ANALYTICAL INVESTIGATION OF P-DELTA EFFECTS IN MEDIUM-HEIGHT STEEL MOMENT RESISTING FRAMES UNDER SEISMIC LOADING

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by

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To my parents

A B S T R A C T

An analytical investigation of the P-delta effects in medium height steel moment resisting frames to selected earthquake motions was undertaken.

The frames with various design drifts and fundamental time periods were designed according to the load provisions for New Zealand seismic risk zone A and to the requirements of Draft Code of Practice for General Structural Design and Design Loading for Buildings, DZ4203.

The study was carried out using a two-dimensional non-linear dynamic analysis computer program.

The effects of strength degradation and vertical acceleration were investigated. Dynamic magnification factors for the member forces and displacement were also observed. The limits of the maximum plastic hinge rotation, inelastic drift, curvature ductility, displacement ductility and prediction of maximum plastic hinge rotation based on the inter-storey drift were suggested.

The satisfactory drift limit is recommended and suggestions for further research are put forward.

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C	Basic seismic coefficient
$C\mu$	Basic seismic coefficient varying with ductility
[C]	Viscous damping matrix
D	Dead load
di	Inelastic drift index
E	Earthquake load
H	Height of the frame
h	Storey height
[K]	Initial stiffness matrix
l	Length of beam between centre lines of support
lc	Average of storey height
Lr	Reduced live load
Ls	Serviceability live load
m	Mass per unit length
M	Bending moment
M_θ	Rotational mass
[M]	Mass matrix
Me	Dependable beam flexural capacity required to resist design earthquake loading, with reference to column centre lines
Mp	Plastic moment
My	Yield moment
P	Axial compression force
Py, Pyc	Axial compression yield force
Pyt	Axial tension force
Qr	Stability index with reference to floor r
R	Risk factor
r	Bi-linear factor
To	Initial fundamental time period
T1	Effective time period measured from a half cycle before and a half cycle after the peak top floor displacement
T2	Effective time period measured from twice of a half cycle before the peak top floor displacement

V	Base shear
W	Weight
Wt	Total reduced gravity load of structure (D+L/3)
Wtr	Total gravity load considered at floor r
Z	Seismic risk zone
λ	Displacement magnification factor
λ_1, λ_2	Critical damping fractions
Δu	Maximum displacement at roof level calculated accordance with NZS4203.
μ	Displacement ductility ratio
ω	Natural circular frequency
θ	Plastic hinge rotation
Σ	Denotes sum of

CHAPTER 1

INTRODUCTION

1.1 GENERAL

Investigations of P-delta effects in recent years indicated that the combination of large gravity loads and lateral displacement, especially in medium to high rise buildings, will cause this second order effect to become significant. Many different studies of P-delta effects have been carried out. In previous investigations the level of significance of the P-delta effect was expressed in terms such as stability indices, drift indices and the ratio of base shear to total mass. When the P-delta effects become significant, consideration must be given to the large increases in displacement, curvature ductility, plastic hinge rotation and drift so as to maintain stability and serviceability of the structures.

There are many different approaches to solve the P-delta effects. In static analyses the increase of secondary moment is taken into account as the product of relative inter-storey displacement and the vertical force. In dynamic time-history analyses the effect of changing coordinates must be taken into account in every step of the time-history analysis, although there is a possible simplification which will be described later.

For static analyses using a drift limit and stability index at a certain level as was done in a previous study [3,19], the effect of P-delta can be dealt with in a practical and simple way. The complexity of P-delta effects in dynamic time-history analyses arise because of the characteristics and the intensities of different earthquakes and structural properties of the materials.

In this project, thirteen steel moment resisting frames were designed for various design drifts and fundamental periods, based on the DZ4203's equivalent static approach with a basic seismic acceleration coefficient for seismic zone A [15]. The behaviour of the frames has been investigated by inelastic time-history analyses using five different earthquake records. Due to time constraint, only two-bay steel moment resisting frames were considered in this project.

Relationships between the results of the equivalent static analysis and dynamic time- history analysis were drawn to convert the inelastic time-history analysis to the static analysis based on DZ4203 since the dynamic time-history analyses are expensive. Satisfactory design drift limits are recommended and the influence of drift limitations in plastic hinge rotation, curvature ductility and in designing the frame lateral stiffness are also described.

Effective time period, displacement ductility, the effect of vertical acceleration and strength degradation were investigated. Amplification factors for column moment, axial force and base shear forces as an indication of force amplification factors for column design were observed as well.

1.2 PREVIOUS STUDIES RELATED TO P-DELTA EFFECTS

Investigations and studies of P-delta effects have been carried out for concrete frames using both static and inelastic time-history analyses. Different approaches have been used to allow for these P-delta effects.

Paulay [19] discussed the probable effect of P-delta moments on inelastic dynamic frame response. It was suggested that if the strength demand due to the P-delta effect exceeds 15% of the ideal lateral load carrying capacity of a sub-frame, the strength demand should be increased. P-delta effect should be considered by evaluating the stability index from the following expression:

$$Qr = (\lambda\, lc\ Wtr\, \Delta u) \,/\, H \sum Me$$

where
- Qr = stability index with reference to floor r
- λ = displacement magnification factor
- lc = average storey height
- Wtr = total gravity load considered at floor r
- Δu = maximum displacement at roof level
- H = total height of frame
- Me = dependable beam flexural capacity required to resist design earth-quakes loading.

where the value of λ should be taken as 2.0, 2.4 and 3.0 in seismic zones A,B and C respectively.

Andrews [1] discussed lateral flexibility and displacement ductility controls to ensure that frame P-delta effect never becomes significant and could be ignored. It was recommended that deflection control provided an efficient, acceptable and certain means of limiting P-delta effect to a tolerable level. The current values of drift limit at 0.01 for zone A was maintained and new limits of 0.008 and 0.006 for zones B and C were introduced.

Moss and Carr [3], using a dynamic time-history analysis program for inelastic frame structures, investigated the response of several concrete frames with different stiffness properties and strength. It was found that with a drift limit of 0.01, P-delta effects can be ignored. For greater inter-storey drift the effect of gravity load leads to a rapidly increasing augmentation of the inter-storey drift and exceeds the ability of the structures to provide the necessary ductility. Increasing the strength for inelastic frames, rather than stiffness, offers the most effective control of increase in displacement.

The future use of time-history analysis of multi-storey frames should include the P-delta effect because this effect is always present in the real structures and consequently any analysis not including P-delta effects are really artificial and incomplete.

Montgomery [11] found in his study that P-delta effect has a significant influence when the ratio of the total weight to base shear is greater than or equal to 10, or the maximum storey drift is more than twice the storey drift at yield.

1.3 METHOD OF ANALYSIS

The analysis was carried out using 'RUAUMOKO' a two-dimensional computer program for dynamic time history analysis, originally developed by Sharpe [20], but now extensively modified by Carr [4]. The motion of two-dimensional frames in this project were simulated under horizontal and combined horizontal and vertical earthquake ground acceleration records.

The time step of 0.01 seconds was found to be adequate in previous sensitivity studies. Three different options are available in this program. Firstly the standard analysis without considering the P-delta effect. Secondly the member properties are redefined at every time step in terms of updated coordinates. Thirdly, for a simplification of the analysis, the stiffness is modified for the gravity induced axial loads at the beginning of the analysis. The disadvantage of updating the coordinates of all joints and axial forces in the frame at every time step result in very great increase of the computational cost.

Four moment-axial interaction surfaces, seventeen alternative moment-curvature hysteresis relationships and four viscous damping models are available in this program.

1.4 DESIGN OF FRAMES

The frames were designed according to DZ4203 with the new basic seismic acceleration spectrum, which seems slightly stronger than El Centro 1940. Calculation of the base shear is based on:-

$$V = C\,W_t$$

$$C = C_\mu\,R\,Z$$

where: C_μ basic seismic coefficient varying with μ

Z Zone factor,
for seismic risk zone A = 0.85

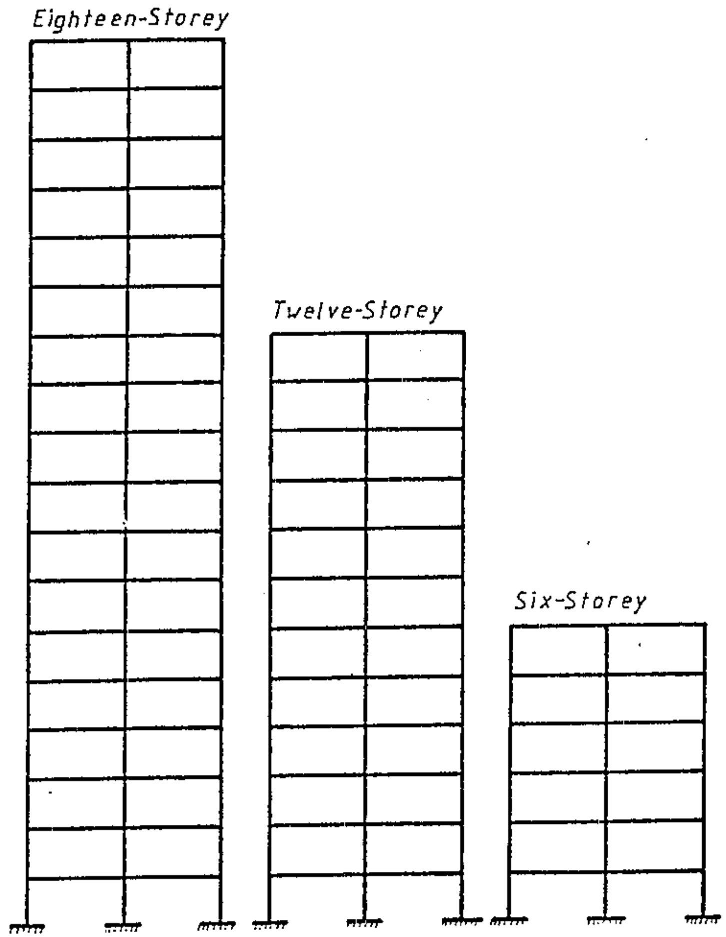

Fig.1.1 Six, Twelve and Eighteen-Storey Steel Moment Resisting Frames.

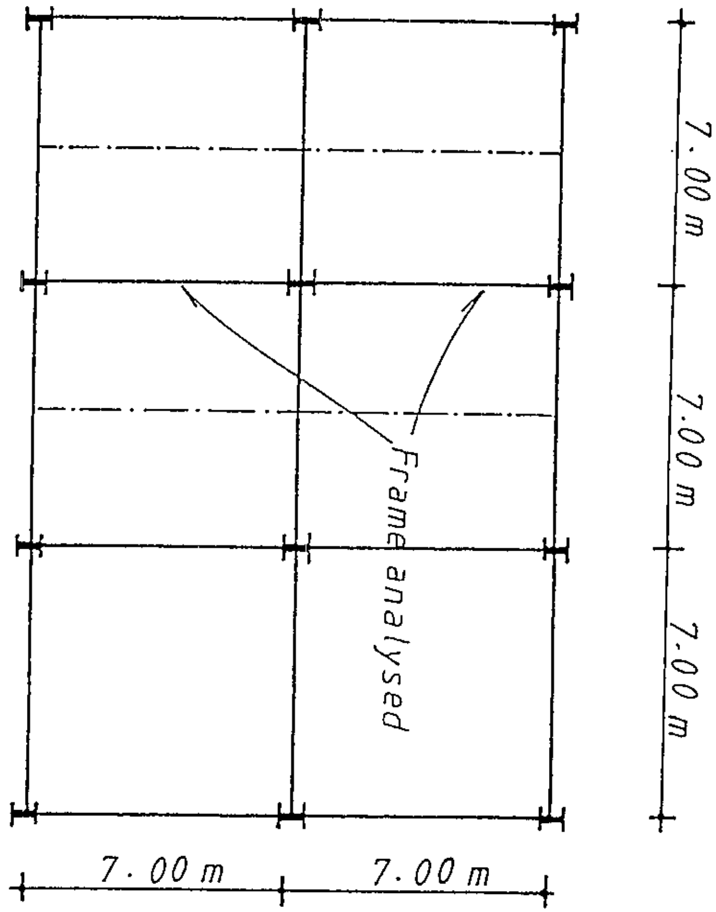

Fig.1.2 Typical Plan View.

R risk factor = 1.0

μ structure ductility = 3.0

A ductility factor of 3.0 for steel moment resisting frames was suggested by the New Zealand Study Group for Steel Structures [18]. The Six, Twelve and Eigteen-storey frames were designed with various design drift limits. The design drift limit was defined as the ductility factor, μ, times the elastic drift from the equivalent static load, as a prediction of the inelastic drift under seismic loading [15].

The following load combinations were used:

1.4 D

1.2 D + 1.6 Lr

1.2 D + 1.2 Ls + E

0.9 D + E

The weight of the floor slab was 2.7 kPa with 0.5 kPa for finishing and combined with self weight of the member gave total uniform dead load of 23.4 kN/m. Live load for general use Lu = 2.5 kPa, serviceability live load Ls = 0.8 kPa. The frames data are shown in Appendix A.

Steel frames, by their very nature, are usually more flexible and therefore have longer fundamental periods than concrete frames. This gives an advantage in reducing the response of the frames under certain types of earthquakes, while in the other cases they could experience the peak response of an earthquake with long period. The critical combination of design load for medium to high rise buildings is usually the combination of gravity load with horizontal earthquake load or with the wind load. Frames were expected to survive without non-structural damage at the design earthquake load. Instability in steel frames may be caused by the failure to achieve moment capacity, excessive joint rotation, storey column hinging mechanisms and local or lateral buckling which results in strength degradation.

To achieve low design drifts in the design some of the above loading combinations become non-critical. This design excluded consideration of the load combinations with wind load. To maintain the column stability, ratio of axial force to compression yield force, P/Py, was considered less than 0.4 as suggested by Buen [16] or P/Py less than 0.5 as suggested by New Zealand Study Group for Steel Structures [2]. The stiffness of both beams and columns should be increased proportionally to achieve a lower drift limit without allowing plastic hinges to shift to the exterior columns. Some hinges in the interior columns may be allowed but will lead to significant increase in plastic hinge rotations and will distribute significant additional moment to the exterior columns, which may then lead to column hinging.

Flange and web slenderness ratios of beams and columns were chosen within the limits as suggested by the New Zealand Study Group for Steel Structures.

1.5 EARTHQUAKE ACCELERATION RECORDS

Five different earthquake acceleration records were chosen to study the effects of P-delta and the general behaviour of the frames designed to DZ4203 loading criteria under different intensities and characteristics of earthquake motions. The designed frames with various design drifts and fundamental periods were analysed under the following earthquakes:

- Bucharest N-S, 1977 (corrected)
- El Centro N-S, 1940 (corrected)
- Artificial NZ4203 for seismic zone A
- Pacoima Dam S14W, 1971
- Parkfield N65E, 1966

The El Centro 1940 and Artificial NZ4203/A were used as the design level earthquakes for New Zealand seismic zone A. Bucharest, Parkfield and Pacoima Dam were used to gain a complete understanding of the response of the frames under severe earthquakes.

The El Centro, 18 May 1940, North-South component (corrected) with Richter Magnitude of 6.4 and peak ground acceleration of 0.34g, was recorded by the accelerograph which was located 9 km from the epicentre. This earthquake is a vibratory, moderately strong ground excitation. The intensity is not stronger than some of the records of the later earthquakes. This Imperial Valley, California earthquake is used as a base of many seismic design codes including NZS4203 or DZ4203 in determining the basic seismic coefficient. During this study, it was found that all frames behaved well under this earthquake. The first 14 seconds of the record was considered to be adequate. Previous studies have found that the inclusion of P-delta effect during the El Centro 1940 earthquake reduced the response of the structures [3].

The Artificial NZ4203/A record was generated by SIMQKE [5] to match the spectral acceleration for the New Zealand seismic zone A. The intensity is therefore slightly stronger than El Centro 1940, with 20 seconds duration, building up in the first 2 seconds and decreasing in the last 5 seconds. This artificial earthquake can be categorized as a long duration shaking.

The Bucharest earthquake, 4 March 1977, North-South component (corrected), has a long period ground motion with two strong pulses around 3.5 seconds and could significantly affect buildings with long fundamental periods such as medium or high-rise steel moment resisting frames.

The Parkfield earthquake, 27 June 1966, North 65^{o} East component, occured on the San Andreas fault near Parkfield in central California. This earthquake which has a Richter Magnitude of 5.6 with triple peaks at duration of 1.5 seconds is an impulsive earthquake with a peak ground acceleration of 0.50 g. The accelerograph was located at 32 km from the epicentre.

The Pacoima Dam earthquake, 9 February 1971, South 14^{o} West component, peak acceleration of 1.2 g was a very strong ground acceleration record of long duration and of vibratory nature, recorded 9 km from the epicentre of the 1971 San Fernando earthquake. The accelerograph at the Dam site and the effect of the topography raised questions about the recorded accelerogram. Study of the effect indicated that the error was less than 25% of the recorded ground motion.

The vertical component (uncorrected) of El Centro 1940 was used together with the above mentioned horizontal acceleration El Centro 1940 record, to study the influence of vertical acceleration on the second order effects. A previous study [3] showed that this vertical acceleration reduced the response of the structures.