

2000

AMES

[Handwritten signature]

Submitted to the Faculty of Engineering
Ain Shams University

For the Degree of Master of Science
In Structural Engineering

۱۰۰

OSAMA MOHAMED TAWFIC

B.Sc. Structural Division (1967)

624.9
0.7

5222

فانار عبدالرحمن

A C K N O W L E D G E M E N T S

The research work performed in this thesis has been done under the supervision of the late Prof.Dr. M.H. Abbas, Prof.Dr. A. Shaker and Dr. S.A. Saafan. For their supervision, help and valuable guidance, as well as for their constant encouragement, the author wishes to express his deep gratitude.

The author would like to express his thanks to the staff of the scientific computation centre, Ain Shams University for their cooperation.



INTRODUCTION	1
--------------	---

CHAPTER I

REVIEW OF PREVIOUS WORK

1.1	The First Yield Theory	3
1.2	The Rigid Plastic Load	4
1.3	The Elastic Critical Load	6
1.4	The Rankine-Merchant-Load	7
1.5	The Elastic-Plastic Load	10
1.6	Analysis of Structures by Digital Computers	12
	a) Introduction	12
	b) Computer main units	13
	i - Input Unit	13
	ii - Control Unit	14
	iii - Storage Unit	14
	iv - Logical Arithmetic Unit	15
	v - Output Unit	15

CHAPTER II

LINEAR ANALYSIS OF STRUCTURES BY DIGITAL COMPUTER

2.1	Sign Conventions	17
2.2	Basic Equations	17

2.3	Stiffness Coefficients for all types of Structural Frame Member	25
a)	Rigid Jointed Members	25
b)	Member pin jointed at one end and rigid jointed at the other	26
c)	Pin jointed member	26

COMPUTER PROGRAM

2.4	Basic Information Block	27
a)	General Informations	27
b)	Joints Informations	27
c)	Members Informations	28
2.5	Joint Equations of Equilibrium Block	28
a)	Building up member coefficients	28
b)	Building up equations	28
c)	Solving equations	29
2.6	Output Block	30
2.7	Numerical Example	30

CHAPTER III

ELASTIC STABILITY BASED ON SMALL DEFLECTION THEORY

3.1	Introduction	35
3.2	Assumptions	36

3.3	Elastic Critical Load	37
3.4	Iterational Determination of Elastic Critical Load	37
3.5	Experimental Determination	40
3.6	Stability Functions	41
	a) Stiffness and carry over factors S&C	41
	b) Sway function m	45
	c) Bowing of Deformed Members	46
3.7	Stiffness Matrix	50
3.8	Computer Program	57
<u>CHAPTER IV</u>		
<u>ELASTIC CRITICAL LOAD OF MULTI-BAY PORTAL FRAMES</u>		
4.1	Single bay portal Frames	62
4.2	Two bay portal Frames	63
4.3	Five bay portal Frames	63
4.4	Discussion of Results	70
<u>CHAPTER V</u>		
<u>ELASTIC CRITICAL LOAD OF MULTI-BAY GABLE FRAMES</u>		
5.1	Single Bay Gable Frames	71
5.2	Two Bay Gable Frames	72
5.3	Five Bay Gable Frames	72
5.4	Discussion of Results	78

CHAPTER VI

MULTI-BAY GABLE FRAMES WITH VARIABLE INERTIA OF COLUMN

6.1	Single Bay Gable Frames	80
6.2	Two Bay Gable Frames	80
6.3	Three Bay Gable Frames	81
6.4	Discussion	

CHAPTER VII

GABLE-FRAMES WITH CRANE LOADS

7.1	Effect of the Crane Loads	98
7.2	Effect of the Stiffness Ratio of the Rafter to the Column	98
7.3	Effect of the Variation of the Column Cross-section	98
7.4	Discussion	115

CHAPTER VIII

OPTIMUM SLOPE OF THE RAFTER

8.1	Slope 15°	116
8.2	Slope $22\frac{1}{2}^\circ$	117

	TABLE
8.3 Slope 30°	117
8.4 Slope 45°	117
8.5 Slope 60°	118
8.6 DISCUSSION	126
C O N C L U S I O N	127
APPENDIX I : REFERENCES	129
APPENDIX II : NOTATIONS USED IN THE PROGRAM	132
APPENDIX III: LINEAR ANALYSIS PROGRAM FOR PLANE FRAMES.	134

-----e0oe-----

INTRODUCTION

For a long time all calculations of structures were based on Hooke's law. The nonlinear analysis of structural frames is receiving considerable attention in the present research work. The rigid plastic theory because of its simplicity has become to be a rational method of design although the stability of the structure is overlooked.

The natural growth of design methods has led to comparing the behaviour of columns in structures with that of isolated columns. In building codes of design it is assumed that the effective length is a constant fraction of the column length due to the stiffness of the beam to column connections. These methods of design are very empirical of unknown accuracy. The reason is that the effective lengths are dependent on the stiffness of the members which in turn is dependent on the axial load they carry and is not constant. The columns should be designed as a part of a structure and not by analogy with the behaviour of isolated columns.

The object of this thesis is to introduce a study on the elastic critical load of portal and gable frames

and to put before him to the designer the curves and tables which give accurate data about the stability of these frames. Thus enabling the designer to carry out his design on conceptual basis.

A brief summary of the theory of failure of structural frames and description of electronic digital computers are introduced in Chapter I.

Chapter II is devoted for the linear analysis of structures by digital computer. The program has been further developed in Chapter III to include the effects of the instability caused by axial loads.

Chapter IV is concerned with the elastic sway critical load of multi-bay portal frames.

In Chapter V, VI, & VII a thorough study is introduced on the sway critical load of gable frames. The influence of the number of bays, the variable inertia of columns, the relative stiffness of the rafter to the column and the effect of the crane loads on the sway critical load are investigated.

A study on the optimum slope of the rafter of the gable frames is given in Chapter VIII.

The results are summarized in the conclusions.

DESIGN OF FRAMED JOINTS

1.1 The First Yield Theory.

The considered starting point was almost Hook's law. The first yield load is that corresponding to straining actions, distributions for which the most highly stressed fibers are subjected to the yield stresses, Fig. 1.1a .

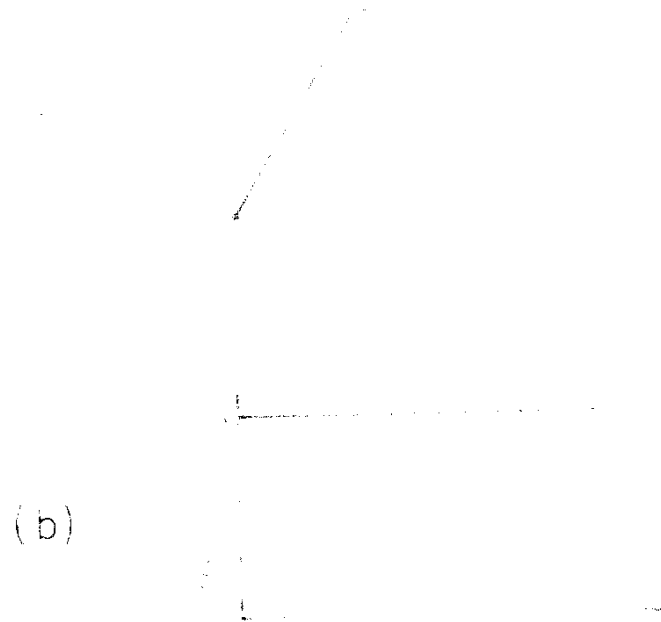
Apart from Euler's work on the buckling of columns, the early investigations in this field were concerned predominantly with the distribution of stresses of structures in the absence of buckling effects.

In fact since the results of elastic analysis cease to be applicable when the yield stress is attained at the most highly stressed cross-section, elastic design methods are of necessity based on the assumption that a frame will become unsafe when yield first occurs and a margin of safety is insured by specifying a working stress which is well below the yield limit.

It has long been known that a determinate steel frame has a greater load-carrying capacity than indicated by the allowable-stress concept. Such frames are able to carry increased loads above the yield value because structural steel has the capacity to yield. Thus the allowable - stress concept has overemphasized the importance of stress rather than strength as a guide in engineering design.

1.2 The Rigid Plastic Load.

The plastic methods of structural analysis have been developed for the purpose of calculating the plastic collapse load for frames, with the ultimate objective of establishing a rotational and economical design procedure. It might be thought that any attempt to evaluate the plastic collapse load for a given frame would require an investigation of the complete behaviour of the frame between the first yielding and its ultimate collapse. However, it is a remarkable fact that if buckling effects are absent it is possible to calculate the collapse



load directly by the plastic methods without considering the intervening elastic-plastic range. When the rigid plastic load is just attained the frame becomes a mechanism. For such method of calculation the stress strain relation is a straight line indicating the yield stress at zero strain, Fig. 1.1b .

1.3 The Elastic Critical Load.

It is the load at which the stiffness of the structure as a whole comes to zero. Herein the stress-strain curve is assumed to be completely elastic, Fig. 1.1c . There are several methods to obtain the elastic critical load of a structure such as the mathematical analysis, the dynamic and the experimental methods. A simplified method has been introduced to obtain the elastic critical loads for rectangular multi-story frameworks by Merchant (1955), Chandler (1955). It was later generalized to include pitched roof frames by Merchant-Saafan (1962).

1.4 Rankine-Mercant Load.

The failure load P_F is lower than either the Euler load P_E or the plastic limit load P_p . Hence $P_F/P_E < 1.0$ and $P_F/P_p < 1.0$. This may be expressed by stating that the solution of any practical strut must lie within the area OABC in Fig. 1.2-a. The straight line AC gives the following relationship.

$$\frac{1}{P_F} = \frac{1}{P_E} + \frac{1}{P_p} \quad \dots\dots (1.1a)$$

$$\frac{P_F}{A} = \frac{\sigma_y}{1.0 + \frac{\sigma_y}{\pi^2 E} \left(\frac{\ell}{r}\right)^2} \quad \dots\dots (1.1b)$$

where :

- P = Failure load of structure;
- P_p = Load obtained from plastic theory;
- P_E = Euler load of stanchion;
- A = Cross-sectional area for stanchion;
- ℓ = Length;
- r = slenderness ratio.