibration modes of individual walls vary gradually along the height, whereas their actual distribution is discontinuous due to the moments at the ends of coupling beams. However, the predicted wall moment smoothly averaged the discontinuity in moments at the beam level. Shear in coupling beams, however, was predicted accurately (Fig. 7).

The stress resultants for the walls obtained by analyzing the reduced system were corrected by distributing, as shown in Fig. 6, the beam end-moments to the wall above and below each beam-wall joint. A correction was also necessary at the base of the structure. Shear forces in the wall are then correspondingly adjusted to equilibrate corrected bending moments. By applying the above adjustment procedure, the corrected bending moments and shears obtained from analyzing the H_6V_3 reduced system -- a much less refined system than the H_1V_8 one -- satisfactorily agreed with the "exact" values (Fig. 7). The H_6V_3 reduced system, with this adjustment, also satisfactorily predicts the stress resultants associated with the third antisymmetrical mode shape (Fig. 8).

Two approaches were used to determine the nonlinear response of the simple idealization for the Mt. McKinley building, wherein yielding of the coupling beams is considered, to a simple ground motion, described by a half-cycle of displacement, with maximum acceleration = 0.5 g in the horizontal direction and one-third of that in the vertical. The H_6V_3 reduced system was analyzed by the procedures outlined earlier and the equations in nodal point coordinates were solved by DRAIN-2D⁵, a computer program based on standard frame analysis procedures. The results (Figs. 9 and 10) indicate that the two analyses lead to essentially the same displacement response, but the forces determined from DRAIN-2D analysis oscillate about those determined from analysis of the reduced system. These oscillations do not disappear even when the integration time step in the DRAIN-2D analysis is reduced to half the value used in analysis of the reduced system. An operation count indicates that the computational effort required for analysis of the reduced system is 20% to 50% of that required for the original system in nodal point coordinates. Obviously, with the use of generalized coordinates, not only is the size of the problem and computational effort greatly reduced but, by eliminating the unimportant higher modes of vibration, the spurious oscillations in the numerical calculations are eliminated.

ANALYSIS OF EARTHQUAKE-DAMAGED BUILDINGS

Some of the damage to McKinley Building caused by the 1964 Alaska earthquake is apparent in Fig. 1. The response of the mathematical model of an end wall of the building (Fig. 2) to a simulated motion⁶, intended to represent the ground shaking in Anchorage, was determined by the procedure outlined earlier. The results are summarized in Figs. 11-13.

Beams on the second through eighth stories underwent the most extensive yielding, each of which accumulated a total plastic rotation of more than 0.02 radians during more than 20 yielding excursions (Fig. 11). Cyclic rotation ductility demand exceeded 10, an excessive demand for an ordinarily reinforced deep beam. The analysis thus predicted severe inelastic action in these beams which failed due to inadequate ductility. The prediction was generally consistent with the observed damage, beams from the second through the ninth stories having been severely damaged during the earthquake.

Axial force envelopes for walls (Fig. 12) indicate no resulting axial tension, and therefore no possibility of uplifting of the foundation or failure of walls in tension. A significant difference in the magnitude of developed axial compression in two identical walls gave rise to substantially different sectional moment capacities. Therefore, one of the two identical walls with smaller moment capacity was more vulnerable to yielding than the other wall; this is consistent with the observed damage.

Although yielding in walls was not considered in the analysis, it can be examined by studying the force distribution at selected time steps. For example, at $t = 16.9$ seconds, beams in the third through eighth stories had just undergone three large, consecutive yielding cycles. They were assumed to fail at this point and part of the resistance to story overturning moment, formerly offered by axial forces in the walls, was no longer available at these stories. Wall sections across affected stories had therefore to resist more moment to compensate for the loss of the axial-force couple. This additional moment was assumed to be resisted equally by the two outside walls. The resulting moment distribution is presented in Fig. 13, indicating that wall sections from the fourth story down were stressed beyond yielding capacity. Although actual redistribution of the couple due to axial forces in the walls after some beams have failed is much more complicated, this simple analysis of redistribution of moments indicates a yielding tendency in these lower story wall sections. In fact, yielding did occur in the third story wall section (Fig. 1).

A similar analytical investigation¹ of the performance of the Banco de America building during the Mangua earthquake led to conclusions consistent with the actual damage. Coupling beams underwent significant yielding but the walls were essentially undamaged. The excellent performance of this building suggests that, for coupled shear walls to be most effective as a structural system, walls should be designed to remain elastic, thus justifying the assumption of linearly elastic walls in this analytical procedure.

CONCLUSION

Under the assumption that inelastic action is confined to the coupling beams, coupled shear wall buildings can be most effectively analyzed by

expressing the deflections as a linear combination of the first few natural mode shapes in lateral (horizontal) and longitudinal (vertical) vibration of individual cantilever walls. In this approach, the vertical inertia, important in the dynamics of coupled shear walls, need not be neglected; and any mechanical model for the coupling beams can be employed. This analysis procedure requires considerably less computational effort than standard computer programs do. Using the technique presented earlier, results were given of earthquake response analyses of two existing coupled shear wall buildings damaged during earthquakes. It was shown that damage predictions based on analytical results are generally consistent with observed damage.

ACKNOWLEDGEMENT

This paper is based on a report¹ on research conducted under Grants GI-36387, AEN73-07732, and ENV76-04264 from the National Science Foundation.

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FIG. 1 MCKINLEY BUILDING:
EARTHQUAKE DAMAGE
IN NORTH END WALL

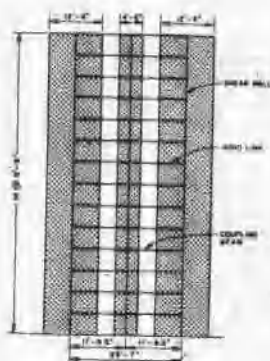


FIG. 2 MCKINLEY BUILDING:
STRUCTURAL IDEALIZATION
OF NORTH END WALL

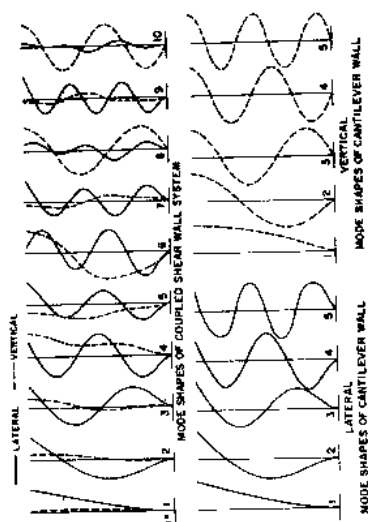


FIG. 4 STRUCTURAL MODE SHAPES AND CANTILEVER WALL MODE SHAPES

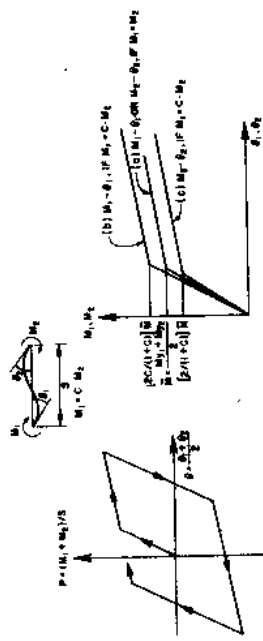


FIG. 3 MECHANICAL MODEL OF COUPLING BEAM

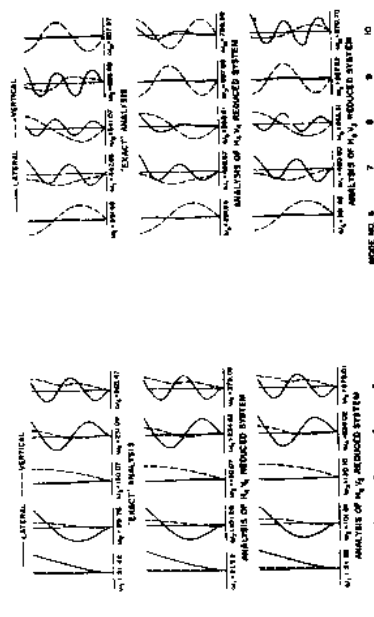


FIG. 5 NATURAL FREQUENCIES AND MODE SHAPES OF VIBRATION OF MCKINLEY BUILDING COMPUTED FROM THREE ANALYSES

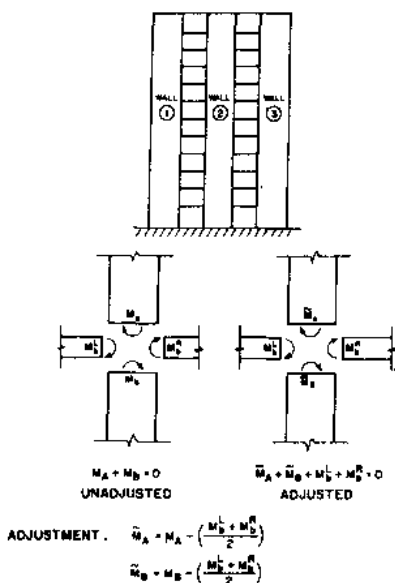


FIG. 6 ADJUSTMENT OF BENDING MOMENT IN WALL

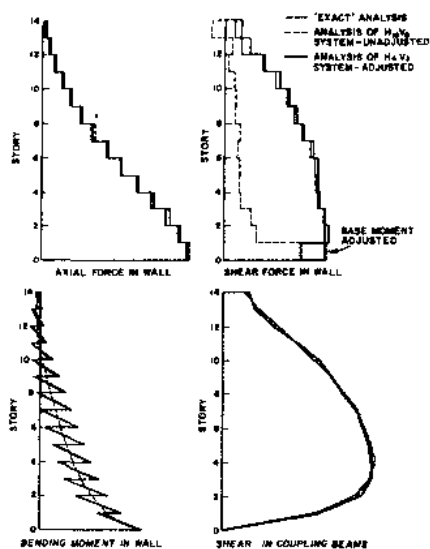


FIG. 7 STRESS RESULTANTS IN FIRST MODE OF VIBRATION OF MCKINLEY BUILDING

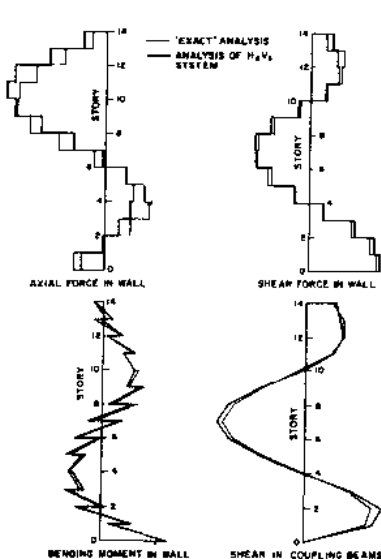


FIG. 8 STRESS RESULTANTS IN THIRD MODE OF VIBRATION OF MCKINLEY BUILDING

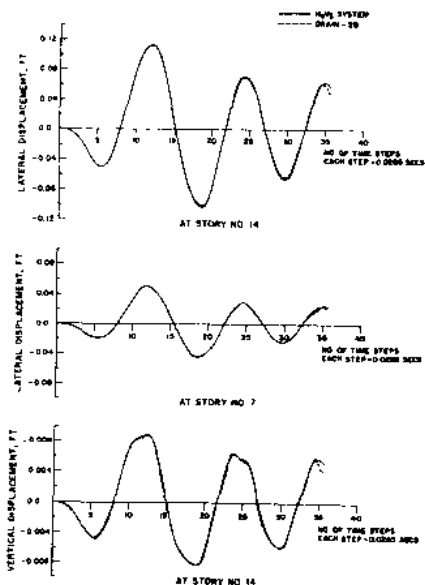


FIG. 9 DISPLACEMENT HISTORY OF WALL 1 OF MCKINLEY BUILDING