THESIS

FOR

NON-LINEAR ANALYSIS & DESIGN OF REINFORCED CONCRETE COLUMNS AND FRAMES

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NON-LINEAR ANALYSIS OF CONCRETE FRAMES USING A DIRECT STIFFNESS LINE ELEMENT APPROACH

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ABSTRACT: A computationally efficient method of non-linear analysis is described for reinforced concrete frames. The Crisfield arc-length solution procedure is used together with developed line element formulation which takes account of both the material nonlinearities and the geometric non-linear effects.
The resulting method has been found to be more reliable and more efficient than the other used methods which relied on load-control and displacement-control procedures.
Results obtained from the analysis of several frames are compared with test results reported in this thesis.

This thesis also describes the evaluation of system safety coefficients for non-linear design of reinforced concrete columns and frames using a back-calibration method. The back-calibration method was described in several previous papers, and was used to evaluate system safety coefficients for continuous concrete beams. Values for systems safety coefficients are proposed for ductile flexural system for systems in which collapse is due to concrete crushing and for system in which collapses by loss of stability. The study of slender frames has shown up an inadequacy in the simplified column design procedure of AS 3600 when applied to slender frames.

This thesis also describes the Correlation Study between Ground Motion Parameters and Damage during Kocaeli Earthquake as stated in section III.

SECTION I - Non-Linear Analysis of Concrete Frames Using a Direct Stiffness Line Element Approach

1 INTRODUCTION

Concrete structures exhibit distinctly non-linear behavior, especially at levels of loading close to collapse. Therefore, correct modeling of the material and geometric non-linearities is required if accurate estimates of behavior at working load and overload are required. The realistic analysis of concrete structures, taking account of non-linear and inelastic behavior, is tedious and time-consuming. However, over recent years, the increasing availability of high-capacity, low-cost computing facilities have encouraged the development of non-linear analysis procedures (Aas-Jakobson & Grenacher, 1974; Bazant, Pan & Pijaudier-Cabot, 1987; Wong, Yeo and Warner, 1988; Sun, Bradford and Gilbert, 1992, 1994; Kawano and Warner, 1995) for use with computers. This and further development of, and improvement to, non-linear analysis procedures in the near future will eventually bring about the introduction and wider use of an accurate non-linear methods of design (Warner, 1993).

Element type, solution procedure and the type of non-linearities which has been considered by various investigators are shown in Table 1 below.

Table 1: Summary of selected procedures for non-linear analysis

Investigators	Year published	Non-linearity	Solution Procedure	Element type
Aas-Jakobson & Grenacher	1974	Geometric & Material	Load control & Displacement control (deflection)	Line element
Crisfield	1983	Geometric & Material	Combined Load & Displacement control (arc-length)	Finite element
Bazant, Pan & Pijaudier-Cabot	1987	Material	Displacement control	Finite element
Wong, Yeo & Warner	1988	Geometric & Material	Displacement control (curvature)	Line element (segmented)
Sun, Bradford & Gilbert	1992, 1994	Geometric & Material	Combined Load & Displacement control (arc-length)	Finite element
Kawano & Warner	1995	Geometric & Material	Load control & Displacement control	Finite element
Wong & Warner (Present method)		Geometric & Material	Combined Load & Displacement control (arc-length)	Line element

Non-linear procedures for the analysis of concrete frames use either a finite element formulation or a direct stiffness line element formulation. This thesis describes a line element formulation for non-linear analysis of concrete structures.

2 LINE ELEMENT APPROACHES

Direct stiffness line element approaches (Aas-Jakobsen and Grenacher, 1974) discretise structures and their components into numerous axial flexural beam elements. The property of each element is usually assumed uniform, and dependent on a chosen section along the element which is modeled numerically by dividing the steel and concrete into layers. The axial and flexural stiffnesses are then calculated and these stiffnesses are used to form the element stiffness matrix. In a typical finite-element approach, the structure is also modeled by elements, each with numerous layers. However, the terms of the element stiffness matrix are formed directly from the properties of the layers at a few sections (usually three) along the element (Sun, Bradford and Gilbert, 1992, 1994; Kawano and Warner, 1995).

Aas-Jakobsen and Grenacher (1974) described a procedure which uses a geometric non-linear elastic frame analysis as the main routine to analyze concrete frames. Line elements were used to model these structures. Geometric non-linearities within an element were taken into consideration by augmenting each linear-elastic element stiffness k_e with a non-linear geometric element stiffness matrix k_g. Jennings(l968) had earlier pointed out in his paper on modeling of elastic plane frames that using such a formulation to model geometrical non-linearity effects, while including the modification of the element axial stiffness due to axial load, failed to include the modification of axial stiffness due to bowing (ie. lateral deflection). Material non-linearities were included by using a section analysis subroutine which provides flexural and axial stiffnesses of elements based on known thrust and bending moments. Uniform elements' with properties equal to those at mid-element were assumed by Aas-Jakobsen et al.

A different line element formulation with segmented elements was previously used by Wong, Yeo and Warner (1988). The properties of the segments making up an element were used to form its element stiffness matrix. This greatly reduces the size of the global stiffness matrix, which resulted in a reduction in both computer memory storage space and program execution time. However, the use of segmented elements, while increasing the computational efficiency, can reduce the accuracy of the modeling of the geometric non-linearities. To enable members with significant bowing, for example compression members, to be modeled accurately, several segmented elements were used to represent each of these members. Geometric non-linearities caused by bowing within the members, with several internal nodes and changing position of loaded nodes, were included by updating the position of nodes during the analysis. Thus, members which were least affected by geometric nonlinearities, e.g. beams, were modeled using a single segmented element.

Both the line element procedures of Aas-Jakobsen et al. and Wong et al. described above fail to model the geometric non-linearities accurately unless a large number of elements are used per member. This thesis describes a line element procedure which can model accurately the geometric non-linearities present in a frame.

3 PROPOSED METHOD OF ANALYSIS

The development of the computer program has been undertaken in two distinct stages. The first stage involved the development of a program capable of giving accurate solutions to geometrically non-linear plane frames. While binary-coded programs capable of analyzing geometrically non-linear frames are available commercially, the independent development of such a program enabled computer source codes to be created for subsequent modification to include the material nonlinearity effects. After having ascertained that the developed geometric non-linear analysis program was accurate, by comparing results with those obtained from published analytic solutions (Frish-Fay, 1962; Lee et al., 1968), the second stage of adding the appropriate subroutines for material non-linearities was carried out. Sections 3.1 and 3.2 below describe the program in accordance to these two stages of development.

Program for Analysis Second-Order Elastic Frames

In developing a procedure for the analysis of geometrically non-linear frames, the line element formulation of Jennings (1968) was used, together with the arc-length solution method of Crisfield (1981, 1983).

Jennings' line element formulation allows for significant change in geometry under loading. The same formulation was also been used by others (Meek and Tan, 1983; Meek, 1991). With this formulation, it is possible to determine the behavior of a plane frame with linear-elastic material properties until its deformed shape bears little resemblance to its original configuration (Jennings, 1968; Meek and Tan, 1983).

This formulation includes two parts which make it well-suited for use with a predictor-corrector solution procedure such as that of Crisfield. These two parts are (1) a tangent structural stiffness matrix which allows the prediction of an incremental load scaling factor and the corresponding displacements of the frame, and (2) a secant formulation which allows element forces to be determined based on the total displacements in the structure predicted using the tangent formulation of (1). Part (1) forms the predictor and part (2) forms the corrector for use after each prediction. A predictor-corrector solution procedure has also been adopted by Kawano and Warner (1995).

In other words, for each iterative cycle the structural tangent stiffness is used to predict the incremental displacements to obtain the latest position of the structure at the end of the iterative cycle, and the structural secant formulation is used to determine the forces required at the nodes to maintain the frame at this position. After having determined these forces, the out-of-balanced forces are determined. These out-of-balance forces are used for the next iterative cycle to improve the accuracy of the solution until the solution converges to within an acceptable tolerance.

Jennings' formulation takes into consideration all geometrical nonlinearities present in a structure and solutions obtained have been found to predict the behavior of elastic structures accurately (Jennings, 1968). The geometrically non-linearity effects considered in this formulation are:

? The change of element lateral stiffness,
? The finite deflection of joints, and
? The change of element length due to bowing.

Jennings' matrix formulation will not be presented here. It is described elsewhere (Jennings, 1968; Meek, 1991).

Modification of program to include modelling for material non-linearities

Jennings' line element formulation accurately predicts the behavior of geometrically non-linear frames and is most suitable for use as the foundation for the non-linear analysis of concrete structures because accurate modeling of the geometrical non-linearities is assured.

Solution procedures for non-linear structural analysis problems can be based on load control, displacement control or combined load displacement control. Load control procedures, while useful for structures subjected to working loads, are not suitable for tracing behavior at collapse. Using a displacement control procedure can be inefficient as the control displacement parameter needs to be decided before the commencement of the analysis. The control displacement parameter must be able to act as a monitor for the stiffness degradation of the chosen structure at all stages of loading, irrespective of where in the structure this degradation occurs. For example, for a free-standing cantilever column, the degradation of stiffness, indicated by the horizontal deflection at the top of the column, is mainly affected by the deformation of the segment at the base. Therefore, the curvature of this segment is suitable for use as the control displacement parameter. However, it is difficult to use a purely displacement control procedure at one location, e.g. a curvature or a deflection, for complicated frames where degradations occurring almost simultaneously in several regions.

For example, the yielding in a beam forming part of a multi-storey building does not affect the sway deflection as much as the yielding in a column.
Choosing an unsuitable control displacement results in non convergence of the solution procedure. For such cases, an arc-length control procedure can be used.

The arc-length control procedure, which is based on a control parameter in the form of a constant 'length' in a multi-dimensional load displacement space. For a structure with many degrees of freedom of movement, load-displacement control can be simultaneously carried out at all the degrees-of-freedom at each node. It is, therefore, more efficient for sensing the occurrence of multiple non-linear events, such as simultaneous yielding in different parts of a structure. Hence the arc length solution procedure was selected for use in the present program, to trace non-linear behavior, up to, and beyond, the point of collapse. Details of the arc-length procedure are given in Appendix A. In the present method, the tangent stiffness is updated at the start of every iterative cycle rather than at the start of the incremental step shown in Figure A1.

Jennings (1968) mentioned in his paper that a program had been developed to include the effect of material non-linearities using his formulation. However, the details of this program were not given. It is not known whether this program was developed to analyze concrete structures.

Sun et al. (1992, 1994) describe the development of a program which also uses the arc-length solution procedure of Crisfield. This program uses a finite element formulation instead of the direct stiffness line element formulation used in the present approach. A program developed by Kawano and Warner (1995) uses a finite element method similar to that by Sun et al. but it includes time-dependent effects and utilizes a displacement control solution procedure.

The procedure to analyze non-linear concrete frames is illustrated using the flow-diagram shown in Figure 1. Terms shown in this figure are the same as those given in Appendix A. This figure shows diagrammatically the inclusion of the section analysis subroutine. This section analysis routine models sections by using steel and concrete layers. The stress strain relation used is that first described by Warner (1969), with the concrete in tension having a relation described by Kenyon and Warner (1993). Unloading paths are included for concrete in both compression and tension. Concrete in tension is assumed to unload towards the origin, and concrete in compression is assumed to unload with initial slope. This is shown in Figure 2. The stress-strain relation for steel reinforcement is assumed to be elastic-plastic; steel reinforcements in both tension and compression are assumed to unload with initial slope.

Advantages of the Present Approach

The present approach ensures that the geometrical non-linearities are taken into consideration accurately in a non-linear frame. Premature buckling for a frame with compression members can thus be predicted accurately.

The use of uniform property elements enables finite length hinges to develop in the structure during the analysis. The present approach assumes that the property of an element is uniform, equal to that of the most critically stressed end-section. This assumption is conservative for most elements. Previous researchers such as Bazant (1976) and Bazant, Pan and Pijaudier-Cabot (1987) showed that hinges in concrete structures are of finite length. Bazant et al. (1987) suggested that the length of hinges should be approximately equal to its depth.

4 COMPARISONS WITH TEST RESULTS

In order to check the accuracy of the present approach, theoretical results have been obtained for some columns and frames used in previous experimental studies. The following material properties are assumed in the analysis. Modulus of elasticity for steel E_s is taken as 2.0E5 MPa and that of concrete E_c as 5050 $\sqrt{fcm}$ MPa, where f_{cm} is the mean concrete strength. The mean in-situ concrete strength is assumed to be equal to the mean cylinder strength. Value of the strain at maximum stress ?$_{cmax}$ is assumed to be 0.002. The parameter ?$_2$ used to define the shape of the concrete stress-strain curve (Wamer, 1969) is assumed to be 3.0. Where strength of concrete was determined using cubes, the conversion of $f_{cm} = 0.8\,f_{cube}$ was used.

Short Columns

Short columns subjected to concentric and eccentric loadings were tested by Hognestad (1951) and Hudson (1965). Results from these tests, and predictions from the present analysis are summarized in Table 2 and Table 3, respectively. The ratios of the test to calculated ultimate loads for the columns tested by Hognestad and Hudson are listed in the last columns of these tables. Members with concentric load were analyzed by assuming a very small load eccentricity of 1.0 mm.

The average and the standard deviation for P_{test}/P_{calc} obtained for Hognestad's test columns are 0.94 and 0.07, and those for Hudson's test columns are 1.04 and 0.07.

Table 2: Hognestad's tests of short columns

Tested by	Slender-ness l/d	Speci-men	Concrete strength, f_{cm} (MPa)	Cross-Section Width (mm)	Cross-Section Depth, d (mm)	Ecc. ratio *e/d*	P_{test} (kN)	P_{calc} (kN)	P_{test}/P_{calc}
Hognestad (1956) group II	7.5	B-6a	28.1	254	254	0.00	2028	1967	1.03
	7.5	B-6b	27.9	254	254	0.00	1868	1989	0.94
	7.5	C-6a	13.9	254	254	0.00	1001	1218	0.82
	7.5	C-6b	10.5	254	254	0.00	898	1052	0.85
	7.5	A-7a	36.1	254	254	0.33	1219	1168	1.04
	7.5	A-7b	40.1	254	254	0.25	1263	1491	0.85

	7.5	B-7a	28.1	254	254	0.25	1139	1157	0.98
	7.5	B-7b	27.9	254	254	0.25	1103	1153	0.96
	7.5	C-7a	13.6	254	254	0.25	627	731	0.86
	7.5	C-7b	10.5	254	254	0.25	564	638	0.88
	7.5	A-8a	38.1	254	254	0.50	721	823	0.88
	7.5	A-8b	40.1	254	254	0.50	676	842	0.80
	7.5	B-8a	32.4	254	254	0.50	694	764	0.91
	7.5	B-8b	29.4	254	254	0.50	649	730	0.89
	7.5	C-8a	12.5	254	254	0.50	440	463	0.95
	7.5	C-8b	12.5	254	254	0.50	440	463	0.95
	7.5	A-9a	35.2	254	254	0.75	396	439	0.90
	7.5	A-9b	35.6	254	254	0.75	406	441	0.92
	7.5	B-9a	32.4	254	254	0.75	418	433	0.97
	7.5	B-9b	30.1	254	254	0.75	398	427	0.93
	7.5	C-9a	13.0	254	254	0.75	325	349	0.93
	7.5	C-9b	11.9	254	254	0.75	291	338	0.86
	7.5	A-10a	35.2	254	254	1.25	205	209	0.98
	7.5	A-10b	35.6	254	254	1.25	196	209	0.94
	7.5	B-10a	29.4	254	254	1.25	193	205	0.94
	7.5	B-10b	30.1	254	254	1.25	196	206	0.95
	7.5	C-10a	15.9	254	254	1.25	198	194	1.02
	7.5	C-10b	12.2	254	254	1.25	200	191	1.05

Table 2 -contd: Hognestad's tests of short columns

Tested by	Slender-ness l/d	Speci-men	Concrete strength, f_{cm} (MPa)	Cross-S ection		Ecc. ratio *e/d*	P_{test} (kN)	P_{calc} (kN)	P_{test}/P_{calc}
				Width (mm)	Depth, d (mm)				
Hognestad (1956) group III	7.5	B-11 a	26.7	254	254	0.00	2224	2436	0.91
	7.5	B-11 b	27.7	254	254.	0.00	2157	2485	0.87
	7.5	C-11 b	14.3	254	254	0.00	1570	1758	0.89
	7.5	A-12a	28.6	254	254	0.25	1401	1419	0.99
	7.5	A-12b	34.8	254	254	0.25	1446	1596	0.91
	7.5	B-12a	29.7	254	254	0.25	1348	1447	0.93
	7.5	B-12b	27.7	254	254	0.25	1263	1388	0.91
	7.5	C-12a	15.9	254	254	0.25	1121	1048	1.07
	7.5	C-12b	15.2	254	254	0.25	1023	1026	1.00
	7.5	A-13a	36.9	254	254	0.50	979	1070	0.91
	7.5	A-13b	33.4	254	254	0.50	934	1012	0.92

	7.5	B-13a	24.7	254	254	0.50	801	861	0.93
	7.5	B-13b	29.6	254	254	0.50	916	946	0.97
	7.5	C-13a	15.9	254	254	0.50	672	699	0.96
	7.5	C-13b	14.3	254	254	0.50	609	671	0.91
	7.5	A-14a	36.9	254	254	0.75	632	720	0.88
	7.5	A-14b	35.2	254	254	0.75	681	712	0.96
	7.5	B-14a	24.7	254	254	0.75	617	640	0.96
	7.5	B-14b	31.7	254	254	0.75	489	691	0.71
	7.5	C-14a	13.5	254	254	0.75	514	498	1.03
	7.5	C-14b	14.3	254	254	0.75	463	511	0.91
	7.5	A-15a	35.2	254	254	1.25	391	365	1.07
	7.5	A-15b	33.4	254	254	1.25	351	364	0.97
	7.5	B-15a	26.2	254	254	1.25	329	357	0.92
	7.5	B-15b	31.9	254	254	1.25	376	362	1.04
	7.5	C-15a	13.5	254	254	1.25	322	330	0.98
	7.5	C-15b	14.3	254	254	1.25	331	334	0.99
							Average		0.94
							Std Deviation		0.07

Table 3: Hudson's tests of short columns

Tested by	Slender-ness l/d	Speci-men	Concrete strength, f_{cm} (MPa)	Cross-Section		EcG. ratio e/d	P_{test} (kN)	P_{calc} (kN)	P_{test}/P_{calc}
				Width (mm)	Depth, d (mm)				
Hudson (1956) Series I	8	11	24.8	102	102	0.00	267	272	0.98
	8	12	24.8	102	102	0.00	267	272	0.98
	8	13	24.8	102	102	0.00	299	272	1.10
	8	14	24.8	102	102	0.00	264	272	0.97
	8	21	26.9	102	102	0.00	311	300	1.04
	8	22	26.9	102	102	0.00	289	300	0.96
	8	23	26.9	102	102	0.00	309	300	1.03
	8	24	26.9	102	102	0.00	311	300	1.04
	8	31	28.3	102	102	0.00	307	306	1.00
	8	32	28.3	102	102	0.00	311	306	1.02
	8	33	28.3	102	102	0.00	289	306	0.94
	8	34	28.3	102	102	0.00	288	306	0.94
	8	41	25.5	102	102	0.00	289	279	1.04
	8	42	25.5	102	102	0.00	306	279	1.10
	8	43	25.5	102	102	0.00	306	279	1.10
	8	44	25.5	102	102	0.00	307	279	1.10

Series II	8	11	24.8	102	102	0.30	156	157	0.99
	8	12	24.8	102	102	0.30	196	157	1.25
	8	13	24.8	102	102	0.30	165	157	1.05
	8	14	24.8	102	102	0.30	178	157	1.13
	8	21	26.9	102	102	0.30	165	169	0.97
	8	22	26.9	102	102	0.30	196	169	1.16
	8	23	26.9	102	102	0.30	173	169	1.03
	8	24	26.9	102	102	0.30	173	169	1.03
	8	31	28.3	102	102	0.30	200	179	1.12
	8	32	28.3	102	102	0.30	200	179	1.12
	8	33	28.3	102.	102	0.30	178	179	0.99
	8	34	28.3	102	102	0.30	169	179	0.94
	8	41	25.5	102	102	0.30	200	196	1.02
	8	42	25.5	102	102	0.30	200	196	1.02
	8	43	25.5	102	102	0.30	200	196	1.02
	8	44	25.5	102	102	0.30	187	196	0.95
							Average		1.04
							Std Deviation		0.07

Long Columns

The results from the analysis of long columns are compared with test data in the last column of Tables 4 through 11. The average and standard deviation of the ratios of P_{test}/P_{calc} are also given in these tables. The concrete mean strength was assumed equal to the average cylinder strength. The reasonably good agreement for the individual investigation and the absence of any definite trends with major variables such as slenderness, load eccentricity, material properties suggests that the ultimate load of hinged columns can be predicted with good accuracy using the present analysis. . Members with concentric load were analyzed by assuming a small load eccentricity.

Table 4: Columns tested by Breen and Ferguson (1969)

Speci-men	Slender-ness L/h	Concrete strength, f_{cm} (MPa)	Cross-Section		Ecc. ratio e_2/h	P_{test} (kN)	P_{calc} (kN)	P_{test}/P_{calc}
			Width (mm)	Depth, h (mm)				
G1	20	25.6	152	102	0.30	151	169	0.89
G2	40	25.2	152	102	0.60	48	47	1.02
G3	50	25.5	152	102	0.75	30	31	0.97
G4	50	25.5	152	102	0.30	53	52	1.03
G5	60	28.7	152	102	0.90	29	23	1.28
G6	50	30.2	152	102	0.30	49	48	1.02
G7	40	33.4	152	102	0.20	67	76	0.88

G8	60	28.0	152	102	0.27	48	50	0.96
G9	20	27.4	152	102	0.30	147	164	0.90
G10	10	27.7	152	102	0.30	209	230	0.91
						Average		0.98
						Std Deviation		0.12

Table 5: Columns tested by Chang and Ferguson (1963)

Speci-men	Slender-ness L/h	Concrete strength, f_{cm} (MPa)	Cross-Section		Ecc. ratio e_2/h	P_{test} (kN)	P_{calc} (kN)	P_{test} / P_{calc}
			Width (mm)	Depth, h (mm)				
1	31	23.3	156	103	0.07	168	187	0.90
2	31	35.0	156	103	0.39	69	84	0.82
3	31	28.9	156	103	0.06	189	229	0.83
4	31	30.1	156	103	0.38	73	80	0.91
5	31	32.8	156	103	0.21	123	132	0.93
6	31	33.6	156	103	0.06	197	250	0.79
						Average		0.86
						Std Deviation		0.06

Table 6: Columns tested by Martin and Olivieri (1965)

Speci-men	Slende-r-ness L/h	Concrete strength, f_{cm} (MPa)	Cross-Section		Ecc. ratio e_2/h	P_{test} (kN)	P_{calc} (kN)	P_{test}/ P_{calc}
			Width (mm)	Depth, h (mm)				
402-1	40.0	30.0	127	90	0.00	147	157	0.93
402-2	40.0	24.3	127	90	0.00	125	137	0.91
412-1	40.0	33.6	127	90	0.21	118	123	0.96
412-2	40.0	25.0	127	90	0.21	89	102	0.87
422-1	40.0	34.9	127	90	0.39	93	89	1.05
422-2	40.0	25.7	127	90	0.39	76	76	0.99
432-1	40.0	37.3	127	90	0.28	96	113	0.85
432-2	40.0	26.4	127	90	0.28	93	93	1.00
						Average		0.95
						Std Deviation		0.07

Note: Columns with double curvature $e_1 / e_2 = - 0.5$

Table 7: Columns tested by Thomas (1939)

Speci-men	Slender-ness L/h	Concrete strength, f_{cm} (MPa)	Cross-Section Width (mm)	Depth, h (mm)	Ecc. ratio e_2/h	P_{test} (kN) .	P_{calc} (kN)	P_{test}/ P_{calc}
LC1	14.75	24.27	152.4	152.4	0.00	588	554	1.06
LC2	20.75	24.27	152.4	152.4	0.00	545	606	0.90
LC3	23.75	24.27	152.4	152.4	0.01	478	498	0.96
LC4	26.75	24.27	152.4	152.4	0.01	465	439	1.06
LC5	26.75	24.27	152.4	152.4	0.05	456	363	1.26
LC6	23.75	24.27	152.4	152.4	0.04	448	451	0.99
LC7	20.75	24.27	152.4	152.4	0.04	463	493	0.94
LC8	14.75	24.27	152.4	152.4	0.03	474	531	0.89
LC9R	26.75	24.27	152.4	152.4	0.02	360	360	1.00
LC10	23.75	24.27	152.4	152.4	0.04	374	374	1.00
LC11	20.75	24.27	152.4	152.4	0.04	418	379	1.10
LC12	14.75	24.27	152.4	152.4	0.03	438	492	0.89
PLC1	33.1667	24.27	76.2	76.2	0.06	82	62	1.32
PLC2	33.1667	24.27	76.2	76.2	0.06	81	63	1.28
						Average		1.05
						Std Deviation		0.14

Table 8: Columns tested by Green and Hellesland (1975)

Speci-men	Slender-ness L/h	Concrete strength, f_{cm} (MPa)	Cross-Section Width (mm)	Depth, h (mm)	Ecc. ratio e_2/h	P_{test} (kN)	P_{calc} (kN)	P_{test}/ P_{calc}
S1	15	34.4	178	127	0.10	502	556	0.90
S5	15	33.6	178	127	0.09	621	707	0.88
						Average		0.89
						Std Deviation		0.02

Table 9: Columns tested by MacGregor and Barter (1965)

Speci-men	Slender-ness	Concrete strength, f_{cm} (MPa)	Cross-Section Width (mm)	Depth, h (mm)	Ecc. ratio e_2/h	P_{test} (kN)	P_{calc} (kN)	P_{test}/ P_{calc}
A1	27	33.6	112	64	0.20	169	185	0.91
A2	27	32.7	112	64	0.20	169	186	0.91
B1	27	29.0	112	64	1.50	33	30	1.10
B2	27	32.6	112	64	1.50	31	30	1.05

Average	0.99
Std Deviation	0.10

note: Columns bent in double curvature, $e_1 / e_2 = -1$

Table 10: Columns tested by Drysdale and Huggins (1971)

Speci-men	Slender-ness	Concrete strength, f_{cm} (MPa)	Cross-Section Width (mm)	Cross-Section Depth, h (mm)	Ecc. ratio e_2/h	P_{test} (kN)	P_{calc} (kN)	P_{test}/P_{calc}
D-1-A	31	30.3	127	127	0.20	173	165	1.05
D-1-B	31	30.3	127	127	0.20	172	165	1.04
D-2-C	31	29.2	127	127	0.20	177	163	1.08
D-2-D	31	29.2	127	127	0.20	180	163	1.11
						Average		1.07
						Std Deviation		0.03

Table 11: Columns tested by Absel-Sayed and Gardner (1975)

Speci-men	Slender-ness	Concrete strength, f_{cm} (MPa)	Cross-Section Width (mm)	Cross-Section Depth, h (mm)	Ecc. ratio e2/h	P_{test} (kN)	P_{calc} (kN)	P_{test}/P_{calc}
D1	23	30.8	150	150	0.423	127	144	0.88
D4	23	31.2	150	150	0.847	62	71	0.89
D8	23	31.0	150	150	1.267	44	46	0.95
						Average		0.91
						Std Deviation		0.04

Frames

The results from the analysis of simple rectangular frames by the present method and the analysis carried out by Chang (l967) are compared with test data in Table 12 for the test frames shown in Figure 3. Chang used an inelastic analysis which was applicable to simple box frames. The analysis took into consideration both material and geometric non-linearities. For the present approach, the ratio P_{test}/P_{calc} ranges from 0.82 to 1.18 with an average and a standard deviation of 0.96 and 0.14 respectively. From the approach by Chang (1967) for simple frames, the ratio of P_{test}/P_{calc} obtained by him ranges from 0.73 to 1.19 with an average and a standard deviation of 0.98 and 0.15 respectively.

The differences between the values from tests and those obtained from the present analysis may be due to any of the following:

? Variations between the mean test strengths and the in-situ material strengths. In the present analysis, they were assumed to be the same.
? Joint effects on the test loads.
? Un-intentional restraining effect from the applied loads.

Therefore, comparisons between results from the present analysis with those obtained by the analysis by Chang (l967) were carried out. The average and standard deviation for the ratio of the ultimate load calculated by Chang to the ultimate load calculated by the present method are 0.99 and 0.09. The standard deviation is smaller than the value of 0.14 obtained earlier for P_{test}/P_{calc} for the present method. While the analysis by Chang gave results comparable to those obtained using the present analysis, the present analysis has the advantage of not been restrictive in its application.

5 CONCLUDING REMARKS

An efficient approach for the non-linear analysis of reinforced concrete frames using line elements has been developed. This approach was developed by including a routine for analyzing material non-linearity into a geometric non-linear solution procedure which uses an accurate, existing line element formulation presently used for the analysis of geometrically non-linear frames.

Table 12: Test results versus calculated values for rectangular frames

Frame	Width/ height	P_{test} (kN)	Present Method		Results by Chang (1967)		P_{calc}(Chang) / P_{calc} (Present Method)
			P_{calc} (kN)	P_{test}/ P_{calc}	P_{calc}(Chang) (kN)	P_{test}/ P_{calc}	
Frames tested by Furlong and Ferguson (1965)							
F-1	21	267	296	0.90	294	0.91	0.99
F-2R	21	274	333	0.82	298	0.92	0.89
F-3R	21	177	172	1.03	154	1.15	0.90
F-4	21	234	224	1.04	211	1.11	0.94
F-5	16	247	276	0.89	241	1.02	0.87
F-6	16	200	171	1.17	184	1.09	1.08
Frames tested by Breen and Ferguson (1964)							
F-1	30	262	222	1.18	227	1.15	1.02
F-2	30	262	287	0.91	274	0.96	0.95

F-3	15	271	250	1.08	227	1.19	0.91
F-4	15	371	361	1.03	316	1.17	0.88
Frames tested by tested by Ferguson and Breen (1965)							
L-1	20	167	225	0.74	217	0.77	0.96
L-2	20	111	121	0.92	139	0.80	1.15
L-3	20	138	143	0.97	145	0.95	1.01
L-5	10	189	170	1.11	191	0.99	1.12
L-6	10	245	287	0.85	299	0.82	1.04
L-7	10	178	237	0.75	245	0.73	1.03
Average				**0.96**		**0.98**	**0.99**
Std dev.				**0.14**		**0.15**	**0.09**

The resulting approach allows the modeling of frames which include the formation of finite length hinges in region of strength degradation.

Comparison of the collapse loads obtained from the present analysis with tests reported in the literature shows that the present analysis gives good estimates of collapse loads.

The approach is a suitable tool for use with non-linear design method. It is at present being used to determine the safety requirements associated with using such an analytical approach for non-linear design. A preliminary report of this work has been published (Wong and Warner, 1997).

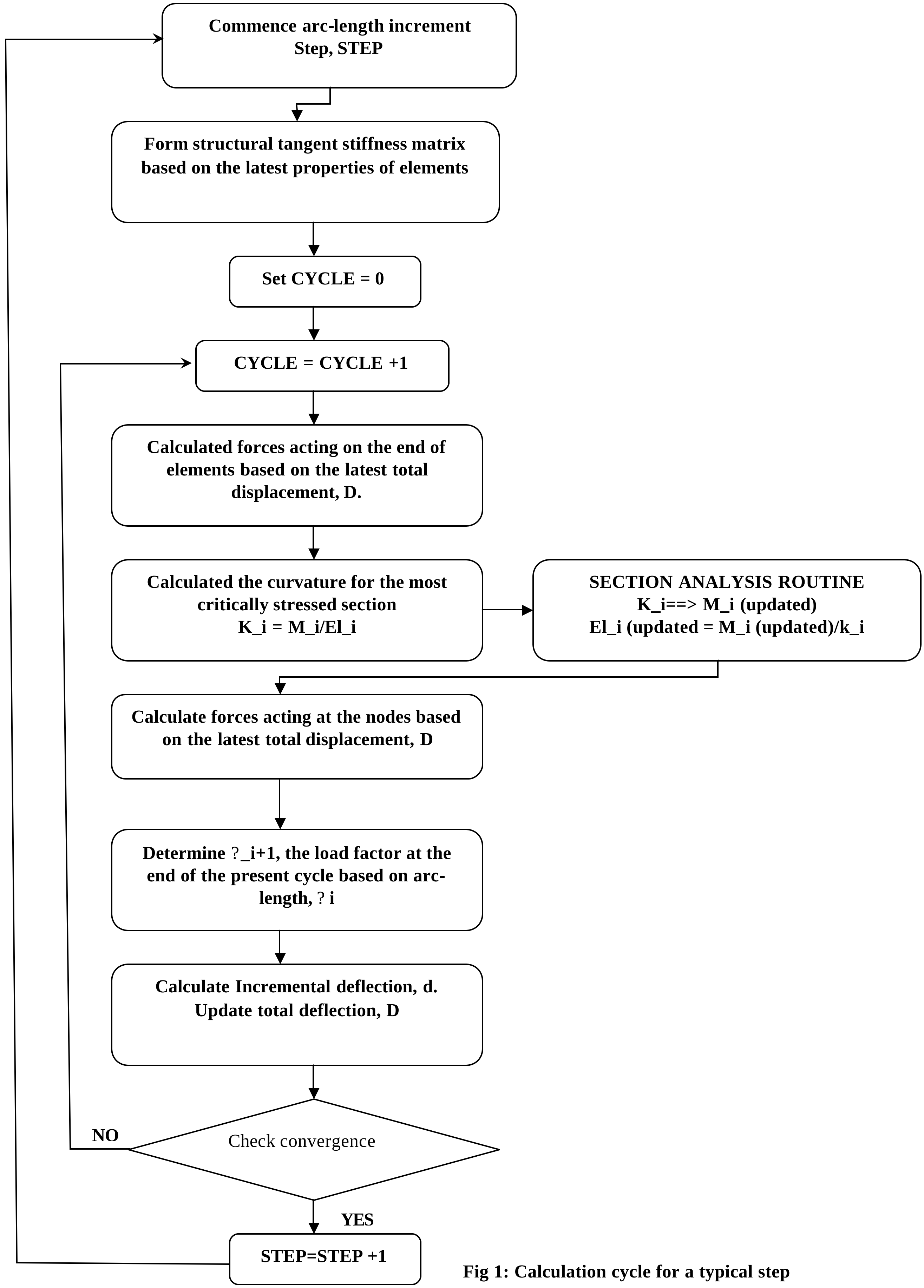

Fig 1: Calculation cycle for a typical step

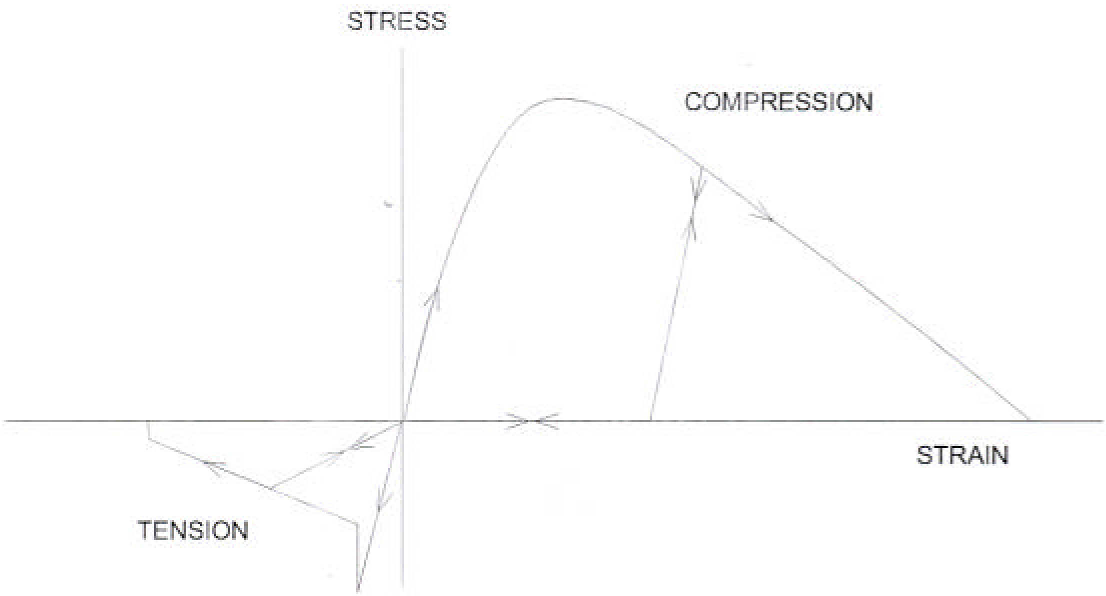

Fig 2: Diagram showing unloading paths for concrete

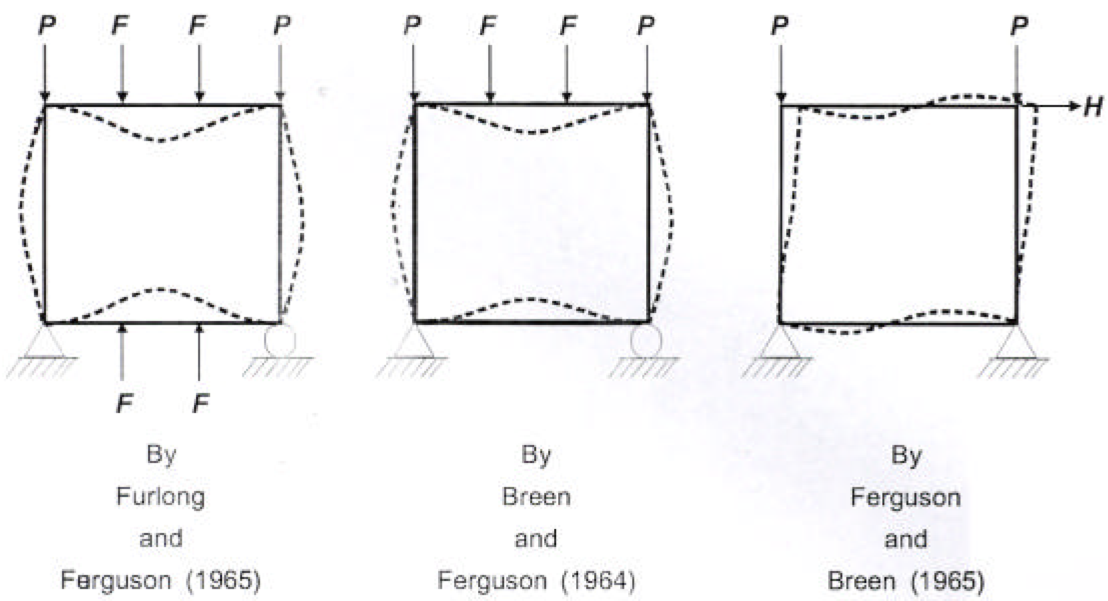

Fig 3: Rectangular test frames (after Chang, 1967)

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APPENDIX A: CRISFIELD'S ARC-LENGTH SOLUTION PROCEDURE

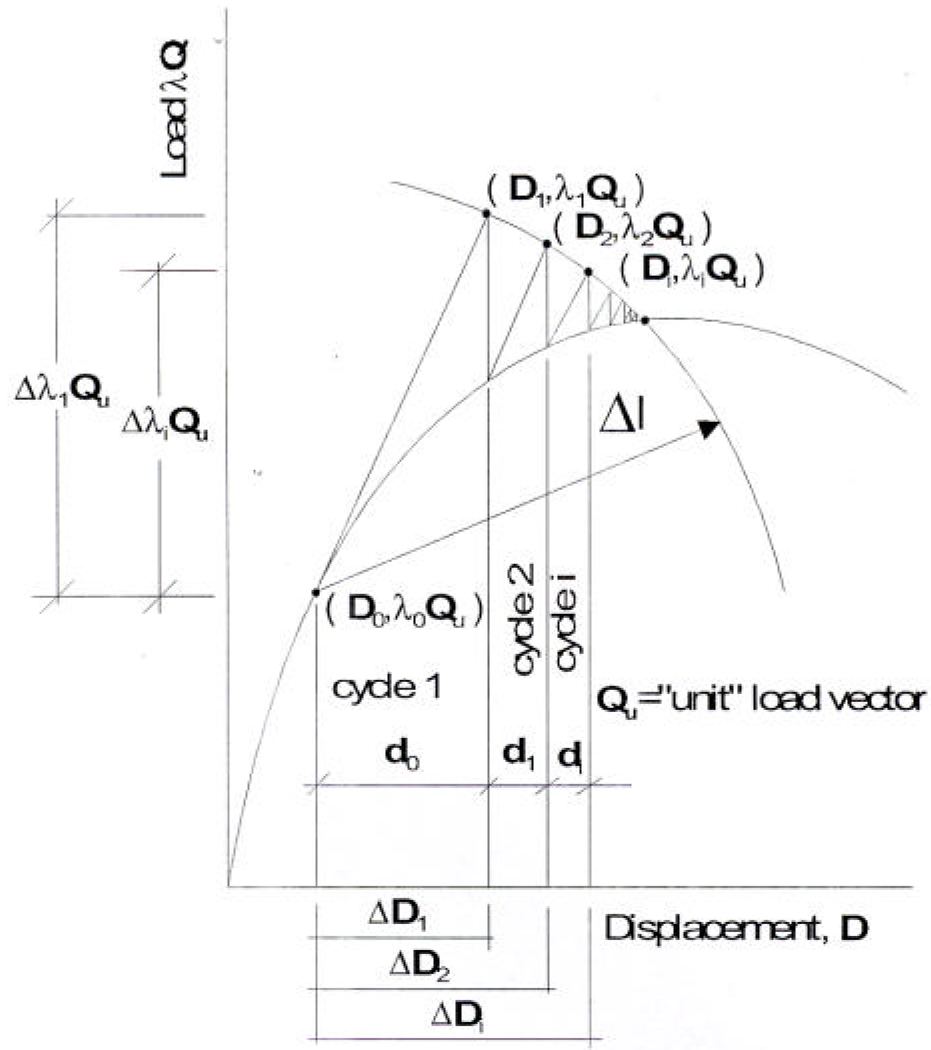

Fig. A1: The arc-length solution procedure (Crisfield, 1983)

This appendix describes Crisfield's arc-length solution procedure (Crisfield, 1983)

Figure A1 shows that for a typical iterative cycle *i*, the incremental displacement $\mathbf{d}_i$ is calculated by:

$$d_i \; ? \; K_t^{?1} ? f \, ?D_i ?? \; ?_i \; ? \; Q_u \; ?$$
$$? \; ? K_t^{?1} f \, ?D_i ?? \; ?_i \; ?_1 \, K_t^{?1} Q_u$$
$$? \; d_{Ni} \; ? \; ?_i \; ?_1 \, d_T$$

Where $\mathrm{d}_{Ni} \; ? \; ? \mathrm{K}_t^{?1} \, f \, ?\mathrm{D}_i \, ? \; and \; ?\mathrm{D}_i \, ?$ is the nodal forces based on the total displacement at the end of iterative cycle *i*.

At the beginning of the step $\mathrm{d}_T \; ? \; \mathrm{K}_t^{?1} \, \mathrm{Q}_u$ and d_{Ni} are known. To obtain d_i for each cycle $?_{i+1}$ needs to be determined.

The arc-length constraint equation to $?_{i+1}$ is

$$a_1 ?_{i?1}^2 \; ? \; a_2 ?_{i?1} \; ? \; a_3 \; ? \; 0$$

where

$$a_1 \; ? \; \mathrm{d}_T^T \, \mathrm{d}_T$$

$$a_2 \; ? \; 2\mathrm{d}_1 ? \; 2\mathrm{d}_2$$

$$a_3 \; ? \; \mathrm{d}_3 ? \; 2\mathrm{d}_4 ? \; ?? \; \mathrm{D}_i^T \; ? \; \mathrm{D}_i ? \; ?l^2 ?$$

$$\mathrm{d}_1 \; ? \; \mathrm{d}_T^T \; ? \; \mathrm{D}_i, \; \mathrm{d}_2 \; ? \; \mathrm{d}_T^T \mathrm{d}_{Ni}, \; \mathrm{d}_3 \; ? \; \mathrm{d}_{Ni}^T \mathrm{d}_{Ni} \; \text{ and } \; \mathrm{d}_{Ni}^T \; ? \; \mathrm{D}_i$$

Solving he constraint equation a above gives two value $?_{i+1}$; and substituting these values into the equation below gives two corresponding values of cos*?*, say *c1* and *c2*

$$\cos ? \; ? \, 1 ? \frac{1}{?l^2} ?d_4 \; ? \; ?_i ?_1 d_1 ?$$

If one of *c1* and *c2* I positive, choose $?_{i+1}$ corresponding o the one which is positive. However if both are positive, choose $?_{i+1}$ closest to $?_{i+1,lin}$ given below:

$$?_{i?1,lin} \; ? \; ? \frac{a_3}{a_2}$$

The arc-length $?l$ is based on an initial guess of $??$, and is obtained from the expression

$$?? \; ? \; a \frac{?l}{\sqrt{\mathrm{d}_T^T \mathrm{d}^T}}$$

Where *a* is the sign of *r* and $r \; ? \; \mathrm{d}_T^T \, \mathrm{K}_t \, \mathrm{d}_T$

SECTION II - Non-Linear Design of Reinforced Concrete Columns and Frames

1. INTRODUCTION

Back-calibration was proposed as a method of evaluating system safety coefficients for use in the non-linear, collapse load design of concrete structures by Wong and Warner. An extensive numerical study has also been undertaken to obtain appropriate values for the non-linear design of continuous reinforced concrete beams (Wong and Warner, 1998). For ductile flexural members it was found that the system safety coefficient value of

$$?_{system} = 0.68 \qquad (1)$$

was appropriate. This value can thus be used in the collapse load design equation,

$$?_{system}\ w_{u.rig}\ ?\ \mathrm{w}^* \qquad (2)$$

where w* is the design ultimate load, and $w_{u.rig}$ is the collapse load, calculated using a rigorous non-linear analysis.

The advantages of moving on from the current section strength design methods to collapse load design have also been discussed elsewhere (Wong and Warner, 1997a).

In this report, the back-calibration method is used to evaluate $?_{system}$ for other structural systems, in particular columns and frames, in which collapse may be governed by compression failure of the concrete and overall system instability.

To apply the back-calibration method, a structure is first designed according to the present code requirements, in this case, AS 3600 (Standard Australia, 1994), to carry a prescribed design ultimate load, say w^*. A rigorous, non-linear collapse load analysis is then undertaken to obtain its collapse load, $w_{u.rig}$, and the safety coefficient is then evaluated as

$$?_{system}\ ?\ \frac{w^*}{w_{u.rig}} \qquad (3)$$

The assumption here is that the same design loads and load combination factors will be used to determine w* in both the section strength design method and the collapse design load method, so, that design using Eq. 2 gives a structure with about the same safety margin as would be obtained from the current design procedures.

2. NON-LINEAR VALUES OF SYSTEM SAFETY COEFFICIENT FOR ISOLATED BRACED COLUMNS

Back-calibration calculations were initially undertaken for 70 slender columns with pin supports and equal end eccentricities as shown in Fig 1. Their cross sections were all 400 mm x 400 mm. The total amount of reinforcement was 3200 mm^2, with a concrete cover to centroid of steel reinforcement of 50 mm. Slenderness ratio L_e/r of the columns ranges from 17 to 121, and three values of end eccentricities e/D of 0.0625, 0.75 and 4.0 were used. Here e is the load eccentricity and D is the overall depth of the section. Note that the minimum end eccentricity e/D of 0.0625 is comparable to the minimum e/D of 0.05 specified by AS 3600. The following values were used: a characteristic strength of concrete of 32.0 MPa and a mean value of 37.2 MPa, and a steel yield stress of 400 MPa and a mean value of 460 MPa. The steel reinforcement is two percent of the gross cross-sectional area.

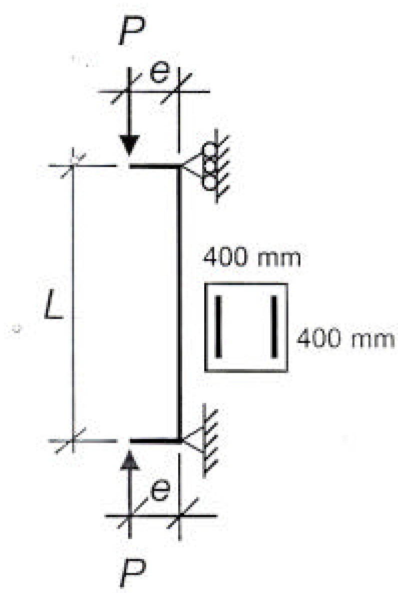

Figure 1: Braced column

As described in Section 1, the back-calibration procedure requires values of w^*, the design ultimate load, and $w_{u.rig}$, the accurately determined load capacity, for a wide range of structures. In the normal design situation, w* is known and by trial and error design details are found such that for critical sections, $?M_u$? M^*. However, in the present investigation we fixed the design details, and then determined both w* and $w_{u.rig}$ for each particular structural system. The calculation of $w_{u.rig}$ was carried out using the procedure described by Wong and Warner (1997b).

As the calculation of $w_{u.rig}$ does not include long-term effects such as creep and shrinkage, the creep factor $?_d$ (defined by AS 3600 as the ratio of the dead load to the sum of the dead load plus live load) used in the calculation of the corresponding w* was assumed to be zero.

The system safety coefficients for all the columns investigated are shown as a function of their slenderness ratios and end eccentricities in Fig. 2. The enormous scatter in results, with $?_{system}$ ranging from about 0.33 to 0.70, is not unexpected and can be explained by the peculiarities of the simplified methods used in AS 3600 for the analysis and design of columns.

Considering firstly the highest range of values, which occur for *e/h* varying from 4.0 down to, say, 0.75, we note that these all correspond to primary tension flexural failure in the mid-depth section of the column. The capacity reduction factor ? used in AS 3600 is close to 0.8 for these cases because ? for columns slides from 0.6 up towards 0.8 as the load capacity of the column falls below the load which produces 'balanced' failure.

On the other hand, at smaller values of *e/h* (e.g. below 0.5) and relatively small slenderness the values of $?_{system}$ cluster around 0.5. For these columns, failure is by primary compression and the capacity reduction factor ? from AS 3600 is 0.6. Furthermore, the effect of slenderness on strength is minimal at L_{eff}/r values of around 20. Thus if we reduce the value of $?_{system}$ for beams (0.68) by the ratio *0.6/0.8*, we obtain a value of about 0.5.

On the other hand, with progressively increasing slenderness the $?_{system}$ values diverge, usually upwards, but for the smallest eccentricities *(e/h* ? 0.125) the divergence is severely downwards. This reflects the very conservative nature of the AS 3600 moment magnification factor method at very small eccentricities and high slenderness.

The increase in calculated values of $?_{system}$ with increasing slenderness, as shown in Fig 2 for *e/h* va1ues of 0.5 and 0.25, requires explanation. It appears that at low slenderness the failure mode is in primary compression; however, with increasing slenderness there is sufficient lateral bending and outward movement of the central region of the column to lead to primary flexure failure and hence to the use of increasing ? values greater than

0.6. This view of column behavior is of course highly simplified and while convenient for design, is inaccurate. For example, it can be shown that stability failure can precede local section failure in a real column of finite length (Warner et al, 1989).

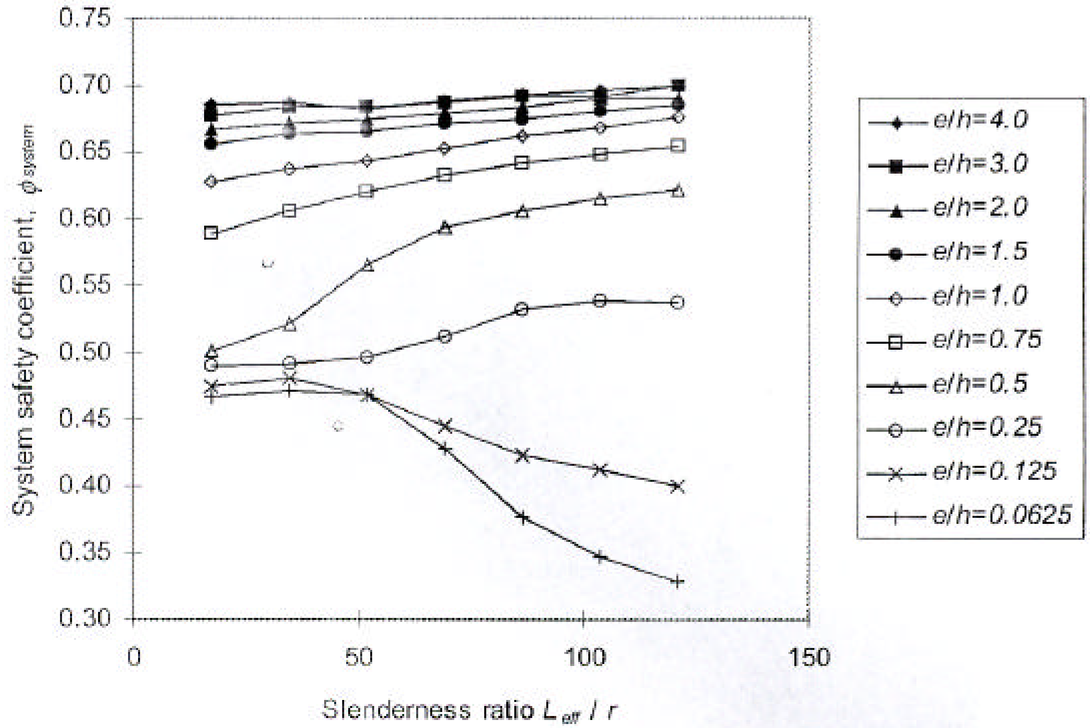

Figure 2: System safety coefficients for braced columns

The results shown in Fig 2 allow some preliminary suggestions to be made for values of $?_{system}$. Working from the basic $?_{system}$ value of 0.68 for ductile continuous beams, a smaller value, say 0.5 seems appropriate, for cases where collapse is due to compression type failure, and a further reduction, perhaps to 0.4, in cases of high slenderness, where stability failure dominates. It is emphasized that these are only indicative figures which will have to be confirmed or modified by additional modeling.

3. EVALUATION OF SYSTEM SAFETY COEFFICIENTS OF PORTAL FRAMES BY BACK-CALIBRATION

In this initial study of safety coefficients for structural frames, attention has been concentrated on simple portal frames. It should be noted that frames with ductile strong columns and weak beams with uniformly distributed loads acting along the beams fail when a plastic collapse mechanism occurs in the beam. For such cases the value of $?_{system}$ = 0.68 can be used for design (Wong and Warner, 1997a). Of interest here are frames which may fail by bending in both the beams and columns, and overall instability. The portal frames shown in Fig 3 were therefore used in the study, with a horizontal load *H* at floor level and point loads *P* on the columns, with the ratio *H/P* varying from 0.1 up to 1.5. Both fixed bases (as shown) and pinned bases were considered, with three frame heights of 4m, 6m, and 8m for the fixed-base frames; and two frames heights of 4m and 6m for the pinned-base frames. The slenderness ratios L_{eff}/r for the columns for the fixed-base frames, calculated in accordance with AS 3600, were 46, 66 and 85; and those for the pinned-base frames, 67 and 95. The total amount of reinforcement was 3200 mm^2 in the cross-section of the columns, with a concrete cover to centroid of steel reinforcement of 50 mm. Proportional loading was assumed, with *P* and *H* applied simultaneously, although a recent study suggests that the results would not be significantly different with a non-proportional *P-H* load sequence (Wong and Warner, 1997c).

For each frame the load capacity $P_{u,rig}$ was calculated by an accurate nonlinear analysis (Wong and Warner, 1997b), and the design ultimate load *p** determined in accordance with the simplified approach of AS 3600. The system safety coefficient was then determined:

$$?_{system} \; ? \; \frac{P\,*}{P_{u.rig}} \qquad (4)$$

The system safety coefficient values are shown in Tables I and II for the fixed base and pinned base frames, respectively, together with the moment magnification factor $?_s$, used in the design of the columns in the frames. The values of $?_{system}$ are also shown graphically in Fig 4 as a function of the load ratio *H/ P*.

In the case of the fixed base frames, the larger end moments in the columns are in the lower ends, next to the fixed-bases.

Hinges formed first in the lower ends of the column followed by the formation of hinges next to the ends of the beams. The values of $?_{system}$ close to 0.68 obtained for the frames is due to the formation of the second set of hinges in the beams at the beam column joints rather than in the columns, thus limiting the amount of load the frames can carry. The formation of the second set of hinges in the beams rather than in the columns reflects the strong-column weak-beam design which resulted from the simplified approach of AS 3600. The system safety coefficients are thus closer to those associated with ductile beam failure, rather than the more conservative values associated with column instability failure.

There are noticeable increases in safety margin in the three frames with column sections having the design strength governed by primary compression failure. The extra conservativeness is due to the use of a section strength design reduction factor *?* of 0.6 for columns which failed by primary compression failure when compared with a value of between 0.6 to 0.8 for those which failed by primary tension failure.

The moment magnification factor $?_s$ *is* also reasonably close to unity for *H/P* values at 0.5 and greater in the case of the pinned-base frames, although for some reason the values of $?_{system}$ are consistently higher than the expected value of 0.68 for *H/P* values at 0.5 and greater. The larger end moments in the columns are next to the connecting beams.

The values of $?_s$ increase markedly as the value of *H/P* decreases below 0.5. Quite unexpectedly the $?_{system}$ value increases. The frame which satisfies the design requirement of AS 3600 and has the least safety margin is the 4m tall frame with *H/P* = 0.1; it has a $?_{system}$ value of 0.89. This suggests that the current design methods give relatively less conservative results for pinned-base portal frames with large moment magnifiers $?_s$. in the columns.

A previous study by Pagay et al (1970) shows that the strength of beams has a pronounced effect on the strength of concrete frames, even though some present design standards fail to take proper account of this. For example, AS 3600 does not specify any requirement for increasing the reinforcement in adjoining beams, to take account of the additional moments transferred from the ends of the columns. Reinforcement in beams is incorrectly obtained using the action effects from a linear elastic analysis, assuming gross section properties for members.

A reassessment of the system safety coefficients was made for the pinned-based portal frame, but with the beams designed to carry the increased bending moment obtained from the first-order elastic analysis, magnified by the sway moment magnifier ?$_s$. The system safety coefficients obtained are given in Table III and illustrated in Fig 5. All now lie close to the value of 0.68. This confirms that the reduced conservativeness was caused by the failure of AS 3600 to require an increase in the reinforcement of the beam to complement the moment magnification effect acting on slender columns. As observed in a previous preliminary study (Wong and Warner, 1997a) on frames, the present study confirms that the conservative moment magnifier approach of AS 3600 for the design of columns always results in beam failure in the frames.

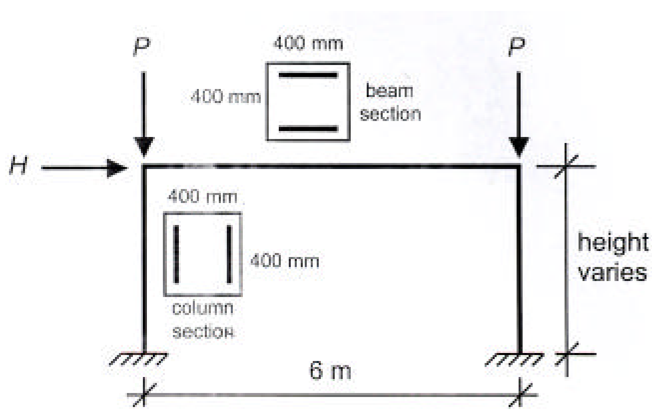

Figure 3: Fixed base portal frame

Table I

Moment magnifiers and system safety coefficients for fixed-base portal frames

	4m tall portal		6m tall portal		8m tall portal	
H/P	? s	? *system*	? s	? *system*	? s	? *system*
0.05	1.56*	0.51#	1.98*	0.60#	2.37*	0.68
0.10	1.34	0.53#	1.54*	0.63	1.63*	0.69
0.20	1.19	0.60	1.24	0.64	1.28	0.67
0.30	1 .11	0.61	1.14	0.63	1.17	0.67
0.40	1.08	0.62	1.10	0.63	1.13	0.67
0.50	1.06	0.63	1.08	0.61	1.10	0.66
0.60	1.05	0.63	1.06	0.62	1.08	0.66
0.80	1.04	0.63	1.05	0.63	1.06	0.64
1.00	1.03	0.61	1.04	0.63	1.05	0.66
1.50	1.02	0.62	1.02	0.64	1.03	0.66

note: * exceeds maximum value of 1.50 allowed in AS 3600
primary compression failure in critical column section in simplified design method of AS3600

Table II

Moment magnifiers and system safety coefficients for pinned-base portal frames

	4m tall portal		6m tall portal	
H/P	? s	? *system*	? s	? *system*
0.10	1.47	0.89	1.56*	0.93
0.20	1'.20	0.78	1.25	0.81
0.30	1.12	0.76	1.16	0.76
0.40	1.09	0.73	1 .11	0.73
0.50	1.07	0.69	1.09	0.72
0.60	1.06	0.70	1.07	0.71
0.80	1.04	0.68	1.05	0.69
1.00	1.03	0.68	1.05	0.69
1.50	1.02	0.71	1.05	0.69

note: * exceeds maximum value of 1.50 allowed in AS 3600

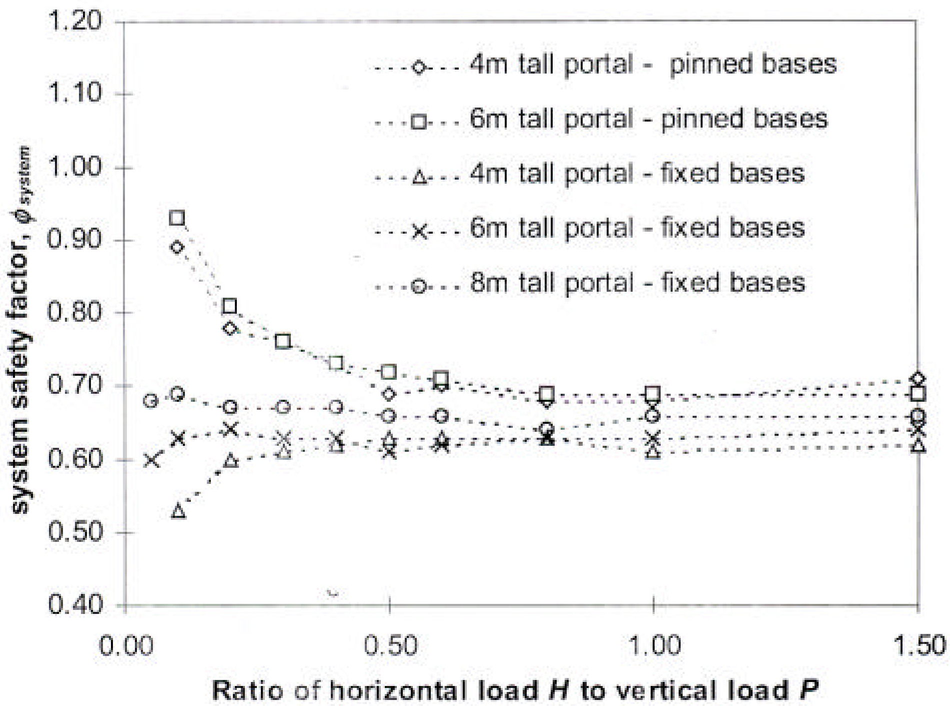

Figure 4: System safety coefficients for portal frames

Table III

Moment magnifiers and system safety coefficients for pinned-base portal frames with increased beam reinforcement

	4m tall portal		6m tall portal	
H/P	?$_{s}$	?$_{system}$	?$_{s}$	?$_{system}$
0.10	1.47	0.66	1.56*	0.68
0.20	1.20	0.67	1.25	0.68
0.30	1.12	0.68	1.16	0.67
0.40	1.09	0.68	1.11	0.67
0.50	1.07	0.66	1.09	0.68
0.60	1.06	0.67	1.07	0.68
0.80	1.04	0.64	1.05	0.67
1.00	1.03	0.67	1.05	0.70
1.50	1.02	0.70	1.05	0.68
note: * exceeds maximum value of 1.50 allowed in AS 3600				

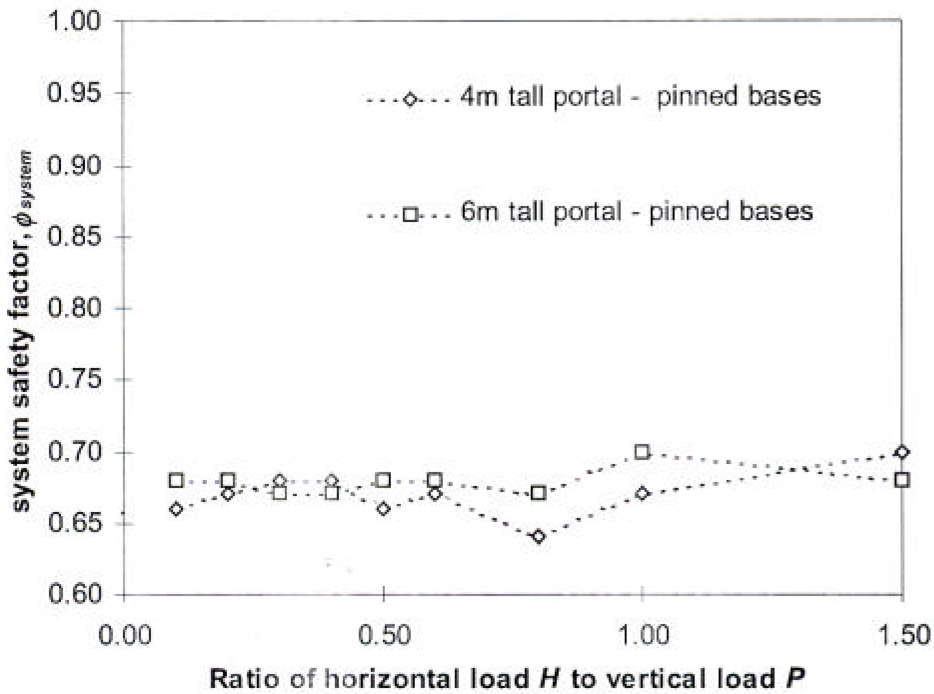

Figure 5: System safety coefficients for pinned-base portal frames with increased reinforcement in beams

4. CONCLUSIONS

The system safety coefficients obtained for the portal frames analyzed generally gave values of approximately 0.68. Several of these portal frames gave larger, and therefore less conservative, system safety coefficients. However, this was found to be caused by a deficiency in AS 3600 in not magnifying moments in beams connected to slender columns. When account is taken of the increased moment in such beams, the system safety coefficients of these frames were found to be close to 0.68.

In the extensive study carried *out* using a back-calibration procedure for pin-ended columns the system safety coefficients for columns designed using the simplified approach of AS 3600 range from 0.33 to 0.70. On investigation, it was shown that the AS 3600 simplified design method can give highly conservative results, and this explains the exceptionally low values sometimes obtained for $?_{system}$.
The present studies on columns and frames need to be extended. Further work will have to be carried out before reliable system safety coefficients can be proposed for the entire

range of structures. Nevertheless, appropriate values of $?_{system}$ for structures with normal strength concrete and f_{sy} = 400 MPa reinforcing steel is likely to be:

For ductile flexural systems and flexural collapse mechanisms:
$?_{system}$ = 0.68
For isolated columns with small load eccentricities and large slenderness ratios: $?_{system}$ = 0.40.
For frames which fail by loss of stability: $?_{system}$ = 0.40
For systems in which collapse is due to crushing of concrete, such as short axially loaded column: $?_{system}$ = 0.50

In structural systems where the mode of collapse is not due to the formation of a flexural collapse mechanism, and the collapse cannot be attributed to the crushing of concrete, the more conservative value $?_{system}$ of 0.40 for instability failure should be used.

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SECTION III - A Correlation Study between Ground Motion Parameters and Damage During Kocaeli Earthquake

Abstract:

Traditionally, ground motion parameters, such as peak ground acceleration *(pga)*, spectral acceleration (S_a), spectral velocity (S_v), and spectral displacement (S_d), are used as a damage indicator during an earthquake. This study is intended to search for a meaningful correlation between ground motion parameters and the structural damage based on 20 ground motion data from 1999 Kocaeli, Turkey Earthquake. The ground motion parameters are computed for both elastic and inelastic SDOF systems at four different natural periods; 0.25, 0.50, 0.75, and 1.25 seconds. Furthermore, the inelastic SDOF system involved different load deformation models namely bilinear and stiffness degrading. Nonlinear dynamic time history analyses are performed on a typical reinforced concrete frame system of the earthquake region for the 20 ground motion data. The damage indices (Park and Ang, 1985) and maximum inter-story drift ratios are computed for the frame system at four different natural periods. Finally, correlations are found between damage indices and each of the above mentioned ground motion parameters. Also correlation coefficients, which proved a well correlation, are calculated between damage indices and maximum inter-story drift ratios.

Keywords: Correlation, reinforced concrete frame, ground motion parameters, damage index, maximum inter-story drift ratio

1. INTRODUCTION

It is well known that there is interdependency between ground motion parameters {such as spectral acceleration (S_a), spectral velocity *(S_v)*, spectral displacement *(S_d)*, peak ground acceleration *(pga)*, and effective duration} and the structural response. Traditionally, the above mentioned ground motion parameters are used as a damage indicator during an earthquake. This study helps us to quantify the level of interrelationship between the above ground motion parameters and structural damage at a reinforced concrete frame system during a ground excitation.

Firstly, 20 ground motion data from Kocaeli, Turkey Earthquake (1999) are chosen. A computer analysis is performed and the ground motion parameters are computed for both elastic and inelastic SDOF systems at four different natural periods; 0.25, 0.50, 0.75, and 1.25 seconds. The inelastic SDOF system involved bilinear and stiffness degrading load deformation models. After that, nonlinear dynamic time history analyses are performed to provide structural response for a typical reinforced concrete frame system of the earthquake region. Maximum inter-story drift ratio and Park & Ang damage index (Park and Ang, 1985) are computed as damage indices. The attention is focused on structural damage, so that the number of the damage indices is limited to only two. Non-structural damage is not considered in this study. Finally, correlations are evaluated to express the level of interdependency between damage indices and each of the above mentioned ground motion parameters. Also correlation coefficients are calculated between damage indices and maximum inter-story drift ratios.

2. GROUND MOTION DATA AND PARAMETERS

Ground motion data recorded during Kocaeli Earthquake, as represented in Table 1, are used in this study. The peak ground accelerations *(pga)* are varying between 0.02 and 0.37g, where g is acceleration due to gravity. The first 14 ground motion data in the Table 1 are recorded at rock and stiff soil and the others are recorded at soft soil site. The closest distances for the records are varying 5.0 to 183.4 km.

Seismic parameters are chosen as peak ground acceleration *(pga)*, spectral displacement *(S_d)*, velocity *(S_v)*, and acceleration *(Sa)*. The effective duration is also computed. The effective duration in this study is defined as the duration where ground accelerations are over 0.05g in the record. The ground motion (seismic) parameters are computed for both elastic and inelastic SDOF systems at four different natural periods; 0.25, 0.50, 0.75, and 1.25 seconds. Furthermore, the inelastic SDOF system involved different load deformation models namely bilinear and stiffness degrading. For the seismic parameters computed for inelastic SDOF systems using the stiffness degrading model, the effect of yield strength coefficient and strain hardening are included. The yield strength coefficient is defined as yield strength of the oscillator divided by its weight, *Fy/W* Yield strength coefficients are chosen as 0.05 and 0.1 with 0 and 5 percentage of strain hardening (5 percentage of strain hardening is also included into the analysis for the natural period, 1.25 seconds). The seismic parameters are computed using bilinear load formation model for different ductility levels (ductility ratio=l, 2, 4, 6 and 8) at the natural period, 1.25 seconds.

3. NONLINEAR TIME HISTORY ANALYSES OF R/C FRAME STRUCTURES

Diagram of analyzed 5-story R\C frame structure with typical cross sections and steel reinforcements are shown in Figure 1. Nonlinear dynamic time history analyses are performed on a typical reinforced concrete frame system of the earthquake region for the 20 ground motion data for the evaluation of the structural seismic response. The computer program, IDARC 4.0 (Reinhorn et a1, 1996) is used for this purpose. The hysteretic behavior of the beams and columns has been specified at the ends of the members using "Park Hysteretic Model". This model incorporates stiffness degradation, strength deterioration, non-symmetric response, slip-lock, and trilinear monotonic envelope. The dead, live and seismic loads have been taken into account in design.

All reinforced concrete frame structures consist of three-bay frame, spaced at 800 and 500 cm. The story height is 450 cm in the first story and 300 cm for other stories. The columns are assumed to be fixed to the ground. Yield strength of the steel reinforcements is 22 kN/cm^2 and compressive strength of concrete is 1.8 kN/cm^2.

The natural periods of the frame structure are computed *0.25*, *0.50*, *0.75* and 1.25 seconds. The nonlinear analyses are performed for four different frame structures with same height and stiffness but different weights. As the modal weights for four frame structures are different, their natural periods are found to be different. The cross section of all beams is T -shapes with 25 cm width, 12 cm plate thickness, 50 cm total beam height and 100 cm effective plate width. The cross section of all columns is 60cmx30cm.

4. DAMAGE INDICES AND CORRELATION OF THE RESULTS

As damage indices, maximum inter-story drift ratio and overall structural damage index *(OSDI)* are chosen (Park and Ang, 1985). The focus is on overall structural damage index, *OSDI,* because it summarizes all existing damages on the structural components of the structure. The damage index is correlated with the seismic parameters, peak ground acceleration *(pga),* effective duration, spectral acceleration *(S_a),* spectral velocity *(S_v),* and spectral displacement *(S_d).* The correlation coefficients are presented in Table 2, 3, 4 and 5. Also correlation coefficients, which proved a well correlation, are calculated between overall structural damage index and maximum inter-story drift ratios (Table 2, 3, 4 and 5).

Park and Ang damage index for a structural element is defined in Equation 1.

$$DI_{Element} ? \frac{?_m}{?_u} ? \frac{?}{?_u P_y} ? dE_h \tag{1}$$

where, $?_m$ and $?_u$ are the maximum experienced and the ultimate deformations of the element, respectively, P_y is the yield strength, ? is a model constant parameter, and $?E_h$ is the hysteretic energy absorbed by the element during total duration of ground excitation.

To compute overall structural damage index, story damage indices should be calculated. The story damage index and overall structural damage index are given in Equation 2 and 3 respectively.

$$DI_{Story} ? ? \left(\frac{E_i}{? E_i} \right)_{element} ? DI_{element} \tag{2}$$

$$DI_{overall} ? ? \left(\frac{E_i}{? E_i} \right)_{story} ? DI_{story} \tag{3}$$

If the overall damage index is found to be larger than 1I, the structure is interpreted as collapsed. The structure with a damage index between 0.4 and 1.0 has moderate damage and smaller than 0.4 has moderate damage which is repairable.

The correlation coefficients (normalized covariance) and coefficient of determinations, R^2, are computed to express the level of interdependency between damage indices and each of the mentioned ground motion (seismic) parameters. Both correlation coefficients and determinations *(R^2),* values are between -1 and 1 and they are the measure of the degree of linear relationship between two variables (Ang and Tang, 1976). If the correlation coefficient or determination value is close to -1, there is a strong linear relationship between the observed data. It should be noted that if the number of observed data is large, correlation coefficient will be the same as R^2.

The correlation coefficient values (0.988 to 0.994) in Table 2, 3, 4 and 5 show very strong correlations between overall structural damage index and maximum inter-story drift ratio. However, overall structural damage index shows poor correlation to peak ground acceleration and effective duration in all periods. In Figure 2, correlation of determination, R^2, values are presented for 1.25 second period structure with SDOF system at the same period. Poor correlation between peak ground acceleration and effective duration and strong correlation between maximum inter-story drift ratios *(ISD)* with overall structural damage index can be recognized in the Figure 2. For the 0.25 period frame structure and SDOF system, a better correlation is observed between elastic seismic parameters with overall structural damage index for the frame system compared to correlation of inelastic seismic parameters and *OSDI* in Table 2. Similar to results for 0.25 second period systems, the fair correlation between *OSDI* elastic seismic parameters are observed at the period of 0.50 second (see Table 3). Fair correlation is generally recognized between both elastic and inelastic seismic parameters and *OSDI* at 0.75 second period (see Table 4). Especially, spectral displacement has the highest correlation to *OSDI* among three seismic parameters in Table 4. At the period of 1.25 second, the seismic parameters; spectral displacement and velocity for both elastic and inelastic SDOF system have better correlation to *OSDI* compared to correlation of spectral acceleration with *OSDI* (see Table 5). Finally, the spectral accelerations are computed for SDOF system with ductility 1, 2, 4, 6, and 8 at 1.25 second period. The overall structural damage index for 1.25 second period frame system is correlated to spectral accelerations for different ductility values. Spectral accelerations for higher ductility values (4, 6, and 8) have strong correlation to *OSDI* (see Figure 3).

5. CONCLUSIONS

In this study, the level of interrelationship between ground motion (seismic) parameters {such as spectral acceleration (S_a), spectral velocity (S_v), spectral displacement (S_d) peak ground acceleration *(pga)*, and effective duration} for both elastic and inelastic SDOF systems and structural damage at a reinforced concrete frame system during a ground excitation are quantified. Structural damage is represented by overall structural damage index *(OSDI)*, and maximum inter-story drift ratio *(ISD)*.

The two structural damage indices *OSDI* and *ISD* provide strong correlation to each other. However, peak ground acceleration and effective duration correlated poorly to *OSDI*. At the short periods, 0.25 and 0.50 seconds, we observe better correlation between elastic seismic parameters with *OSDI* compared to correlation of inelastic seismic parameters and *OSDI*. At the long periods, the three seismic (ground motion) parameters (both elastic and inelastic), S_a, S_v, and S_d, have fair correlation to *OSDI*.

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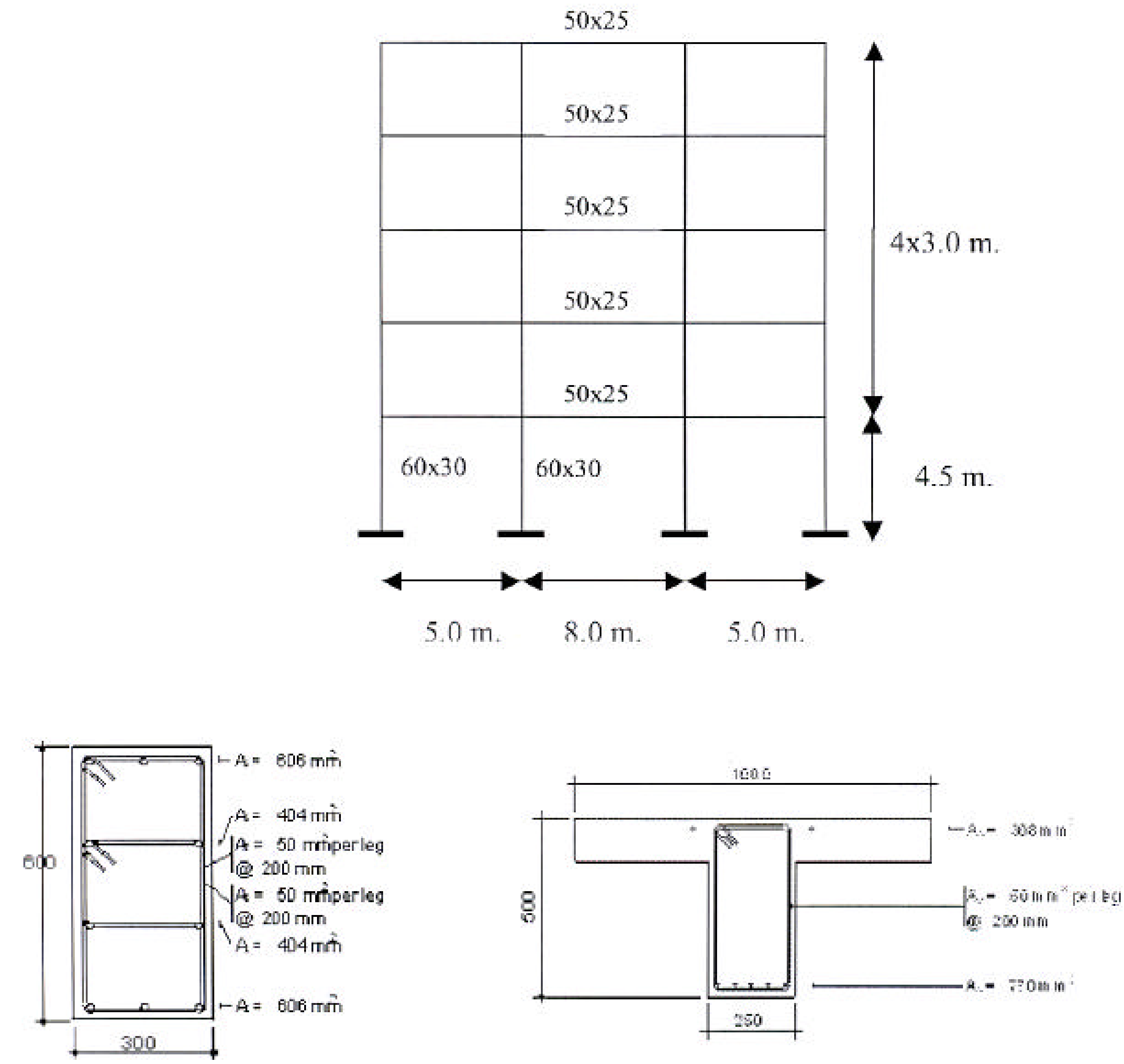

-Typical Column Cross Section-

-Typical Beam Cross Section-

Figure 1. Diagram of analyzed 5-story R\C frame structure with typical cross sections and steel reinforcements

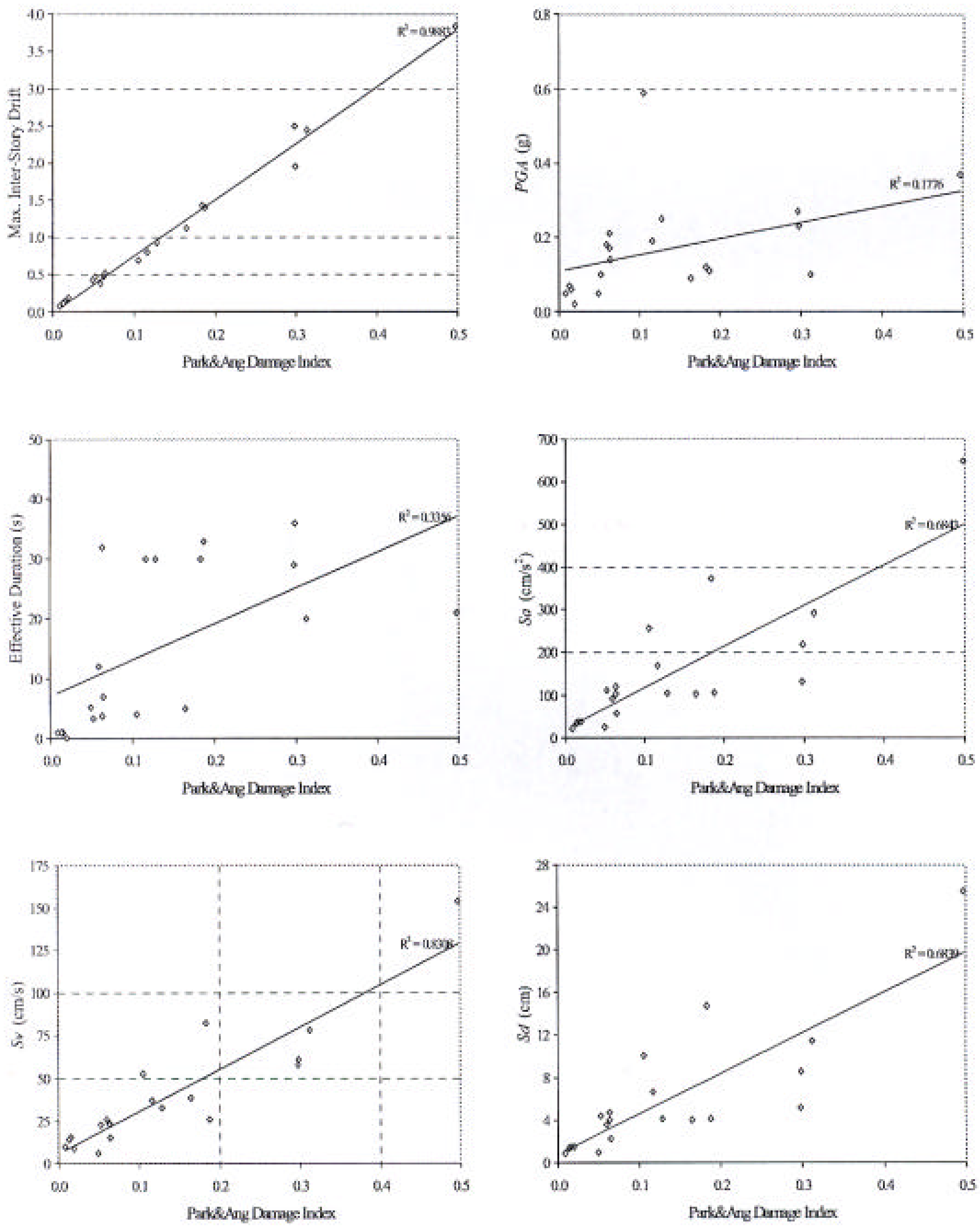

Figure 2. Correlation of Seismic Parameters and Damage Indices for T=1.25 sec.

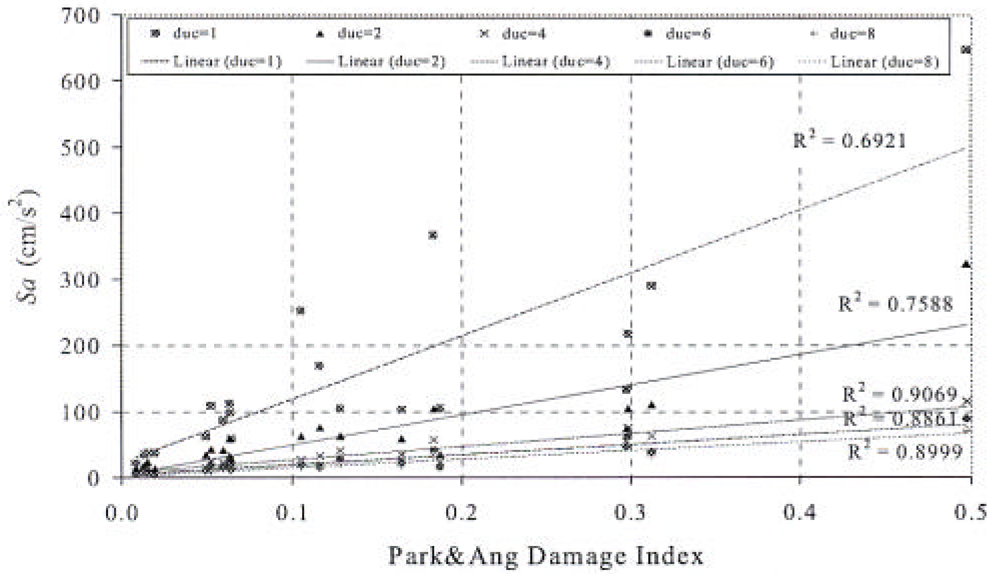

Figure 3. Correlation of S_a and Damage index for ductility 1, 2, 4, 6 and 8 -T=1.25 sec.

Table 1: Ground Motion data from Kocaeli Earthquake

No	Station	Record	*PGA* (g)	Distance (km)	Soil Class
1	Arcelik	Arc-long	0.21	21.6	Stiff Soil
2	Atakoy	Atk090	0.17	67.5	Stiff Soil
3	Balikesir	Blk090	0.02	183.4	Stiff Soil
4	Botas	Bts-long	0.10	136.3	Stiff Soil
5	Bursa Siv.Savunma	Brs-long	0.05	66.6	Stiff Soil
6	Cekmece	Cna-long	0.18	76.0	Stiff Soil
7	Gebze	Gbz000	0.27	13.5	Stiff Soil
8	Goynuk	Gyn000	0.14	35.5	Stiff Soil
9	Havaalani - Istanbul	Dhm-long	0.09	69.3	Stiff Soil
10	Istanbul	Ist-long	0.06	60.7	Stiff Soil
11	Izmit	Izt090	0.23	5.0	Rock
12	Maslak	Msk000	0.05	63.9	Rock
13	Mecidiyekoy	Mcd090	0.07	62.3	Stiff Soil
14	Zeytinbumu	Zyt000	0.12	63.1	Stiff Soil
15	Ambarli	Ats-long	0.25	78.9	Soft Soil
16	Bursa	Bur-long	0.10	62.7	Soft Soil
17	Fatih	Fat-long	0.19	62.2	Soft Soil
18	Kocamustafapasa	KMP000	0.11	62.7	Soft Soil
19	Iznik	IZN090	0.12	29.7	Soft Soil
20	Duzce	DZC270	0.37	12.5	Soft Soil

Table 2: Correlation coefficients between damage indices and seismic parameters, T=0.25, seconds

	Max ISD (%)	***PGA***	***Duration***
OSDI *(Park/Ang)*	0.988	0.286	0.429

	ELASTIC	**INELASTIC**			
		Stiffness Degrading, %0 Fy/W=0.05	*Stiffness Degrading, %5 Fy/W=0.05*	*Stiffness Degrading, %0 Fy/W=0.l*	*Stiffness Degrading, %5 Fy/W=0.l*
	OSDI *(Park/Ang)*				
S_a	0.752	0.535	0.519	0.522	0.537
S_v	0.593	0.476	0.498	0.509	0.515
S_d	0.752	0.503	0.503	0.451	0.464

Table 3: Correlation coefficients between damage indices and seismic parameters, T=0.50 seconds

	Max ISD (%)	***PGA***	***Duration***
OSDI *(Park/Ang)*	0.992	0.263	0.534

	ELASTIC	**INELASTIC**			
		Stiffness Degrading, %0 Fy/W=0.05	*Stiffness Degrading, %5 Fy/W=0.05*	*Stiffness Degrading, %0 Fy/W=O.l*	*Stiffness Degrading, %5 Fy/W=O.l*
	OSDI *(Park/Ang)*				
S_a	0.885	0.773	0.838	0.664	0.774
S_v	0.913	0.708	0.808	0.786	0.861
S_d	0.885	0.828	0.835	0.818	0.814

Table 4: Correlation coefficients between damage indices and seismic parameters, T=0.75 seconds

	Max ISD (%)	***PGA***	***Duration***
OSDI (Park/Ang)	0.994	0.483	0.440

	ELASTIC	**INELASTIC**			
		Stiffness Degrading, %0 Fy/W=0.05	*Stiffness Degrading, %5 Fy/W=0.05*	*Stiffness Degrading, %0 Fy/W=0.l*	*Stiffness Degrading, %5 Fy/W=0.l*
	OSDI (Park/Ang)				
S_a	0.885	0.796	0.924	0.664	0.798
S_v	0.852	0.635	0.653	0.670	0.790
S_d	0.885	0.881	0.936	0.978	0.974

Table 5: Correlation coefficients between damage indices and seismic parameters, T=I.25 seconds

	Max ISD (%)	***PGA***	***Duration***
OSDI (Park/Am!)	0.994	0.421	0.579

	ELASTIC	**INELASTIC**			
		Stiffness Degrading, %0 Fy/W=0.05	*Stiffness Degrading, %5 Fy/W=0.05*	*Stiffness Degrading, %0 Fy/W=0.l*	*Stiffness Degrading, %5 Fy/W=0.l*
	OSDI (Park/Ang)				
S_a	0.827	0.749	0.853	0.685	0.703
S_v	0.911	0.829	0.852	0.845	0.843
S_d	0.827	0.887	0.905	0.880	0.881

	INELASTIC	
	Stiffness Degrading, %-5 Fy/W=0.05	*Stiffness Degrading, %-5 Fy/W=0.l*
	OSDI (Park/Ang)	
S_a	0.841	0.652
S_v	0.827	0.848
S_d	0.832	0.877

SECTION IV - Case Study At Gaza – Palestinian National Authority

1. Introduction

This study is related to frame design with span of 18.0m (one floor) for celebration hall at Gaza.

This design was prepared by Samir Manneh by using computer software (ETABS Nonlinear Version8.4.5).

This hall was constructed on January 2005.

2. Technical Note for ETABS Nonlinear Version8.4.5

This Technical Note presents some basic information and concepts helpful when performing concrete frame design using this program.

2.1 Design Codes

The design code is set using the **Optional Menu > Preferences > Concrete Frame Design** command. You can choose to design for any one design code in any one design run. You cannot design some elements for one code and others for a different code in the same design run. You can, however, perform different design runs using different design codes without rerunning the analysis.

2.2 Units

For concrete frame design I this program, any set of consistent units can be used for input. Typically, design codes are based on one specific set of units. The documentation in the ACI 318-99 series of Technical Notes is typically presented in kip-inch-seconds units.

Again, any system of units can be used to define and design a building in this program. You can change the system of units that you are using at any time.

2.3 Overwriting the Frame Design Procedure for a Concrete Frame

The two design procedures possible for concrete beam design are:

- ? Concrete frame design
- ? No design

If a line object is assigned a frame section property that has a concrete material property, its default design procedure is Concrete Frame Design. A concrete frame element can be switched between the Concrete Frame Design and the "None" design procedure. Assign a concrete frame element the "None" design procedure if you do not want it designed by the Concrete Frame Design postprocessor.

Change the default design procedure used for concrete frame elements by selecting the element(s) and clicking **Design Menu > Overwrite Frame Design Procedure.** This change is only successful if the design procedure assigned to an element is valid for that element. For example, if you select a concrete element and attempt to change the design procedure to Steel Frame Design, the program will not allow the change because a concrete element cannot be changed to a steel frame element.

2.4 Design Load Combinations

The program creates a number of default design load combinations for shear wall design. You can add in your own design load combinations. You can also modify or delete the program default load combinations. An unlimited number of design load combinations can be specified.

To define a design load combination, simply specify one or more load cases, each with its own scale factor. See Technical Notes Design Load Combination Concrete Frame Design ACI 318-99, Design Load Combination Concrete Frame Design BS8110-89, Design Load Combination Concrete Frame Design. CSA-A23.3-94, Design Load Combination Concrete Frame Design EuroCode2-99,. Design Load Combination Concrete Frame NZS3101-95 and Design Load Combination Concrete Frame Design UBC97 for more information.

2.5 Design of Beams

The program designs all concrete frame elements designated as beam sections in their Frame Section Properties as beams (see **Define menu >Frame Sections** command and click the **Reinforcement** button). In the design of concrete beams, in general, the program calculates and reports the required areas of steel for flexure and shear based on the beam moments, shears, load combination factors, and other criteria, which are described in detail in Technical Notes Beam Design Concrete Frame ACI 318-99, Beam Design Concrete Frame BS 8110-89, Beam Design Concrete Frame CSA-A23.3-94, Beam Design Concrete Frame EuroCode2-99, Beam Design Concrete Frame NZS3101-95, and Beam Design Concrete Frame UBC97. The reinforcement requirements are calculated at each output station along the beam span.

All the beams are designed for major direction flexure and shear only. Effects resulting from any axial forces, minor direction bending, and torsion that may exist in the beams must be investigated independently by the user.

In designing the flexural reinforcement for the major moment at a particular section of a particular beam, the steps involve the determination of the maximum factored moments and the determination of the reinforcing steel. The beam section is designed for the maximum positive and maximum negative factored moment envelopes obtained from all of the load combinations. Negative beam moments produce top steel. In such cases, the beam is always designed as a rectangular section. Positive beam moments produce bottom steel. In such cases, the beam may be designed as a rectangular- or T -beam. For the design of flexural reinforcement, the beam is first designed as a singly reinforced beam. If the beam section is not adequate, the required compression reinforcement is calculated.

In designing the shear reinforcement for a particular beam for a particular set of loading combinations at a particular station resulting from the beam major shear, the steps involve the determination of the factored shear force, the determination of the shear force that can be resisted by concrete, and the determination of the reinforcement steel required to carry the balance.

2.6 Design of Columns

The program designs all concrete frame elements designated as column sections in their Frame Section Properties as columns (see **Define menu >Frame Sections** command and click the **Reinforcement** button). In the design of the columns, the program calculates the required longitudinal steel, or if the longitudinal steel is specified, the column stress condition is reported in terms of a column capacity ratio. The capacity ratio is a factor that gives an indication of the stress condition of the column with respect to the capacity of the column. The design procedure for reinforced concrete columns involves the following steps:

? Generate axial force-biaxial moment interaction surfaces for all of the different concrete section types of the model.

? Check the capacity of each column for the factored axial force and bending moments obtained from each load combination at each end of the column. This step is also used to calculate the required reinforcement (if none was specified) that will produce a capacity ratio of 1.0.

? Design the column shear reinforcement.

The shear reinforcement design procedure for columns is very similar to that for beams, except that the effect of the axial force on the concrete shear capacity needs to be considered. See Technical Notes Column Design Concrete Frame ACI 318-99, Column Design Concrete Frame BS 8110-89, Column Design Concrete Frame CSA-A23.3-94, Column Design Concrete Frame EuroCode2-99, Column Design Concrete Frame NZS3101-95, and Column Design Concrete Frame UBC97 for more information.

? Beam/Column Flexural Capacity Ratios

When the ACI 318-99 or UBC97 code is selected, the program calculates the ratio of the sum of the beam moment capacities to the sum of the column moment capacities at a particular joint for a particular column direction, major or minor. The capacities are calculated with no reinforcing over strength factor, ? , and including ? factors. The beam capacities are calculated for reversed situations and the maximum summation obtained is used.

The moment capacities of beams that frame into the joint in a direction that is not parallel to the major or minor direction of the column are resolved along the direction that is being investigated and the resolved components are added to the summation.

The column capacity summation includes the column above and the column below the joint. For each load combination, the axial force, P_u, in each of the columns is calculated from the program analysis load combinations. For each load combination, the moment capacity of each column under the influence of the corresponding axial load P_u is then determined separately for the major and minor directions of the column, using the uniaxial column interaction diagram. The moment capacities of the two columns are added to give the capacity summation for the corresponding load combination. The maximum capacity summations obtained from all of the load combinations is used for the beam/column capacity ratio.

The beam/column flexural capacity ratios are only reported for Special Moment-Resisting Frames involving seismic design load combinations.

See Beam/Column Flexural Capacity Ratios in Technical Note Joint Design Concrete Frame ACI 318-99 or Beam/Column Flexural Capacity Ratios in Technical Note Joint Design Concrete Frame UBC97 for more information.

2.7 Second Order P-Delta Effects

Typically, design codes require that second order P-Delta effects be considered when designing concrete frames. The P-Delta effects come from two sources. They are the global lateral translation of the frame and the local deformation of elements within the frame.

Consider the frame element shown in Figure 1, which is extracted from a story level of a larger structure. The overall global translation of this frame element is indicated by ? . The local deformation of the element is shown as ?. The total second order P-Delta effects on this frame element are those caused by both ? and ?.

The program has an option to consider P-Delta effects in the analysis. Controls for considering this effect are found using the **Analyze menu> Set Analysis Options** command and then clicking the Set P-Delta Parameters button. When you consider P-Delta effects in the analysis, the program does a good job of capturing the effect due to the ? deformation shown in Figure 1, but it does not typically capture the effect of the ? deformation (unless, in the model, the frame element is broken into multiple pieces over its length).

In design codes, consideration of the second order P-Delta an effect is generally achieved by computing the flexural design capacity using a formula similar to that shown in Equation. 1.

$$M_{CAP} = aM_{nt} + bM_{lt} \qquad \text{Eqn.l}$$

where,

M_{CAP} = Flexural design capacity

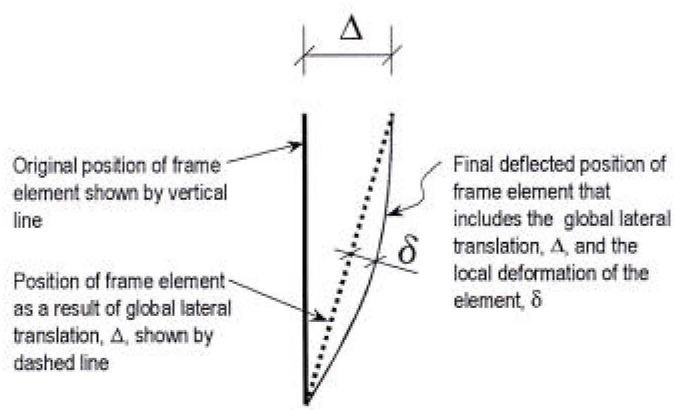

Figure 1 The Total Second Order P-Delta Effects on a Frame Element Caused by Both Δ and δ

M_{nt} =required flexural capacity of the member assuming there is no translation of the frame (i.e., associated with the ? deformation in Figure 1)

M_{lt} = =Required flexural capacity of the member as a result of lateral translation of the frame only (i.e., associated with the ? deformation in Figure 1)

a=Unitless factor multiplying M_{nt}

b = Unitless factor multiplying M_{lt} (assumed equal to 1 by the program; see below)

When the program performs concrete frame design, it assumes that the factor *b* is equal to 1 and it uses code-specific formulas to calculate the factor a. That *b* = 1 assumes that you have considered P-Delta effects in the analysis, as previously described. Thus, in general, if you are performing concrete frame design in this program, you should consider P-Delta effects in the analysis before running the design,

2.8 Element Unsupported Lengths

The column unsupported lengths are required to account for column slenderness effects. The program automatically determines these unsupported lengths. They can also be overwritten by the user on an element-by-element basis, if desired, using the **Design menu> Concrete Frame Design> View/Revise Overwrites** command.

There are two unsupported lengths to consider. They are L_{33} and L_{22}, as shown in Figure 2. These are the lengths between support points of the element in the corresponding directions. The length L_{33} corresponds to instability about the 3-3 axis (major axis), and L_{22} corresponds to instability about the 2-2 axis (minor axis). The Length L_{22} is also used for lateral-tensional buckling caused by major direction bending (i.e., about the 3-3 axis).

In determining the values for L_{22} and L_{33} of the elements, the program recognizes various aspects of the structure that have an effect on these lengths, such as member connectivity, diaphragm constraints and support points. The program automatically locates the element support points and evaluates the corresponding unsupported length.

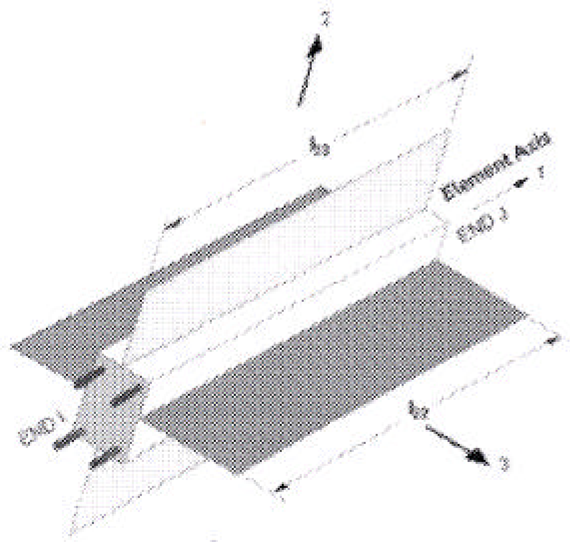

Figure 2 Major and Minor Axes of Bending

It is possible for the unsupported length of a frame element to be evaluated by the program as greater than the corresponding element length. For example, assume a column has a beam framing into it in one direction, but not the other, at a floor level. In this case, the column is assumed to be supported in one direction only at that story level, and its unsupported length in the other direction will exceed the story height.

2.9 Analysis Sections and Design Sections

Analysis sections are those section properties used to analyze the model when you click the **Analyze menu> Run Analysis** command. The design section is whatever section has most currently been designed and thus designated the current design section.
Tip:
It is important to understand the difference between analysis sections and design sections.

It is possible for the last used analysis section and the current design section to be different. For example, you may have run your analysis using a W18X35 beam and then found in the design that a W16X31 beam worked. In that case, the last used analysis section is the W18X35 and the current design section is the W16X31. Before you complete the design process, verify that the last used analysis section and the current design section are the same. The **Design menu> Concrete Frame Design > Verify Analysis > Design Section** command is useful for this task.

The program keeps track of the analysis section and the design section separately. Note the following about analysis and design sections:

? Assigning a beam a frame section Property using the **Assign menu > Frame/Line > Frame Section** command assigns the section as both the analysis section and the design section.

? Running an analysis using the **Analyze menu > Run Analysis** command (or its associated toolbar button) always sets the analysis section to be the same as the current design section.

? Assigning an auto select list to a frame section using the **Assign menu > Frame/Line > Frame Section** command initially sets the design section to be the beam with the median weight in the auto select list.

? Unlocking a model deletes the design results, but it does not delete or change the design section.

? Using the **Design menu> Concrete Frame Design > Select Design Combo** command to change a design load combination deletes the design results, but it does not delete or change the design section.

? Using the **Define menu > Load Combinations** command to change a design load combination deletes the design results, but it does not delete or change the design section.

? Using the **Options menu** > references > **Concrete Frame Design** command to change any of the composite beam design preferences deletes the design results, but it does not delete or change the design section.

? Deleting the static nonlinear analysis results also deletes the design results for any load combination that includes static nonlinear forces. Typically, static nonlinear analysis and design results are deleted when one of the following actions is taken:

  ✍ Use the **Define menu > Frame Nonlinear Hinge Properties** command to redefine existing or define-new hinges.

  ✍ Use the **Define menu > Static Nonlinear / Pushover Cases** command to redefine existing or define new static nonlinear load cases.

  ✍ Use the **Assign menu > Frame/Line > Frame Nonlinear Hinges** command to add or delete hinges.

Again, note that these actions delete only results for load combinations that include static nonlinear forces.

3. Design Results

The design results show that, the level of interrelationship between ground motion (seismic) parameters such as spectral acceleration, spectral velocity, spectral displacement, peak ground acceleration, and effective duration for both elastic and inelastic SDOF systems and structural damage at a reinforced concrete frame system during a ground excitation are quantified.